COMPARISON OF RECENT NEW ZEALAND AND UNITED STATES SEISMIC
DESIGN PROVISIONS FOR REINFORCED CONCRETE BEAM-COLUMN JOINTS
AND TEST RESULTS FROM FOUR UNITS DESIGNED ACCORDING TO THE
NEW ZEALAND CODE

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SYNOPSIS

A comparison is made of the seismic design provisions for reinforced concrete beam-column joints required by the new New Zealand concrete design code NZS 3101 and recently proposed United States procedures. Large differences are shown to exist between these new provisions of the two countries. Results are reported of cyclic load tests which were conducted according to the requirements of the new NZS 3101. The test results showed that location of plastic hinges in beams away from the column faces may be of considerable advantage in the design of joints, when member sizes are small and joint shears are high, due to less congestion of reinforcement and better anchorage conditions.

INTRODUCTION:

The procedures for the seismic design of reinforced concrete beam-column joints given in the new New Zealand concrete design code NZS 3101 (1) are based on a considerable amount of test evidence accumulated through the years and on behavioural models for joint core shear resistance based on those test results. A summary of tests conducted in New Zealand on reinforced concrete beam-column joints is given elsewhere(2). The mechanisms of joint core shear resistance on which the code equations are based are also described elsewhere(1,2,3,4). It is of interest that current proposals for the revision of codes in the United States show that a greatly different approach to the seismic design of beam-column joints is being adopted in that country. The United States approach leads to less transverse reinforcement in the joint core in some cases and to more in other cases. The differences between the New Zealand and United States approaches are of interest, particularly since difficulties are often experienced in placing the amount of shear reinforcement required by codes in joint cores.

This paper first sets out a comparison between the New Zealand and the recently proposed United States approaches for the design of reinforced concrete beam-column joints. The results of some recent tests conducted at the University of Canterbury on four reinforced concrete beam-column joints are then described. The four test specimens were designed to illustrate various design approaches permitted by NZS 3101, including the concept of locating the plastic hinge away from the joint core, and to examine possible conservatism in the New Zealand approach.

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COMPARISON OF RECENT NEW ZEALAND AND
UNITED STATES SEISMIC DESIGN PROVISIONS
FOR REINFORCED CONCRETE BEAM-COLUMN JOINTS:

The new New Zealand code NZS 3101 (1) contains detailed provisions for the seismic design of reinforced concrete beam-column joints. Currently in the United States drafts are available of proposed revisions to the existing Appendix A of ACI 318-77(5) which give special provisions for seismic design. Also, ASCE-ACI Committee 352 on Joints and Connections in Monolithic Concrete Structures, which published its last report in 1976(6), is currently revising its recommendations(7). These two proposed United States procedures are similar in approach and the proposed ASCE-ACI Committee 352 approach is compared in detail with the New Zealand approach below.

Approach of New Zealand Code NZS 3101(1)

(i) Design Assumptions

The NZS 3101 code provisions are intended to ensure that joints are designed in such a way that when inelastic lateral displacements occur in ductile frames the required energy dissipation occurs in the potential plastic hinge regions of the adjacent members and not in the joint core regions. Accordingly, the joint core should be designed to resist the forces arising when the overstrength of the framing members is developed. That is, the stresses in the flexural steel at the plastic hinges are assumed to be 1.25 times the specified yield strength in the case of Grade 275 steel, or 1.4 times the specified yield strength in the case of Grade 380 steel. The design horizontal shear force V_{jh} and the design vertical shear force V_{jv} are found by rational analysis taking into account the effect of all the forces acting on the joint. When beams frame into the joint in two directions, these forces need only be considered in

each principal direction independently.

In determining the shear strength of the joint core the strength reduction factor ϕ is taken as unity. The shear applied to the joint core is assumed to be carried by a mechanism consisting of a concrete diagonal compression strut and a mechanism consisting of truss action from a concrete diagonal compression field and the shear reinforcement. The first mechanism is commonly referred to as the "shear carried by the concrete" and the second as the "shear carried by the shear reinforcement". Shear reinforcement is detailed to carry the design shear forces in excess of those carried by the concrete.

In order to prevent the concrete diagonal compression strut from crushing, the nominal horizontal shear stress v_{jh} in either principal direction is limited to $1.5\sqrt{f'_c}$ MPa, where

$$v_{jh} = \frac{v_{jh}}{b_j h_c} \quad (1)$$

The effective joint width, b_j , is defined as

a) when $b_c > b_w$

either $b_j = b_c$ or $b_j = b_w + 0.5h_c$,
whichever is smaller.

b) when $b_c < b_w$

either $b_j = b_w$ or $b_j = b_c + 0.5h_c$,
whichever is the smaller.

(ii) Horizontal Joint Shear

The total area of horizontal shear reinforcement placed between the outermost layers of top and bottom beam reinforcement is required to be not less than

$$A_{jh} = V_{sh}/f_{yh} \quad (2)$$

where the horizontal design shear force to be resisted by this shear reinforcement is given by

$$V_{sh} = V_{jh} - V_{ch} \quad (3)$$

In Eq. 3, V_{ch} should be taken as zero unless one of the following situations applies:

(a) When the minimum average compressive stress on the gross concrete area of the column above the joint exceeds $0.1 f'_c/C_j$

$$V_{ch} = \frac{2}{3} \sqrt{\frac{C_j P_e}{A_g} - \frac{f'_c}{10}} (b_j h_c) \quad (4)$$

(b) When the design is such that plastic hinging occurs in the beam at a distance away from the column face not less than the beam depth nor 500 mm, or for external joints

where the flexural steel is anchored outside the column core in a beam stub, the value of V_{ch} may be increased to

$$V_{ch} = 0.5 \frac{A'_s}{A_s} V_{jh} \left(1 + \frac{C_j P_e}{0.4 A_g f'_c}\right) \quad (5)$$

where A'_s/A_s should not be taken larger than 1.0. When the axial column load results in tensile stresses over the gross concrete area exceeding $0.2f'_c$, $V_{ch} = 0$. For axial tension between these limits V_{ch} may be obtained by linear interpolation between zero and the value given by Eq. 5 when P_e is taken as zero.

(c) For external joints without beam stubs at the far face of column, Eq. 5 may be used when multiplied by the factor

$$\frac{3h_c (A_{jv} \text{ provided})}{4h_b (A_{jv} \text{ required})} \quad (5a)$$

which should not be taken as greater than 1.0. Use of this factor requires that the beam bars be anchored using a 90° standard hook in the joint core in accordance with the relevant code section.

(d) When the ratio h_c/h_b is greater than or equal to 2.0, V_{ch} need not be taken as less than

$$V_{ch} = 0.2b_j h_c \sqrt{f'_c} \quad (6)$$

(iii) Vertical Joint Shear

The total area of vertical shear reinforcement, normally in the form of intermediate column bars on the side faces of the column crossing the critical corner to corner diagonal tension crack, should not be less than

$$A_{jv} = V_{sv}/f_y \quad (7)$$

where the vertical design shear force to be resisted by this shear reinforcement is

$$V_{sv} = V_{jv} - V_{cv} \quad (8)$$

In Eq. 8, V_{cv} is given by

$$V_{cv} = \frac{A'_{sc}}{A_{sc}} V_{jv} \left[0.6 + \frac{C_j P_e}{A_g f'_c}\right] \quad (9)$$

except where axial load results in tensile stresses over the column section. When P_e is tensile, value of V_{cv} is interpolated linearly between the value given by Eq. 9 when P_e is taken as zero and zero when the axial tensile stress over the gross concrete area is $0.2f'_c$.

