7th Canadian Conference on Earthquake Engineering / Montreal / 1995
7ième Conférence canadienne sur le génie paraséismique / Montréal / 1995

Recent Structural Concrete Research and Seismic Design Developments in New Zealand

Robert Park[1]

ABSTRACT

The results of a range of simulated seismic load tests and associated analysis, involving reinforced concrete elements and subassemblages, conducted at the University of Canterbury in recent years are briefly outlined. Much of this research has led to the changes in the revised New Zealand concrete design standard NZS 3101 published in 1995. These investigations have involved reinforced concrete columns to investigate the transverse reinforcement required for adequate ductility and shear strength, reinforced concrete beam-column joints, to determine shear and bond requirements, high strength concrete and steel reinforcement to investigate their use, and precast concrete to investigate means of connecting frame elements together and of providing adequate support of precast floors. Reinforced concrete columns and beam-column joints with reinforcing details typical of old (now sub-standard) seismic codes have also been tested to investigate their performance and to determine methods to retrofit them so as to improve their strength and ductility.

INTRODUCTION

The New Zealand concrete design standard NZS 3101:1982 (Standards Association of New Zealand, 1982) has been revised and the new edition NZS 3101:1995 (Standards New Zealand, 1995) has been published. Several changes to the seismic design provisions for moment resisting frames have been made especially to the sections on columns and beam-column joints.

A range of simulated seismic load tests and associated analyses have been conducted at the University of Canterbury, New Zealand in recent years to provide information for the revision of NZS 3101. This research has involved mainly columns and beam-column joints, investigating basic behaviour, the use of high strength materials and the performance of old (now sub-standard) reinforcement details. Also, connections between and support of precast concrete elements has been investigated. In addition, the retrofit of older reinforced concrete frames by concrete jacketing has been studied. The experimental procedure used involving quasi-static cyclic loading to predetermined ductility factors is described elsewhere (Park, 1989).

This paper will outline the main results of the experimental testing and associated analyses. Also, the main changes in the 1995 New Zealand concrete design standard will be summarised.

---

[1]Professor of Civil Engineering, University of Canterbury, Private Bag 4800, Christchurch, New Zealand

REINFORCED CONCRETE COLUMNS OF BUILDING FRAMES AND BRIDGE PIERS

Background

Damage caused by earthquakes through the years, including the Hyogo-ken Nanbu earthquake in Kobe in January 1995, has indicated that inadequately designed reinforced concrete columns can be very vulnerable elements of buildings and bridges during major earthquakes. This has been due mainly to the lack of capacity design procedures which has resulted in columns with inadequate flexural strength to avoid plastic hinging, and to insufficient transverse reinforcement to avoid brittle behaviour. The previous New Zealand concrete design code NZS 3101:1982 (Standards Association of New Zealand, 1982) had design provisions for ductility of reinforced concrete columns which were derived from analytical and experimental studies conducted in New Zealand mainly during the 1970s. In more recent years considerable additional experimental and analytical research has been conducted to further improve the design provisions for ductility, which have been induced in the new edition of NZS 3101:1995 (Standards New Zealand, 1995). That research is described below.

It is to be noted that the rules for protecting columns of moment resisting frames, by ensuring that as far as possible strong column-weak beam behaviour of tall ductile frames occurs, that were introduced in NZS 3101:1982 have remained practically the same in NZS 3101:1995. Those rules involve multiplying the column bending moments determined from elastic frame analysis, for the load cases involving the seismic design forces, by a factor which takes into account the beam flexural overstrength, higher mode effects and concurrent seismic forces. The multiplier depends on the frame variables and is at least 1.81 for ductile frames (Park, 1992a).

Equations for Quantities of Transverse Confining Reinforcement

Recent research at the University of Canterbury has resulted in refined design equations for the quantities of transverse reinforcement required in reinforced concrete columns to achieve specified levels of curvature ductility factor (Watson et al, 1994). These refined equations are an improvement on those recommended in NZS 3101:1982. The derivation was based on stress-strain relationships for concrete with compressive cylinder strengths up to about 40 MPa, confined by various quantities and arrangements of transverse reinforcement, previously obtained from experimental tests and analysis (Mander, et al 1988a and b). The stress-strain relations were used in cyclic moment-curvature analyses of a range of reinforced concrete columns to derive design charts for the available curvature ductility factor ϕ_u/ϕ_y of reinforced concrete columns (Zahn, et al, 1986).