However, if plastic hinges are expected to form in the column above or below the joint core, but not when elastic behaviour is assured in the column or

column stub on the opposite side of the joint, V_{cv} should be taken as zero for any axial c_v load on the column.

The spacing of vertical shear reinforcement in each plane of any beam framing into the joint should not exceed 200 mm and in no case should there be less than one intermediate bar in each side of the column in that plane.

(iv) Confinement

The horizontal transverse confinement reinforcement in the joint core should not be less than that required in the potential plastic hinge regions in the adjacent columns. Thus for columns with hoops and supplementary cross ties the total area of transverse steel in each of the principal directions of the cross section should be at least equal to

$$A_{sh} = 0.3s_h h'' \left(\frac{A_g}{A_c} - 1 \right) \frac{f'_c}{f_{yh}} \left(0.5 + 1.25 \frac{P_e}{\phi f'_c A_g} \right) \quad (10)$$

but not less than

$$A_{sh} = 0.12s_h h'' \frac{f'_c}{f_{yh}} \left(0.5 + 1.25 \frac{P_e}{\phi f'_c A_g} \right) \quad (11)$$

However if the joint has beams framing into all four column faces and is designed using the conditions applicable for Eq. 5, the transverse reinforcement in the joint core may be reduced to one-half of that required by Eqs. 10 and 11.

In no case shall the spacing of transverse reinforcement in the joint core exceed 10 times the diameter of the longitudinal column bar or 200 mm, whichever is less.

(v) Bar Anchorage in Interior Joints

To keep bond stresses to an acceptable level, the diameters of longitudinal bars d_b passing through a joint core are limited as follows:

Beam bars:

When plastic hinging can occur adjacent to the column face:

$$d_b \leq h_c/25 \text{ when } f_y = 275 \text{ MPa, or}$$

$$d_b \leq h_c/35 \text{ when } f_y = 380 \text{ MPa.}$$

When plastic hinging is located at a distance from the column face of at least the beam depth or 500 mm, whichever is less:

$$d_b \leq h_c/20 \text{ when } f_y = 275 \text{ MPa, or}$$

$$d_b \leq h_c/25 \text{ when } f_y = 380 \text{ MPa.}$$

Column bars:

When columns are intended to develop plastic hinges:

$$d_b \leq h_b/20 \text{ when } f_y = 275 \text{ MPa, or}$$

$$d_b \leq h_b/25 \text{ when } f_y = 380 \text{ MPa.}$$

When columns are not intended to develop plastic hinges:

$$d_b \leq h_b/15 \text{ when } f_y = 275 \text{ MPa, or}$$

$$d_b \leq h_b/20 \text{ when } f_y = 380 \text{ MPa.}$$

(vi) Bar Anchorage at Exterior Joints

The basic development length of a deformed bar in tension terminating with a standard 90° hook is

$$l_{hb} = \frac{66d_b}{\sqrt{f'_c}} \frac{f_y}{275} \quad (14)$$

Where the bar diameter is 32 mm or smaller with side cover not less than 60 mm and cover on tail extension not less than 40 mm, the value may be reduced to $0.7l_{hb}$, or where the concrete is suitably confined the value may be reduced to $0.8l_{hb}$.

The basic development length for a deformed bar in compression is

$$l_{db} = 0.24d_b f_y / \sqrt{f'_c} \quad (15)$$

$$\text{but not less than } 0.044d_b f_y. \quad (15a)$$

Where the concrete is suitably confined the value may be reduced to $0.75l_{db}$.

The anchorage is considered to commence within the column at distance $0.5h_c$ or $10d_b$ from the column face, whichever is less, except that when the plastic hinge is located away from the column face anchorage may be considered to commence at the column face.

Draft Approach of ASCE-ACI Committee 352 (7)

(i) Design Assumptions

The draft revisions of ASCE-ACI Committee 352(7) have adopted a fundamentally different approach to the whole problem of joint shear, which is also similar in principle to that proposed in the draft Appendix A of the 1983 revision of the building code of the American Concrete Institute, ACI 318. In the existing ACI and ASCE recommendations (5,6) the approach was similar, although more simplistic, to that used in New Zealand. The draft ASCE-ACI Committee 352 proposals are reviewed below.

Provisions are given for two types of joints, essentially differentiating between joints expected to be subjected to cyclic inelastic deformations (Type 2) and those not (Type 1). The requirements for Type 2 joints only will be reviewed. Only horizontal joint shear is considered in the approach.

(ii) Horizontal Joint Shear

The forces in the reinforcing bars of the beams acting at the joint core boundaries are determined assuming that the steel stress is 25% greater than the specified yield strength, regardless of the grade of steel. For joints with beams framing in from two perpendicular directions the horizontal shear in the joint is checked independently in each direction. The design horizontal shear force, V_u , is computed for the horizontal plane at midheight of the joint by considering the shear forces on the boundaries of the free body of the joint and the normal tension and compression forces in the members framing into the joint.

The calculated value of V_u should satisfy

$$V_u \leq \phi \gamma \sqrt{f'_c} b_c h_c \quad (16)$$

where ϕ is the strength reduction factor for shear taken as 0.85, and b_c and h_c are the gross width and thickness of the column, respectively. However, the value for b_c in Eq. 16 should not be taken as greater than twice the width of the beam framing into the joint. The value for f'_c used in Eq. 16 should not be taken as larger than 34 MPa. The value of γ depends on the joint configuration and is 1.33 for an interior joint, 1.00 for an exterior joint, and 0.67 for other joints. To be classified as an interior joint, members must frame into all four sides of the joint and cover at least three-quarters of the width and depth of the joint face. To be classified as an exterior joint, members must frame into three sides of the joint and the width and total depth of the beams on opposite faces of the joint must not vary by more than 25%.

(iii) Vertical Joint Shear

No calculation procedure is recommended to check resistance for vertical joint shear forces.

(iv) Confinement

Where rectangular hoop and cross tie transverse reinforcement is used, the total area of a single or overlapping hoops, or hoops with cross ties of the same size, in each direction should be at least equal to

$$A_{sh} = 0.3 \frac{s_h h' f'_c}{f_{yh}} \left(\frac{A_g}{A_c} - 1 \right) \quad (17)$$

but not less than

$$A_{sh} = 0.09 \frac{s_h h' f'_c}{f_{yh}} \quad (18)$$

For interior joints, the required transverse steel can be one-half of that required by Eqs. 17 and 18. The hoop spacing s_h should not exceed one quarter

of the minimum column dimension, 6 times the diameter of the longitudinal bar or 200 mm, but need not be less than 150 mm.

The centre-to-centre spacing between adjacent longitudinal column bars should not exceed the larger of 200 mm or one-third of the column cross section dimension in that direction.

(v) Bar Anchorage

The diameter of all straight bars passing through joints should be selected such that for beam bars $d_b \leq h_c/24$, and for column bars $d_b \leq h_b/24$.

The development length of a bar terminating with a standard 90° hook is given by

$$l_{dh} = 0.21 f_y d_b / \sqrt{f'_c} \quad (19)$$

but not less than $8d_b$ or 150 mm, whichever is greater. Bar diameters should not exceed 35 mm and hooks should be situated in the column core located as far from the critical section as possible. If the confinement steel spacing does not exceed $3d_b$, l_{dh} may be reduced by 20%. The anchorage is considered to commence at the edge of the concrete core.

Comparison of the NZS 3101 and the Draft ASCE-ACI Committee 352 Approaches

The main differences:

There are large differences in the approaches to joint core shear design adopted in NZS 3101 and in the draft ASCE-ACI Committee 352 procedures.

The NZS 3101 requirements are based on a rational model for the mechanisms of shear resistance of the joint core, namely a mechanism consisting of a concrete diagonal strut and a mechanism consisting of truss action of a concrete diagonal compression field and shear reinforcement. Account is taken of the reduced capacity of the diagonal compression strut mechanism, particularly in interior joints, when plastic hinging forms adjacent to the core faces and results in full depth flexural cracking there during reversed loading. Increased concrete shear capacity and less severe bond and anchorage criteria are permitted if plastic hinging is forced to occur away from the joint core faces. Both horizontal and vertical shear reinforcement are designed to carry that shear in excess of the concrete capacity.

The draft ASCE-ACI Committee 352 approach assumes that providing the design horizontal shear force on the joint core does not exceed a quantity

$$\phi \gamma \sqrt{f'_c} b_c h_c,$$

the amount of transverse reinforcement required for column confinement, reduced by one half in those cases where the joint is adequately confined by structural members on all four faces, will also be adequate for

shear resistance in the joint core. That is, once the size and spacing of transverse reinforcement in the potential plastic hinge regions in the ends of the column have been established, that quantity or one half is continued through the joint core. This approach has been adopted evidently because Meinheit and Jirsa (8) have concluded that shear strength of joint cores was not as sensitive to joint core shear reinforcement as is implied in the earlier report by ASCE-ACI Committee 352(6).

In the view of the authors, this ASCE-ACI Committee 352 approach is largely empirical and is too simplistic. It does not apply to the design of joints with unusual configurations, it makes little distinction between interior and exterior joints, and it does not allow for the difference in performance of joints with plastic hinges adjacent to or removed from the joint core. When compared with the NZS3101 approach it is conservative in some cases and unconservative in others.

The lack of a calculation procedure for vertical joint steel in the draft ASCE-ACI Committee 352 approach may well be offset by the requirement that at least an eight bar column be used. However, the amount of vertical shear reinforcing required may be greater than that provided to satisfy column flexural demand.

The anchorage requirements of the draft ASCE-ACI Committee 352 approach are considerably less severe for bars passing through joints than those of NZS 3101. For the case of exterior joints, anchorage is considered to commence at the surface of the concrete core. That is, loss of bond in the cover concrete only is assumed.

Examples of Comparisons of Joint Shear Reinforcement

- (1) Comparison of an interior joint with beams on all four faces.

Draft ASCE-ACI Approach

For a large column, if $h'' = 0.9b_c$, from Eq. 18

$$A_{sh} = 0.5 \times 0.09 \frac{s_h 0.9b_c f'_c}{f_{yh}}$$

$$\therefore A_{sh} f_{yh} / s_h = 0.0405 b_c f'_c \quad (i)$$

Also from Eq. 16, the shear strength of the joint core is

$$V_u = 0.85 \times 1.33 \sqrt{f'_c} b_c h_c$$

$$= 1.13 \sqrt{f'_c} b_c h_c \quad (ii)$$

That is, the amount of joint shear reinforcement given by Eq. i should satisfy the horizontal joint shear force imposed in Eq. ii.

NZS 3101 Approach

- (a) For plastic hinge forming adjacent to joint core:

If $P_e / f'_c A_g = 0.1$, from Eq. 4 $V_{ch} = 0$

Design joint horizontal shear force, from Eqs. 2, 3 and 4 and assuming that

$$A_{jh} = A_{sh} h_b / s_h, \text{ is}$$

$$V_{jh} = V_{sh} = A_{jh} f_{yh} = A_{sh} f_{yh} h_b / s_h$$

If $A_{sh} f_{yh} / s_h$ from Eq. i is substituted

$$V_{jh} = (0.0405 b_c f'_c) h_b$$

$$= (0.0405 \sqrt{f'_c} \frac{h_b}{h_c}) \sqrt{f'_c} b_c h_c$$

Say $f'_c = 25 \text{ MPa}$ and $h_b / h_c = 1$, then $V_{jh} = 0.203 \sqrt{f'_c} b_c h_c$

But from Eq. ii, draft ASCE-ACI approach would allow

$V_{jh} = 1.13 \sqrt{f'_c} b_c h_c$, which is 5.6 times the V_{jh} allowed by NZS 3101.

- (b) For plastic hinge forming away from the joint core:

If $P_e / f'_c A_g = 0.1$, $A'_s = A_s$ and

$C_j = 0.5$, from Eq. 5

$$V_{ch} = 0.5 V_{jh} (1 + \frac{0.5 \times 0.1}{0.4}) = 0.536 V_{jh}$$

$\therefore$ Design horizontal shear force, from Eqs. 2, 3 and 5 and assuming that

$$A_{jh} = A_{sh} h_b / s_h$$

$$V_{jh} = 0.563 V_{sh} + V_{sh}$$

$$\therefore V_{jh} = 2.29 V_{sh} = 2.29 A_{sh} f_{yh} h_b / s_h$$

If $A_{sh} f_{yh} / s_h$ from Eq. i is substituted

$$V_{jh} = 2.29 \times 0.0405 b_c f'_c h_b$$

$$= (0.0927 \sqrt{f'_c} \frac{h_b}{h_c}) \sqrt{f'_c} b_c h_c$$

Say $f'_c = 25 \text{ MPa}$ and $h_b / h_c = 1$, then $V_{jh} = 0.463 \sqrt{f'_c} b_c h_c$

But from Eq. ii, draft ASCE-ACI approach would allow $V_{jh} = 1.13 \sqrt{f'_c} b_c h_c$ which is 2.4 times the V_{jh} allowed by NZS 3101.

Note: For higher axial load levels than $0.1 f'_c A_g$, more shear will be carried by the concrete in the NZS 3101 approach than in the above example and the difference between the two approaches would be reduced.

- (2) Comparison of a corner joint.

Draft ASCE-ACI Approach

$A_{sh} f_{yh} / s_h = 0.081 b_c f'_c$ (i.e. twice that for the interior joint)

$$V_u = 0.85 \times 0.67 \sqrt{f'_c} b_c h_c = 0.57 \sqrt{f'_c} b_c h_c$$

NZS 3101 Approach

- (a) For plastic hinges forming adjacent to joint core without a beam stub.

$$\text{If } P_e / f'_c A_g = 0.1, A'_s = A_s,$$

$$C_j = 0.5, h_c = h_b,$$

and A_{jv} provided = A_{jv} required,
from Eqs. 5 and 5a,

$$V_{ch} = \left[0.5 V_{jh} \left(1 + \frac{0.5 \times 0.1}{0.4} \right) \right] \times \frac{3}{4} = 0.422 V_{jh}$$

$$\therefore V_{sh} = V_{jh} - V_{ch} = 0.578 V_{jh}$$

$$\therefore V_{jh} = 1.73 V_{sh} = 1.73 (0.081 b_c f'_c) h_b$$

$$= (0.140 \sqrt{f'_c} \frac{h_b}{h_c} \sqrt{f'_c} b_c h_c)$$

Say $f'_c = 25$ MPa and $h_b/h_c = 1$,

$$\text{then } V_{jh} = 0.70 \sqrt{f'_c} b_c h_c$$

But draft ASCE-ACI approach would allow $V_{jh} = 0.57 \sqrt{f'_c} b_c h_c$,

which is 0.81 times the V_{jh} allowed by NZS 3101.