The yield curvature ϕ_y was defined as shown in Fig. 1, being the curvature when yield is first reached in the longitudinal tension reinforcement or when the extreme fibre concrete compressive strain reaches 0.002, whichever occurs first. The ultimate curvature ϕ_u was defined as the curvature when, after four cycles of imposed bending moment to that curvature in each direction, either the moment of resistance has reduced by 20%, or the tensile strain in the transverse or longitudinal reinforcement has reached a limiting value, or the compression strain in the longitudinal reinforcement has attained a value where significant buckling occurs, whichever is least.

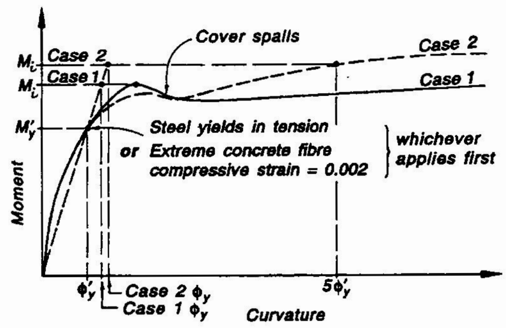


Fig. 1 Definition of the yield curvature ϕ_y

The refined design equations were derived for typical values of the axial load ratio, the concrete compressive strength, the mechanical reinforcing ratio, the ratio of thickness of concrete cover to column depth, and the required curvature ductility factor ϕ_u/ϕ_y. The 95 percentile values for the required transverse reinforcement were obtained and regression analysis was used to obtain the best-fit equations by the least squares method. The derived equations were as follows:

For columns with rectangular hoops and cross ties:

$$A_{sh} = \frac{\left(\frac{\phi_u}{\phi_y} - 33p_t m + 22\right) s_h h''}{111} \frac{A_g}{A_c} \frac{f_c'}{f_{yt}} \frac{N}{\phi f_c' A_g} - 0.006 s_h h'' \qquad (1)$$

For columns with circular hoops or spirals:

$$p_s = \frac{\left(\frac{\phi_u}{\phi_y} - 33p_t m + 22\right)}{79} \frac{A_g}{A_c} \frac{f_c'}{f_{yt}} \frac{N}{\phi f_c' A_g} - 0.008 \qquad (2)$$

where A_{sh} = area of transverse bars in direction under consideration within centre to centre spacing of hoop sets s_h, h'' = dimension of core of column at right angles to direction to transverse bars under consideration, A_g = gross area of column, A_c = core area of column, ϕ_u/ϕ_y = curvature ductility factor, $p_t = A_{st}/A_g$, A_{st} = total area of longitudinal column reinforcement, $m = f_y/0.85f_c'$, f_y = yield strength of longitudinal steel, f_{yt} = yield strength of transverse steel, f_c' = concrete compressive cylinder strength, N = axial compressive load on column, ϕ = strength reduction factor and p_s = ratio of volume of transverse circular hoop or spiral steel to volume of concrete core of column.

Eqs 1 and 2 have been adopted in NZS 3101:1995 for the potential plastic hinge regions of columns of ductile moment resisting frames in which the design seismic forces are determined using a displacement ductility factor $\mu = 6$ assuming: $\phi_u/\phi_y = 10$ for the potential plastic hinge regions of columns protected by capacity design, and $\phi_u/\phi_y = 20$ for the potential plastic hinge regions of the bottom storey columns or in columns of one or two storey frames where strong beam-weak column design is permitted, where $\mu = \Delta/\Delta_y$ where Δ = maximum lateral displacement and Δ_y = lateral displacement at first yield.

The design axial compressive load on columns is not permitted to exceed $0.7P_o$, where P_o is the concentric load strength of the column.