- (b) For plastic hinges forming away from the joint core.

$$\text{If } P_e / f'_c A_g = 0.1, A'_s = A_s, C_j = 0.5,$$

and $h_c = h_b$, from Eq. 5

$$V_{ch} = 0.5 V_{jh} \left(1 + \frac{0.5 \times 0.1}{0.4} \right) = 0.563 V_{jh}$$

$$\therefore V_{jh} = 2.29 V_{sh} = (0.185 \sqrt{f'_c} \frac{h_b}{b_c}) \sqrt{f'_c} b_c h_c$$

Say $f'_c = 25$ MPa and $h_b/h_c = 1.0$,
then $V_{jh} = 0.927 \sqrt{f'_c} b_c h_c$

But draft ASCE-ACI approach would allow $V_{jh} = 0.57 \sqrt{f'_c} b_c h_c$,

which is 0.62 of the V_{jh} allowed by NZS 3101.

Note: Again for higher axial load levels than $0.1 f'_c A_g$, more shear would be carried by the concrete in the NZS 3101 approach.

TEST PROGRAMME:

Details of the Beam-Column Joint Units

Four reinforced concrete beam-column joint units were tested. Two were interior joints (Units 1 and 2) and two were exterior joints (Units 3 and 4). The units were designed according to the seismic provisions of NZS 3101 (1) to illustrate possible approaches permitted by that code. The units were tested

under simulated seismic loading to compare the performance resulting from the different design approaches.

The overall dimensions of the units are shown in Fig. 1. The size of the cross sections may be taken as being representative of about $\frac{1}{2}$ to $\frac{3}{4}$ of that of full scale members of a multistorey building frame. The units can be regarded as being that part of the joint regions of a plane frame between the midspan of the beams and the midheight of the columns. The columns of the units were designed to be stronger than the beams so that during severe seismic type loading the plastic hinges occurred in the beams. The plastic hinges in the beams were designed to occur either at the column faces (conventional design) or away from the column faces (relocated plastic hinge design), as illustrated in Fig. 2.

The units were loaded as shown in Fig. 2 by axial loads P at the ends of the columns and by vertical loads V at the ends of the beams while the ends of the columns were held in a vertical line by the loads V on the beams the effects of earthquake loading was simulated. The load reversals were applied slowly. The axial column load was held constant at $0.1 f'_c A_g$ during the tests.

Material Properties

The concrete was from Ordinary Portland cement and graded aggregate with a maximum aggregate size of 20 mm. The concrete properties are shown in Table 1. The test units were cast in the horizontal plane and damp cured for a week after casting. The steel reinforcement had the measured yield and ultimate strengths shown in Table 2 and the stress-strain curves shown in Fig. 3 measured over a 51 mm gauge length. All reinforcing steel was of Grade 275, except for the longitudinal column steel which was of Grade 380.

Table 1 : Properties of Concrete

Unit	1	2	3	4
Slump, mm	100	80	90	90
Age at Test, days	69	49	42	38
f'_c at Test, MPa	41.3	46.9	38.2	38.9

Loading of Units 1 and 2

Loading of Units 3 and 4

All dimensions in millimetres.

Fig. 1 Dimensions of the Test Units

Fig. 3 Reinforcing Steel Measured Stress-Strain Curves

Conventional Design:

Reinforcement is provided so that required M_u is achieved with critical plastic hinge section at A.

Relocated Plastic Hinge Design:

Reinforcement is provided so that required M_u is achieved with critical plastic hinge section at B and so that yielding at A cannot occur unless the moment at B reaches its overstrength value.

Fig. 2 Loading of Test Units and Concepts for the Location of Plastic Hinges in Beams

Table 2 :

Properties of
Steel
Reinforcement

Grade of Bar	Grade 275						Grade 380	
Bar designation	R10	R12	R16	D16	D20	D24	D20	D24
Measured f_y (or f_{yh}) MPa	321	286	320	315	307	303	485	473
Measured f_u , MPa	437	414	468	463	458	453	784	767

Design Features of the Test Units

The design features of the four beam-column joints are summarized below.

Unit 1 was an interior beam-column joint with the plastic hinge regions in the beams designed to be located adjacent to the column faces (conventional design). The ratios of longitudinal steel for the beams were $\rho = \rho' = 1.75\%$, and this relatively high steel content led to considerable congestion of shear reinforcement in the joint core, as shown in Fig. 4.

Unit 2 was an interior beam-column joint with the plastic hinge regions in the beams designed to be located away from the column faces (relocated plastic hinge design). This was achieved by the beam reinforcing details shown in Fig. 5, which has two main advantages: less shear reinforcement is required in the joint core, and larger diameter beam bars are allowed. This improved design situation is because the joint core and nearby beam regions are considered to remain in the elastic range during loading. Therefore, the joint shear carried by the concrete diagonal compression strut is not considered to degrade during cyclic loading, and a longer length of beam bar is present between the positive and negative moment plastic hinge regions so that bond in the joint is not so critical. The beams of Unit 2 had approximately the same strength for end load as those of unit 1, and hence the design joint core shear forces were similar for the two units. The ratios of longitudinal steel for the beams at the critical sections 500 mm from the column faces were $\rho = \rho' = 1.32\%$ and at the column face were $\rho = \rho' = 2.04\%$. The design was such that yielding of the beam flexural reinforcement at the column faces was not expected unless a moment of 1.16 times the theoretical flexural strength, based on the measured f_y and f'_c values, was reached at the critical sections 500 mm from the column face. Units 1 and 2 can be regarded as alternative solutions to the same design problem.

Unit 3 was an exterior beam-column joint with the plastic hinge region in the beam designed to be located adjacent to the column face (conventional design) and with the beam bars anchored in a beam stub at the far face of the column. The ratios of longitudinal steel for the beam were $\rho = \rho' = 1.90\%$. The beam stub was necessary because bar anchorage was considered to commence at the mid-depth of the column and the column section was not large enough to allow anchorage within the column. The reinforcing details are

shown in Fig. 6.

Unit 4 was an exterior beam-column joint with the plastic hinge region in the beam designed to be located away from the column face (relocated plastic hinge design). This was achieved using the reinforcing details shown in Fig. 7. The ratios of longitudinal steel for the beams at the critical section 500 mm from the column face were $\rho = \rho' = 2.68\%$. The design was such that yielding of the beam flexural reinforcement at the column face was not expected unless a moment of 1.20 times the theoretical flexural strength based on the measured f_y and f'_c values was reached at the critical section 500 mm from the column face. Units 3 and 4 can be regarded as alternative solutions to the same design problem. However unit 4 was stronger due to the greater areas of longitudinal steel provided.

Theoretical Strengths of Units

In all strength calculations the strength reduction factor ϕ was taken as unity.

At the applied axial load of $0.1f'_c A_g$, the ratio of the theoretical flexural strength of the columns to that of the beams calculated using the measured material strengths were 1.55 and 1.64 for units 1 and 2, respectively. For units 3 and 4 at an axial load of $0.1f'_c A_g$ the ratio of the sum of the column theoretical flexural strengths above and below the joint to the beam theoretical flexural strength calculated using the measured material strengths were 1.78 and 1.75, respectively. The relatively high flexural strengths of the columns was partly due to the high measured yield strength of the Grade 380 steel in the columns.

The required shear strength of the joint cores, V_{jh} and V_{jv} required, calculated from the forces acting on the joint, are shown in Table 3 for units 1, 2, 3 and 4 and also for a unit S1 of a previous test (9). The ratios of $V_{jh}/\sqrt{f'_c}$ are also shown in the table, and these values do not exceed 1.5, as is required by the code. The table also shows the components of shear carried by the concrete, V_{ch} and V_{cv} provided, and hence the shear required to be carried by the shear reinforcement V_{sh} and V_{jv} required. That component can be compared with the shear strength provided by the shear reinforcement actually present, V_{sh} and V_{sv} provided. It is evident that in all cases, except in the previous unit S1,

Fig. 4 Reinforcing Details for Unit 1

Fig. 5 Reinforcing Details for Unit 2

the joint core shear code requirements were satisfied.

The ratio of beam bar diameter to column depth was 1/25.4 for unit 1 and 1/20.3 for unit 2, both of which were very close to the maximum values permitted by the code. Anchorage of other bars met the code requirements.

Table 3 : Components of Joint Core Shear Resistance
According to NZS 3101, kN

UNIT	For Horizontal Shear Forces					For Vertical Shear Forces			
	V_{jh} req'd	$\frac{V_{jh}}{\sqrt{f_c}}$	V_{ch} prov'd	V_{sh} req'd	V_{sh} prov'd	V_{jv} req'd	V_{cv} prov'd	V_{sv} req'd	V_{sv} prov'd
1	980	1.23	0	980	1030	1103	772	331	428
2	974	1.15	609	365	388	1096	767	329	428
3	545	0.64	341	204	202	614	430	184	304
4	597	0.77	373	224	252	672	470	202	304
Sl	966	1.34	178	788	614	1088	910	178	259

Notes: i) V_{jh} req'd is calculated using a longitudinal beam bar stress of 1.25 times the specified yield stress.

ii) $V_{jv} = V_{jh} \cdot h_b / h_c$

iii) $V_{jh} = V_{jv} \cdot b_j \cdot h_c$

iv) V_{sh} and V_{sv} prov'd are calculated using the measured f_{yh}

Test Procedure

The beam-column units were subjected to several slow load reversals simulating very severe earthquake loading. The first loading cycle was in the elastic range, and this was followed by a series of deflection controlled cycles in the inelastic range comprising two full cycles to each of the displacement ductility factors of 2, 4, 6 and sometimes higher, as illustrated in Fig. 8. The "first yield" displacement at the end of the beam was found by loading to 3/4 of the flexural strength of the beam, as calculated on the basis of the measured material strengths, and multiplying that deflection by 4/3. That deflection can be considered as the deflection at ultimate load taking into account cracking and only elastic behaviour.

Longitudinal strains in the beam steel were measured using a Demec (demountable mechanical) strain gauge with a 102 mm gauge length. The Demec points were attached to the ends of steel studs which had been welded to the longitudinal steel and which projected sideways through holes in the cover concrete. Strains on the transverse steel hoops in the joint core were measured using electrical resistance strain gauges which were positioned on the steel in the direction of the horizontal shear so that any bending of the hoop bar due to the tendency of the concrete to bulge outwards

would not alter the strain reading. Curvatures of the beam in the potential plastic hinge regions were measured using dial gauges attached to steel holding frames which in turn were attached to horizontal steel bars which passed through the concrete core just inside the longitudinal steel. The shear distortion of the joint core was found from dial gauge readings made in the direction of the joint core diagonals. The dial gauges for the shear distortion readings were attached to the ends of horizontal steel bars which passed through the joint core just inside the intersecting beam and column longitudinal bars. Deflections of the units were measured using dial gauges.

The column ends were grouted into steel caps and the column loads were applied through steel pins which allowed free rotation during the testing. The beam loads were also applied through steel pins which allowed free rotation at the load points.

General Behaviour of Test Units and Definition of "Adequate Ductility"

Figs. 9, 10, 11, 12 and 13 show for the four test units the measured beam end load versus beam end deflection curves, the measured strains in the bottom bars of the beams, the measured strains in the joint core hoops, and photographs illustrating damage during testing. The percentage of the measured overall deflection caused

Fig. 6 Reinforcing Details for Unit 3

Note :- Column as for Unit 3
Beam transverse steel as for Unit 2

Fig. 7 Reinforcing Details for Unit 4

Fig. 8 Imposed Displacement Ductility Factors During Loading Runs

by shear deformation of the joint core, and the percentage of the horizontal shear in the joint core resisted by the hoops, are shown in Table 4. The contribution to the overall deflection from joint core deformation was calculated from the diagonal displacements measured on the joint core. The horizontal shear carried by the joint core hoops was calculated from the strains measured on those hoops, and the imposed horizontal joint core shear was calculated from the internal forces required to achieve the measured beam end loads.

It should be noted that the commentary on the New Zealand loadings code (10) gives an approximate criterion for "adequate ductility" to be met in the case of reasonably regular symmetrical frames without sudden changes in storey stiffness. The approximate criterion is that the building as a whole should be capable of deflecting laterally through at least eight load reversals so that the total horizontal deflection at the top can reach at least four times that at first yield without the horizontal load carrying capacity being reduced by more than 20%. The horizontal deflection at the top at first yield can be taken as that at the design seismic load calculated on the assumption of elastic behaviour. The detailing procedures of the concrete design code (1) are meant to assure that ductile structures are capable of meeting this criterion.

It is evident that the eight load reversals (that is, four load cycles) to a displacement ductility factor μ of 4 amounts to a cumulative μ of $4 \times 2 \times 4 = 32$. For the purpose of assessing the results of the tests the criterion will be taken as requiring that the strength of the units should not decrease to less than 80% of the theoretical strength of the units during the two load cycles to $\mu = 2$, the two load cycles to $\mu = 4$ and the one load cycle to $\mu = 6$, which is a cumulative μ of 36. The theoretical strength of the units will be defined as that strength calculated using the measured (actual) f_y and f'_c values and a strength reduction factor ϕ of unity. The strength of units was governed by the flexural strength of the beams and the theoretical strengths so defined are shown as dashed lines in Figs. 9a, 10a, 11a and 12a.