The refined design equations, Eqs 1 and 2, give only the transverse reinforcement required for concrete confinement. The transverse reinforcement provided must also be checked to ensure that the tie requirements for preventing premature buckling of longitudinal compression bars, and the requirements for shear reinforcement, are satisfied. This check may lead to more transverse reinforcement being required to prevent bar buckling and/or to prevent shear failure than required for concrete confinement.

Within the potential plastic hinge regions of the columns of ductile frames the vertical spacing of transverse reinforcement is not permitted to exceed the smaller of 6 longitudinal bar diameters or one-quarter of the least lateral dimension of the column section, and the horizontal spacing of transverse reinforcement in rectangular columns is not permitted to exceed the larger of 200 mm or one-quarter of the adjacent lateral dimension of the column section.

An example of the quantities of transverse reinforcement required by NZS 3101:1995 in the potential plastic hinge regions of columns when $\phi_u/\phi_y = 20$ is required is shown in Fig. 2. Note that the requirement for concrete confinement (Eq. 1) governs at higher axial loads, and the requirement for preventing premature buckling of longitudinal reinforcement governs at lower axial loads. A comparison with the requirement of the NZS 3101:1982 and with the current requirements of the ACI building code (American Concrete Institute, 1989) is also shown.

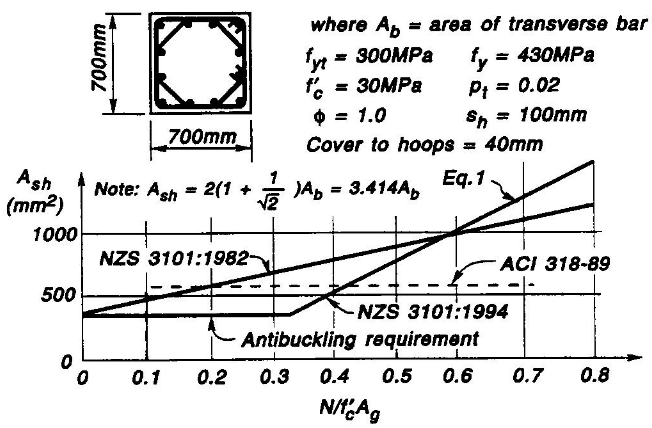


Fig. 2 Example of transverse reinforcement required for a ductile column

Simulated seismic load tests were conducted on eleven reinforced concrete columns to check the above analytical approach for determining the available ductility of a range of columns (Watson and Park, 1994). The columns had either 400 mm square or octagonal cross section (see Fig. 3) and were tested under constant axial compression, with N/f'_cA_g ranging between 0.1 and 0.7, and reversible quasi-static lateral loading applied to a stub. The tests confirmed that the approach of NZS 3101:1995 provides a rational basis on which to determine the quantities of transverse reinforcement required for confinement.

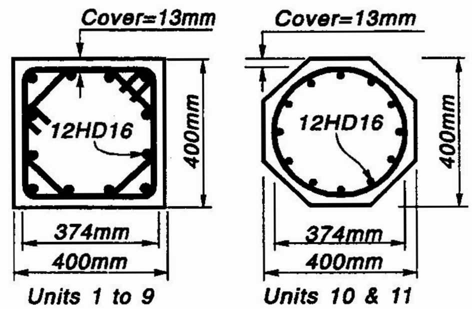


Fig. 3 Cross sections of columns tested (Watson and Park, 1994)

<u>**Length of End Region of Column to be Confined**</u>

The confined length of column adjacent to the section of maximum bending moment (see Fig. 4) needs to be sufficiently long to extend over the region of major plastic curvature and to ensure that the higher flexural strength of the column in the confined region does not lead to flexural failure of the column in the adjacent less confined region. The second requirement is particularly important for normal strength concrete columns with high axial compression, since for such columns the flexural strength is markedly increased by confinement of the concrete (Priestley and Park, 1987; Watson and Park, 1994).