Table 4 : Behaviour of Joint Cores of Test Units

Load Run No. ^a		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	
Displacement Ductility Factor, μ		3	3	2	-2	2	-2	4	-4	4	-4	6	-6	6	-6	8	-8	8	-8	8	-8	
Measured Percentage of Total Deflection of Beam Ends due to Joint Core Shear Deformation	Unit 1	9	1	2	7	10	9	6	11	5	8	10	35	32	36							
	Unit 2	18	21	15	15	18	16	12	12	17	20	32	66	59	73	64						
	Unit 3	11	18	8	15	16	23	8	13	8	14	5	8	10	16	12	18	23	30	35	43	
	Unit 4	9	6	9	6	10	8	7	7	10	10	11	21	35	51	70						
Measured Percentage of Horizontal Joint Core Shear Resisted by the Hoops, $V_{sh}/V_{jh}^{\%}$	Unit 1	17	38	35	58	57	65	67	75	80	76	93	73	100	100							
	Unit 2	27	21	28	22	26	21	27	27	32	32	34	37	40	41	44	70					
	Unit 3	1	3	10	17	22	21	25	31	29	36	31	33	33	29	31	35	34	35	35	37	
	Unit 4	19	26	26	30	26	34	27	36	31	37	35	38	39	45	48	60					
																						NZS 3101 $V_{sh}/V_{jh}^{\text{req'd}}$ percentage
																						100
																						37
																						37
																						38

^a As shown on the beam end load-deflection curves in Figs. 9a, 10a, 11a and 12a.

Unit 1 Results

Fig. 9a shows that the theoretical strength of the unit was approached but not exceeded during the test. Nevertheless, at the maximum deflection during the first cycle to $\mu = 6$, the load carried was 88% of the theoretical strength and hence the unit satisfied the NZS 4203 criterion for adequate ductility. The pinching of the load-deflection response was due to the change in stiffness caused by the closure during the loading runs of open cracks in the concrete "compression" zone and in the joint core. The rounding of the load-deflection response near peak load was due to the Bauschinger effect on the stress-strain curve of the longitudinal steel. Strain hardening of the longitudinal steel did occur, as is shown in Fig. 9b, and resulted in steel stresses which would have been at least 1.25 times the specified yield strength.

It was apparent that the shear strength of the joint core degraded during the loading cycles and eventually the joint core strength governed the strength of the unit. Table 4 summarizes the behaviour of the joint core in terms of components of deformation and shear resistance. The increase in the deflection component from joint core deformation, and the reduction in the joint core shear carried by the concrete diagonal strut mechanism, as the loading progressed, are both evident. Despite the provision of sufficient hoops in the joint core to resist the entire horizontal shear force, yielding eventually occurred in that transverse steel in the first load cycle to $\mu = 6$ (see Fig. 9c) and at the end of the test the whole of the joint shear was carried by the hoops as assumed in the design (see Table 4). Visible damage to the joint during testing is shown in Fig. 13.

It is of interest to note that although the horizontal shear stress in the joint core was comfortably within the maximum allowed by NZS 3101 ($V_{jh}/\sqrt{f'_c} = 1.23 < 1.5$) there was extreme congestion of joint core shear reinforcement. Hence it is obviously better to use larger size member sections to ease congestion of reinforcement in the joint core for this type of design with beam plastic hinging adjacent to the column faces.

(a) Vertical Beam End Load Versus Beam End Displacement

(b) Strains in Beam Longitudinal Bottom Bars

(c) Strains in Joint Core Hoops

Fig. 9 Test Results from Unit 1

Unit 2 Results

Fig. 10a shows that the theoretical strength of the unit was exceeded during the test. In the first cycle to $\mu = 6$, the beam loads attained peaks which were about 10% greater than the theoretical strengths, due to strain hardening of the steel raising the stress above the measured yield stress (see Fig. 10b). Hence the NZS 4203 criteria for adequate ductility was satisfied. Less pinching of the load-deflection response occurred in this unit, mainly because of the better control of shear cracking in the joint core.

The joint core retained its shear strength well, as is shown in Table 4. The shear resistance assigned to the concrete diagonal compression strut mechanism in the design, $0.63V_{jh}$, was indeed being carried during the first load cycle to $\mu = 6$. Yielding of the joint hoops occurred during that load cycle (see Fig. 10c).

The strain distributions along the longitudinal bars adjacent to the joint core shown in Fig. 10b are of interest. Initially yielding of these beam bars occurred only in the vicinity of the designed plastic hinge regions away from the column face. During the loading cycles to $\mu = 2$ strain hardening of steel commenced in those plastic hinge regions. This strain hardening increased with further load cycles and eventually the beam flexural capacity at the critical section 500 mm from the column face was greater than 1.16 times the value based on the measured yield strength and yield penetration had progressed along the beam to the column face, accompanied by a corresponding increase in beam curvature at the column face (see Fig. 10b). At the peak of the second cycle to $\mu = 4$ it was considered that the joint core was no longer in the elastic range. During the first cycle to $\mu = 6$ the beam bars at the column faces reached strains close to that associated with strain hardening. During the second cycle to $\mu = 6$ the joint core deformation had increased to the point where the plastic rotations in the beams were decreasing significantly, and the joint core strength was degrading and governing the strength of the unit. In the latter stages of testing some evidence of sliding shear deformation was noticed at the designed critical section in the beams, but this was not serious since at the theoretical strength of the unit the nominal shear stress at the beam section was only $0.12\sqrt{f_c}$ MPa. Visible damage to the joint core during testing is shown in Fig. 13.

Unit 3 Results

Fig. 11a shows that the theoretical strength of the unit was exceeded during the test. In the loading cycles to $\mu = 6$ the beam loads attained peaks which were 15 to 20% greater than the theoretical strengths, because of strain hardening of the steel. Hence the performance of the unit was significantly better than required by the NZS 4203 criterion for adequate ductility. (From this observation it could be considered that a

reduction in the quantities of joint core reinforcing could be made, but it should be noted that the joint core hoops were already less than the code minimum necessary for confining steel). The pinching of the load-deflection loops was only noticeable in the loading cycles to high ductility values.

There was not a significant loss of shear stiffness or strength of the joint core during testing, despite the yielding of the joint core hoops (see Fig. 11c) which commenced in the first cycle to $\mu = 4$ and the large inelastic joint core hoop strains that occurred in the subsequent load cycles. Table 4 shows that the deformation of the joint core accounted for not more than 16% of the unit deflection in the two load cycles to $\mu = 6$ and the percentage of joint core shear carried by hoops did not increase markedly during the cyclic loading. The shear resistance assigned to the concrete diagonal compression strut mechanism in the design, $0.63V_{jh}$, can be compared with measured value of $0.67V_{jh}$ carried by that mechanism during the first load cycle to $\mu = 6$. Thus yielding of hoops and full depth flexural cracking in the beams at the face of the column did not significantly reduce the diagonal compression strut mechanism for this exterior joint, whereas it caused a significant reduction in the case of the interior joint of unit 1 (see Table 4). The better performance of exterior joints compared with interior joints is recognised by NZS 3101 (1). It is considered to be due to the diagonal compression strut being able to form between the anchorage bend in the beam tension bars and the column ties placed close to but just outside the joint core, even when full depth cracking occurs in the beam, as postulated by Paulay and Scarpas (11).

The strain distributions measured along the beam longitudinal bars shown in Fig. 11b indicate that yield of beam steel penetrated well into the joint core. The code requirement that the anchorage of those bars be considered to commence at the mid-depth of the column was reasonable for this unit. No significant slip of beam bars was observed to occur. Damage to the unit visible during testing is illustrated in Fig. 13.

Unit 4 Results

Fig. 11a shows that the theoretical strength of the unit was exceeded during the test. During the first cycle to $\mu = 6$ the beam ends sustained peak loads which were about 18% greater than the theoretical strengths based on the measured yield strengths, because of strain hardening of the steel. Hence the performance of the unit was again significantly better than required by the NZS 4203 criterion for adequate ductility. The pinching of the load-deflection loops was limited.