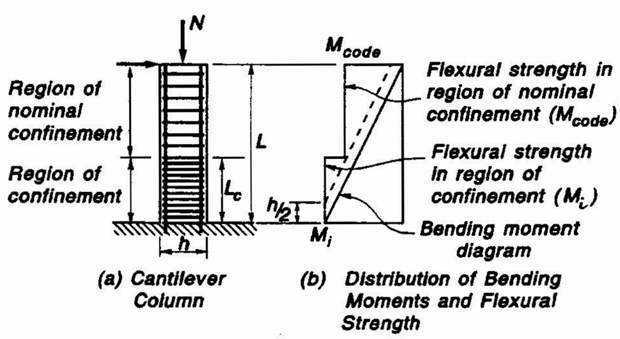


Fig. 4 Determination of the length of the confined end region of column

Fig. 4 shows the distribution of bending moments for a cantilever column due to an imposed lateral load at the top, and the flexural strengths of the confined and nominally confined regions of the column. To compensate for the effects of the spread of yielding due to possible diagonal tension cracking, the moment diagram is spread by h/2 along the member, where h = column depth. The length of the region that needs to be confined L_c can be estimated knowing the enhanced flexural strength M_i in the confined region and the conventionally calculated flexural strength M_{code} outside the confined region.

An analysis by Watson and Park, 1994, of the test results from the columns subjected to simulated seismic loading at the University of Canterbury since the late 1970s, resulted in the following equation:

$$\frac{L_c}{h} = 1 + 2.8 \frac{N}{\phi f_c' A_g} \tag{3}$$

where the notation is the same as for Eqs 1 and 2. It is evident that the confined length L_c should indeed be increased with the axial load level. The requirement of NZS 3101:1995 is based on Eq. 3. The confined length L_c for low axial load levels when $N < 0.25\phi f_c' A_g$ is taken to be the greater of h or where the moment exceeds 0.8 of the adjacent end moment, and L_c for high axial load levels with $N > 0.5\ \phi f_c' A_g$ is taken to be the greater of 3h or where the moment exceeds 0.6 of the adjacent end moment. An intermediate value of L_c is taken for axial load levels in between.

Shear Strength of Columns

Tests have recently been conducted by Li X R et al, 1991 and 1994, on six reinforced concrete columns with uniaxial cyclic lateral loading and variable axial force. For three of the columns the strength was controlled by flexure during the loading cycles and the variation in axial load (from compression to tension) had a significant effect on the flexural strength and stiffness. This variation in stiffness should be taken into account in nonlinear dynamic analysis.

For the other three columns the column height to depth ratio was smaller and the effect of shear was more dominant. The transverse reinforcement in the columns was designed using the Standards Association of New Zealand, 1982 equations for shear outside plastic hinge regions (that is, assuming no degradation in shear strength due to seismic loading). In the tests the applied axial load varied linearly in proportion to the bending moment at the critical section of the column. The shear force resisted by the concrete mechanism V_c was found by subtracting the shear force resisted by the transverse reinforcement calculated assuming a 45° truss mechanism from the applied shear force. The degradation of shear strength was not worsened by the effects of the varying compression-tension axial load, although the presence of tensile axial load encouraged the opening of cracks and delayed the closure of the diagonal tension cracks when the compressive axial load was present.

As an interim recommendation, in plastic hinge regions for displacement ductility factors μ of up 2, V_c could be taken as the value for outside plastic hinge regions. For greater μ values V_c could be taken to reduce linearly to a minimum $0.1\sqrt{f_c'}bd$ MPa at $\mu = 4$, and to remain at 0.1

$\sqrt{f'_c}bd$ MPa for higher values of μ, where b = width of member and d = effective depth of tension reinforcement of member. NZS 3101:1995 takes a more conservative approach, assuming $V_c = 0$ for axial load ratios N/f'_cA_g less than 0.1, due to the difficulty of ensuring that the actual minimum axial load ratio of frames during major earthquakes has been calculated by the designer. Further research is required into this important aspect of seismic design.