The joint core retained its shear strength well, as is indicated in Table 4. The shear resistance assigned to the concrete diagonal compression strut mechanism in the design, $0.62V_{jh}$, was

(a) Vertical Beam End Load Versus Beam End Displacement

(b) Strains in Beam Longitudinal Bottom Bars

(c) Strains in Joint Core Hoops

Fig. 10 Test Results from Unit 2

(a) Vertical Beam End Load Versus Beam End Deflection

(b) Strains in Beam Longitudinal Bottom Bars

(c) Strains in Joint Core Hoops

Fig. 11 Test Results from Unit 3

(a) Vertical Beam End Load Versus Beam End Deflection

(b) Strains in Beam Longitudinal Bottom Bars

(c) Strains in Joint Core Hoops

Fig. 12 Test Results from Unit 4

indeed carried during the first load cycle to $\mu = 6$. Yielding of joint core hoops had commenced in the first cycle to $\mu = 4$ (see Fig. 12c) but this did not cause a marked decrease in the shear strength or stiffness of the joint core, as is shown in Table 4.

The strains measured along the longitudinal bars shown in Fig. 12b indicated that initially the beam underwent plastic rotations in the designed plastic hinge region, but in the later stages of the test yielding of longitudinal steel penetrated along the beam to the column face and into the joint core resulting in plastic rotation occurring over a greater region. No slip of beam bars was noticeable. This yield penetration occurred because strain hardening of the beam reinforcing at the design plastic hinge region 500 mm from the face raised the flexural capacity there sufficiently to cause yielding in the beam at the column face as well. Sliding shear deformation was noticeable at the designed critical section in the later stages of testing, but this was not serious since at the theoretical strength of the unit the nominal shear stress in the beam was only $0.15\sqrt{f'_c}$ MPa. Damage to the unit visible during testing is illustrated in Fig. 13.

Comparison of NZS 3101 and the Draft ASCE-ACI Committee 352 Design Recommendations for the Joint Cores of the Units

The measured percentage of the horizontal shear force in the joint core carried by the joint core hoops (V_{sh}/V_{jh}) in the first loading cycle to $\mu = 6$ shown in Table 1 compared very well with the values of 100, 37, 37 and 38% recommended by NZS 3101 for units 1, 2, 3 and 4, respectively, for the column axial load level of $0.1f'_c A_g$ applied in these tests. It is of interest also to recall the result from the previously tested unit S1 (9), which was an interior beam-column joint with a column axial load level of $0.24f'_c A_g$ and with plastic hinging occurring in the beams at the column faces. As Table 3 shows, unit S1 had only 68% of the horizontal joint core shear reinforcement required by NZS 3101. In that test the beams did not reach their theoretical flexural strength and shear failure occurred in the joint core which resulted in the strength of the unit falling to 61% of the theoretical strength based on beam moment capacity after two load cycles to $\mu = 2$, two load cycles to $\mu = 4$, and one load cycle to $\mu = 6$. Hence the NZS 4203 criterion for adequate ductility was not met by unit S1.

As discussed previously, the joint core shear requirements of NZS 3101 (1) differ markedly from the recommendations of the draft ASCE-ACI Committee 352 report (7). According to the draft ASCE-ACI approach the quantity of hoop steel required in the joint core for both confinement and shear is given by the confinement equations (Eqs. 17 and 18) and the horizontal shear should not exceed a limiting value (Eq. 16). According

to the NZS 3101 approach, the quantity of joint core hoop steel required is that necessary to carry the shear actually present (Eqs. 2 to 5) but should not be less than that required for confinement (Eqs. 10 and 11). The A_{sh}/s_h values calculated for the four units using these two approaches are shown in Table 5; also tabulated is the A_{sh}/s_h actually provided for each unit. Note that the quantity actually provided in unit 3 satisfied the NZS 3101 requirement for shear but was 76% of the NZS 3101 requirement for confinement. In all other cases NZS 3101 was satisfied.

It is of interest to note that according to the draft ASCE-ACI approach the horizontal shear force on the joint cores should not exceed the value given by Eq. 16, namely $V = 1.0 \times 0.67\sqrt{34} \times 305 \times 406N = 484$ kN for all units, whereas according to NZS 3101 the horizontal shear strength of the joint cores as reinforced was 1030, 997, 543 and 625 kN for units 1, 2, 3 and 4, respectively, assuming a strength reduction factor of unity for both approaches. Thus the four test units reinforced according to NZS 3101 were able to sustain much greater horizontal joint core shears than permitted by the draft ASCE-ACI recommendations. Note also that units 1 and 2 contained more hoops than required by the ASCE-ACI approach but units 3 and 4 contained less hoops than required by the ASCE-ACI approach.

These considerable differences between the two design approaches arise because in the draft ASCE-ACI method no consideration is given to the mechanisms of shear resistance in the joint core. Thus the draft ASCE-ACI approach may be conservative in some cases and unconservative in others, depending on the particular joint conditions.

Table 5 : Comparison of Quantity of Joint Core Hoops Required for Shear and Confinement by NZS 3101 and Draft ASCE-ACI Committee 352 Recommendations, and Quantity Actually Provided, A_{sh}/s_h mm²/mm.

Unit	1	2	3	4
Draft ASCE-ACI Requirements for Confinement and Shear	3.22	3.97	2.84	2.89
NZS 3101 Requirements: For Confinement For Shear	2.68 10.5	3.31 4.48	2.37 1.83	2.41 2.48
Actually Provided	11.0	4.76	1.80	2.79

- Notes: (i) All required A_{sh}/s_h values are calculated using the measured f_y and f'_c values.
(ii) Horizontal shear strength of the joints according to the draft ASCE-ACI requirement (Eq. 16) was 484 kN for all units.
Taking $\phi=1$ and $\gamma=0.67$

Unit 1 After Second Load Cycle
to $\mu = 4$

Unit 1 After First Load Cycle
to $\mu = 6$

Unit 2 After First Load Cycle
to $\mu = 4$

Unit 2 After First Load Cycle
to $\mu = 6$

Unit 3 After First Load Cycle
to $\mu = 4$

Unit 3 After First Load Cycle
to $\mu = 8$

Unit 4 After Second Load Cycle
to $\mu = 4$

Unit 4 After Second Load Cycle
to $\mu = 6$

Fig. 13 Damage Visible During Testing of Units

Notes cont'd...

- (iii) Horizontal shear strength according to the NZS 3101 requirements (Eqs. 2 to 5) for the joints as actually reinforced were 1030, 997, 543 and 625 kN for units 1, 2, 3 and 4, respectively.

CONCLUSIONS:

1. The recent draft recommendations for the design of reinforced concrete beam-column joints of ASCE-ACI Committee 352 show large differences from the approach used in NZS 3101. The NZS 3101 approach for joint core shear strength is based on a rational model which sums the shear carried by the concrete diagonal compression strut and the shear carried by truss action of the shear reinforcement. The draft ASCE-ACI approach assumes that providing the horizontal shear stress in the joint core does not exceed a limiting value the amount of transverse steel required for column confinement is satisfactory, and vertical shear is considered by the requirement of at least an eight bar column. In the opinion of the authors, the design of joint core hoop reinforcement on the basis of the quantity of transverse steel required to confine the ends of columns is illogical and cannot produce any degree of accuracy because it does not take into account the possible varying conditions for shear in joint cores. This is especially the case when the wide range of joint types and column axial loads used in design in practice is considered. Recognition of the different concrete diagonal compression strut mechanisms existing in interior and exterior joints also appears necessary.

2. The four reinforced concrete beam-column joint units 1, 2, 3 and 4 which had been designed according to the requirements of NZS 3101 were shown by tests under simulated seismic loading to satisfy the approximate criterion for adequate ductility of NZS 4203. It was apparent that the detailing requirements of NZS 3101 for joint core design were not overly conservative for these designs.

3. Unit 1 was a conventional interior beam-column joint with the critical plastic hinge sections in the beams designed to be located at the column faces. There was considerable congestion of hoop reinforcement in the joint core due to the large shear stresses in the joint core resulting from the high ratios of longitudinal reinforcement in the beams ($\rho = \rho' = 1.75\%$). This congestion could have been eased by using larger member cross sections. The relatively low axial column load of $0.1f'_c A_g$ meant that all the horizontal shear in the joint core needed to be allocated to the hoops.

4. Unit 2 was an interior beam-column joint with the critical plastic hinge sections in the beams designed to be located 500 mm away from the column faces. The beam sizes and strengths were the same as for unit 1. However, because the beam longitudinal steel was designed so as not to yield at the column faces, the

improved bond conditions meant that the diameter of longitudinal beam bars could be 25% greater than in unit 6. Also, because the joint core was considered to remain in the elastic range the concrete diagonal compression strut mechanism could be considered to carry significant shear and only 37% of the horizontal shear in the joint core needed to be allocated to the hoops.

5. Unit 3 was an exterior beam-column joint with the critical plastic hinge section in the beam designed to be located at the column face and with the beam bars anchored in a beam stub at the far face of the column. In exterior joints, even when plastic hinging occurs in the beam at the column face, the concrete diagonal compression strut mechanism can be preserved quite well during cyclic loading, evidently because a steeper diagonal strut can form between the bend in the beam tension steel at the far face of the column and the column ties at the near face just outside the joint core. As a result, only 37% of the joint core horizontal shear needed to be allocated to the hoops. The penetration of steel yield along the beam bars into the joint core demonstrated that requiring the anchorage to commence within the joint core as specified by NZS 3101 was reasonable. This anchorage requirement had meant that to provide sufficient anchorage length for the beam bars a stub was required at the far face of the column because of the relatively small column depth.

6. Unit 4 was an exterior beam-column joint with the critical plastic hinge section in the beam designed to be located 500 mm away from the column face. This design permitted anchorage of the beam bars within the column core, because the beam steel was designed not to yield at the column face, and therefore anchorage could be considered to commence at the column face of entry. Hence an anchorage stub was not needed. Because the joint core was designed to remain in the elastic range only 38% of the joint core shear needed to be allocated to the hoops.

7. In the case of interior beam-column joints the design of plastic hinge regions in beams to be located away from the column faces (that is, "relocated plastic hinges"), so that the joint core remains in the elastic range as in unit 2, was shown to allow much easier detailing of steel when member sizes are small and joint shear are high.

8. In the case of exterior beam-column joints the design of relocated plastic hinges appears to be only of advantage when beam bars cannot otherwise be anchored within in the column core because of small column size, and when beam stubs at the outside face of the column are not present because of architectural or space restrictions.

9. The use of an overstrength factor of 1.25 for Grade 275 reinforcement at relocated plastic hinges, when determining the longitudinal steel areas required in the beams at the column faces to suppress

yield there, should lead to satisfactory design. The overstrength factor used in the design of the interior beam-column joint unit 2 was 1.16 and for the exterior beam-column joint unit 4 was 1.20. In both of these units during the tests, strain hardening of the longitudinal reinforcing at the relocated plastic hinge raised the flexural capacity there sufficiently to cause yield of longitudinal steel to spread along the beam to the column face and to penetrate into the joint core, leading eventually to yield of the joint core hoops. Hence use of an overstrength factor of less than 1.25 for Grade 275 reinforcement would be inadvisable.

10. In general, the use of relocated plastic hinges, as employed in units 2 and 4 seems to be a practical design alternative to conventional design. Note however that if the ratio of gravity load to seismic load induced moment is high the moment gradient may not allow the use of such a design because only a short length of beam will have negative moment. Also, the use of relocated plastic hinges will impose a higher curvature ductility demand on those plastic hinge sections, because the smaller length of beam between the critical positive and negative moment sections will mean that greater plastic hinge rotations are required at these sections to achieve the required displacement ductility factor. This increased curvature ductility demand should not be of concern except for beams with short spans.

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NOTATION

(All units are mm and N)

- A_c = area of concrete core section measured to outside of peripheral hoop.
- A_g = gross area of section
- A_{jh} = total area of effective horizontal joint shear reinforcement
- A_{jv} = total area of effective vertical joint shear reinforcement
- A_s = total area of tension beam reinforcement
- A'_s = total area of compression beam reinforcement
- A_{sc} = area of tension reinforcement in one face of the column section
- A'_{sc} = area of compression reinforcement in one face of the column section
- A_{sh} = total effective area of hoop bars and supplementary cross ties in direction under consideration within spacing s_h

b_c	= overall width of column	V_u	= total horizontal shear force across joint
b_j	= effective width of joint	γ	= joint shear strength factor
b_w	= web width of column	ρ	= ratio of longitudinal tension reinforcement = A_s/bd where b and d are beam width and effective depth, respectively
C_j	= participation factor = $V_{jh}/(V_{jx} + V_{jz})$	ρ'	= ratio of longitudinal compression reinforcement = A'_s/bd where b and d are beam width and effective depth, respectively
d_b	= bar diameter	ϕ	= strength reduction factor
f'_c	= compressive cylinder strength of concrete	μ	= displacement ductility factor = ratio of maximum displacement to displacement at first yield
f_y	= yield strength of longitudinal reinforcement		
f_{yh}	= yield strength of transverse reinforcement		
f_u	= ultimate strength of reinforcement		
h''	= dimension of concrete core of section measured perpendicular to the direction of the hoop bars to outside of peripheral hoop		
h_b	= overall depth of beam		
h_c	= overall depth of column in the direction of the horizontal shear to be considered		
M_u	= design moment due to factored gravity and earthquake loads		
P_e	= design axial load due to factored gravity and earthquake loads		
ℓ_{db}	= basic development length for deformed bar in compression		
ℓ_{dh}	= development length for a deformed bar in tension terminating with standard 90° hook according to ASCE-ACI Committee 352		
ℓ_{hb}	= basic development length for deformed bar in tension terminating with standard 90° hook		
s_h	= centre to centre spacing of hoop sets		
v_{jh}	= $V_{jh}/b_j h_c$		
V_{ch}	= horizontal joint shear strength provided by diagonal concrete strut		
V_{cv}	= vertical joint shear strength provided by diagonal concrete strut		
V_{jh}	= total horizontal shear force across the joint		
V_{jv}	= total vertical shear force across the joint		
V_{jx}	= total horizontal joint shear force in the x-direction		
V_{jz}	= total horizontal joint shear force in the z-direction		
V_{sh}	= horizontal joint shear strength provided by horizontal joint shear reinforcement		
V_{sv}	= vertical shear strength provided by vertical joint shear reinforcement		