BEAM-COLUMN JOINTS

Design Shear Forces

For the purposes of evaluating the forces within an interior joint (see Fig. 5a and b) when plastic hinges form in the beams at the column faces, the stresses in the longitudinal beam tension reinforcement are taken to be at overstrength αf_y, where α = overstrength factor = 1.25 and f_y = lower characteristic yield strength of the longitudinal steel. Then the design horizontal joint shear force for conventionally reinforced members is:

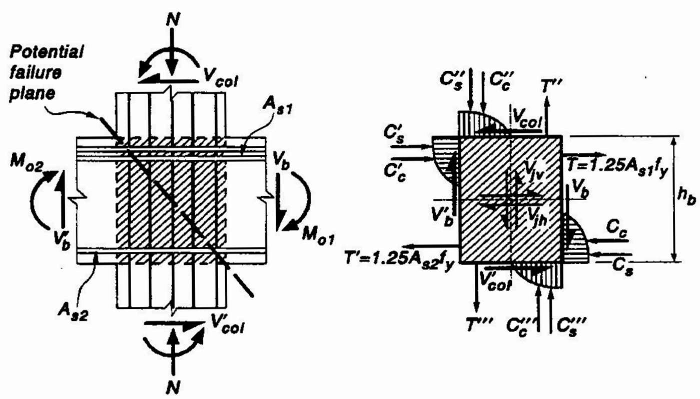
$$V_{jh} = 1.25 f_y (A_{s1} + A_{s2}) - V_{col} \qquad (4)$$

where A_{s1} and A_{s2} are the areas of longitudinal reinforcement in the top and bottom of the beam, respectively, and V_{col} is the shear force in the column when the beams are at overstrength.

The design vertical joint shear force may be approximated as

$$V_{jv} = V_{jh} h_b / h_c \qquad (5)$$

where h_b = depth of beam and h_c = depth of column.



(a) External Actions (b) Interior Actions

Fig. 5 Interior beam-column joint subjected to seismic actions

NZS 3101:1995 (Standards New Zealand, 1995) requires that the nominal horizontal shear stress in the joint v_{jh} shall not exceed $0.2f_c'$ MPa, where f_c' = concrete compressive cylinder strength and

$$v_{jh} = V_{jh}/(b_j h_c) \tag{6}$$

where h_c = depth of column and b_j = effective width of the joint. When $b_c > b_w$ either $b_j = b_c$ or $b_j = b_w + 0.5h_c$, whichever is the smaller, and when $b_c < b_w$ either $b_j = b_w$ or $b_j = b_c + 0.5h_c$, whichever is the smaller, where b_c = width of column and b_w = width of beam web.

Mechanisms of Shear Resistance

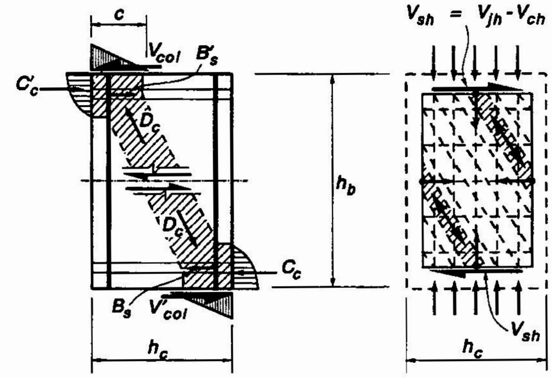
According to both NZS 3101:1982 and NZS 3101:1995 the shear strength of a beam-column joint core is due to the two mechanisms shown in Fig. 6a and b. The superposition of the two mechanisms to resist the total horizontal and vertical joint shear forces is then given by

$$V_{jh} = V_{ch} + V_{sh} \tag{7}$$

$$V_{jv} = V_{cv} + V_{sv} \tag{8}$$

where V_{ch} and V_{cv} are the horizontal and vertical shear forces transferred across the joint core by the diagonal compression strut mechanism, respectively, and V_{sh} and V_{sv} are the horizontal and vertical shear forces transferred across the joint core by the truss mechanism, respectively. NZS 3101:1982 assumed that the diagonal compression strut mechanism (Fig. 6a) transferred only the concrete compression forces acting on the joint core. This diagonal strut can be developed without shear reinforcement. The truss mechanism (Fig. 6b) was considered to transfer all the bond forces from the horizontal and vertical longitudinal reinforcement to the joint core. It is associated with a concrete diagonal compression field which requires for equilibrium the introduction of normal forces at the boundaries of the joint core by means of both horizontal and vertical joint (shear) reinforcement. NZS 3101:1982 assumed that when plastic hinges develop in the beams adjacent to the columns, full depth flexural cracks may occur in the beams at the column faces. As a result the concrete compression forces in the beams would gradually diminish and result in a corresponding increase in the compressive forces in the beam compression reinforcement, resulting in a significant reduction or even a total loss in the shear carried by the diagonal compression strut mechanism.

Recent research (Park and Dai, 1988; Cheung et al 1991, Restrepo et al 1992, Paulay and Priestley, 1992) has shown that the amount of horizontal joint reinforcement specified by NZS 3101:1982 can be somewhat reduced. These New Zealand studies have indicated that the relative contributions of the two shear resisting mechanisms in joints, shown in Fig. 6a and b, are strongly influenced by the distribution of bond forces along the longitudinal bars anchored within the joint core. The bond forces will require a major part of the joint shear force to be transmitted by the truss mechanism, but some deterioration of bond, and a consequent reduction in the ability of beam bars to resist compression forces, will result in an increased contribution to the shear resistance of the diagonal compression strut mechanism. Because of bond deterioration in the joint core the compressive stress in the top reinforcement is unlikely to exceed $0.7f_y$, and also it cannot exceed $1.25\beta f_y$ where β is the ratio of the area of the bottom to top beam reinforcement.



(a) Diagonal Compression Strut Mechanism

(b) Truss Mechanism

Fig. 6 Mechanisms of shear resistance of interior beam-column joint

Hence it can be postulated that the diagonal compression strut mechanism can in fact resist the concrete compression force C_c' plus the part of the total bond force introduced over the flexural compression zone of the column plus the column shear force V_{col}. The remainder of the total bond force may be assigned to the truss mechanism shown in Fig. 6b. Only the truss mechanism requires joint shear reinforcement. By taking into account the beneficial effect of axial compression it may be shown (Cheung et al, 1991) that the horizontal joint shear force V_{ch} resisted by the diagonal compression strut mechanism is at least 30% of the total horizontal joint shear force V_{jh}.

<u>**Design Equations for Shear Reinforcement of NZS 3101:1995**</u>

The design provisions for shear reinforcement in beam-column joints of NZS 3101:1995 are based on the above considerations. Equations are given for the areas of horizontal and vertical shear reinforcement required, obtained from

$$A_{jh} = V_{sh}/f_{yh} = (V_{jh} - V_{ch})/f_{yh} \tag{9}$$

and $$A_{jv} = V_{jv}/f_{yv} = (V_{jv} - V_{cv})/f_{yv} \tag{10}$$

where f_{yh} and f_{yv} are the yield strengths of the horizontal and vertical shear reinforcement, respectively.

(a) **For Horizontal Joint Shear** - The area of total effective horizontal joint shear reinforcement corresponding to each direction of horizontal joint shear force shall be:

For interior joints

$$A_{jh} = \frac{6v_{jh}}{f_c'}\left(1.4 - 1.6\,\frac{C_j N}{f_c' A_g}\right)\frac{f_y}{f_{yh}}\,A_s^* \tag{11}$$

where v_{jh} = nominal horizontal shear stress in joint core, f_c' = compressive cylinder strength of concrete, N = axial compressive load on column, $C_j = V_{jh}/(V_{jx} + V_{jx})$, V_{jx} = total horizontal joint shear in x direction, V_{jz} = total horizontal joint shear in z direction, A_g = gross area of column, f_y = yield strength of longitudinal reinforcement, f_{yh} = yield strength of horizontal joint reinforcement (f_y and f_{yh} are not to exceed 500 MPa), and A_s^* = greater of area of top or bottom beam reinforcement passing through the joint excluding the area of bars in effective tension flanges.

For exterior joints

$$A_{jh} = \frac{6v_{jh}}{f_c'}\,\beta\left(0.7 - \frac{C_j N}{f_c' A_g}\right)\frac{f_y}{f_{yh}}\,A_s \tag{12}$$

where N is taken negative with axial tension in which case C_j = 0 must be assumed, β = maximum ratio of the area of compression beam reinforcement to the area of the tension beam reinforcement, not to be taken greater than unity, and A_s = area of tension beam reinforcement including the area of bars in effective tension flanges where applicable.

Where plastic hinges in beams cannot develop at the face of columns, the yield stress f_y in Eqs 11 and 12 may be replaced by 0.8 times the computed tensile stress, f_s.

The area A_{jh} to be provided in accordance with Eqs 11 and 12 shall not be less than $0.4v_{jh}/f_c'$, and the ratio $6v_{jh}/f_c'$ in Eqs 11 and 13 shall not be taken to be less than 0.85 nor more than 1.2.

The quantity of horizontal joint reinforcement, placed as required above, shall be not less than that required in the end regions of columns immediately above and below a joint.

(b) **For Vertical Joint Shear** - The total area of effective vertical joint shear reinforcement, with columns that are expected to remain essentially elastic, corresponding to each of the two directions of joint actions, shall be:

$$A_{jv} = \left(\frac{0.7}{1 + (N/f_c' A_g)}\right)\frac{h_b}{h_c}\,A_{jh}\,\frac{f_{yh}}{f_{yv}} \tag{13}$$

where h_b = depth of beam and h_c = depth of column.

The vertical joint shear reinforcement shall normally consist of intermediate longitudinal column bars placed in the plane of bending between corner bars. The total area of effective vertical joint shear reinforcement shall be placed within the effective joint width, b_j.

Anchorage of Longitudinal Reinforcement in Interior Beam-Column Joints

In NZS 3101:1982 in order to provide adequate anchorage lengths for longitudinal reinforcement passing through interior beam-column joint cores when plastic hinges form in the beams at the column faces, it was specified that the diameter d_b of the longitudinal beam bars which pass through a column of depth h_c should satisfy

$$d_b/h_c \leq 12/f_y \tag{14}$$

where f_y is in MPa. This limitation on bar diameter has caused problems in design, because of the resulting small diameters of beam bars, unless the column depths are relatively large. The use of high strength concrete leading to smaller column sizes would mean that even smaller bar diameters are needed according to Eq. 14 in order to achieve anchorage of longitudinal reinforcing bars over smaller lengths.

The NZS 3101:1982 requirement (Eq. 14) was based on experimental studies conducted prior to 1980 in which concrete with compressive cylinder strength f_c' as low as 20 MPa was specified. More recent studies (Park and Dai, 1988; Cheung et al 1991; Paulay and Priestley, 1992) indicate that a number of factors in additional to bar yield strength should be taken into account, namely: some bond deterioration should be acceptable, compression yield of beam bars will not occur, the bond strength will increase when f_c' increases and/or when the beam bar is subjected to transverse compression from the column load, and bond strength will decrease in two-way frames due to transverse tensile strains and/or for top bars due to sedimentation of concrete.

Design recommendations for the anchorage of beam bars in interior beam-column joints may be derived as follows. Consider a longitudinal beam bar of diameter d_b passing through a column of depth h_c as shown in Fig. 7. The bar is considered to be yielding in tension at stress $1.25f_y$ at one column face and has compressive stress γf_y at the opposite column face, where $\gamma \leq 1$. The average ultimate bond stress u is assumed to be $k\sqrt{f_c'}$. For equilibrium (see Fig. 7):

$$\frac{\pi}{4} d_b^2 \left(1.25f_y + \gamma f_y\right) = \pi d_b h_c k \sqrt{f_c'} \tag{15}$$

$$\therefore \quad \frac{d_b}{h_c} = \frac{4k\sqrt{f_c'}}{\left(1.25f_y + \gamma f_y\right)} \tag{16}$$

Now when f_c' is as low as 20 MPa and $\gamma = 1$, NZS 3101:1982 permitted $d_b/h_c = 12/f_y$ (Eq. 17), which when used to calibrate Eq. 16 gives $k = 1.51$.

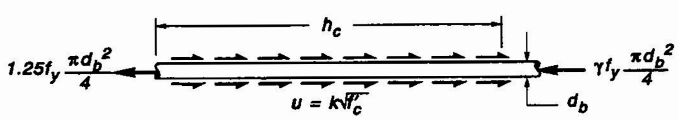


Fig. 7 Longitudinal beam bar passing through a column at an interior beam-column joint when plastic hinging occurs in the beams at the column faces

Design Equations of NZS 3101:1995

Based on the above considerations, the requirements of NZS 3101:1995 for the anchorage of longitudinal bars passing through interior beam-column joints are:

(a) **Longitudinal Beam Bars** - The diameter of longitudinal beam bars passing through interior joints shall be computed from either Eqs 17 or 18 below:

The ratio of longitudinal beam bar diameter to column depth shall be:

$$\frac{d_b}{h_c} \leq 3.3\,\alpha_f \frac{\sqrt{f_c'}}{\alpha_o f_y} \tag{17}$$

where α_f = 0.85 when beam bars pass through a joint in two directions as in two way frames or 1.0 when bars pass only in one direction, α_o = 1.25 when plastic hinges in beams are developed at column faces or 1.0 when by relocation of plastic hinges in beams the sections at the column faces remain in the elastic range.

Alternatively, by considering additional parameters, the ratio of longitudinal beam diameter to column depth may be determined from:

$$\frac{d_b}{h_c} \leq 6\left(\frac{\alpha_t \alpha_p}{\alpha_s}\right)\alpha_f \frac{\sqrt{f_c'}}{\alpha_o f_y} \tag{18}$$

where α_f and α_o are for Eq. 17, α_t = 0.85 for top beam bars (more than 300 mm of concrete cast underneath bars) or 1.0 for bottom beam bars, and

$$\alpha_p = [N/(2f_c' A_g)] + 0.95 \tag{19}$$

with the limitation of $1.0 \leq \alpha_p \leq 1.25$, where N is the minimum axial compression load on the column consistent with the governing ultimate limit state load combination,

$$\alpha_s = 2.55 - (A_s'/A_s) \tag{20}$$

for the bars of area A_s' with the limitation of $0.75 \leq A_s'/A_s \leq 1.0$, or
= 1.55 for the bars of area A_s

(b) **Longitudinal Column Bars** - When columns are designed to develop plastic hinges in the end regions, the maximum diameter of column bars passing through the beams shall satisfy

$$\frac{d_b}{h_b} \leq 3.2 \frac{\sqrt{f_c'}}{f_y} \tag{21}$$

When columns are not intended to develop plastic hinges in the end regions, the maximum diameter of longitudinal column bars at any level may exceed that given by Eq. 21 by 25%. This requirement need not be met if it is shown that the stresses in the column bars during an earthquake remain in tension or compression over the whole bar length contained within the joint.

It is evident that Eqs 17 to 21 will allow larger d_b/h_c ratios for higher concrete strengths. In ductile frames the specified f_c' is not permitted to exceed 70 MPa, according to NZS 3101:1995.

HIGH STRENGTH CONCRETE AND REINFORCING STEEL

Background

For reinforced concrete structures designed for earthquake resistance, NZS 3101:1982 required the use of reinforcing steel with characteristic yield strength not greater than 500 MPa and concrete with a specified compressive cylinder strength not greater than 55 MPa. However, in New Zealand, as in most countries, concrete with compressive cylinder strengths up to about 80 MPa can now be made with locally obtainable materials. The addition of silica fume permits compressive cylinder strengths of 100 MPa or higher to be attained. Also, very high strength steel reinforcement is available in many countries. For example, reinforcing steel with a yield strength up to about 1,300 MPa is now available in Japan.

Aspects of the use of high strength concrete and steel that have been investigated by experimental and analytical studies at the University of Canterbury are the parameters for the compressive stress block to be used in flexural strength calculations and the quantities of transverse reinforcement required for confinement, to take account of the more sharp falling branch of the stress-strain curve of unconfined high strength concrete.

Tests on High Strength Concrete Columns

Five reinforced concrete columns, constructed from concrete with compressive cylinder strengths of 93 or 98 MPa, have been tested subjected to quasi-static cyclic lateral loading (Li B et al, 1994). The column sections are shown in Fig. 8. The transverse reinforcement was either New Zealand manufactured Grade 430 steel or Japanese manufactured Ulbon steel which had a yield strength of f_{yh} = 1,317 MPa at the 0.2% offset strain. The quantity of transverse reinforcement in the columns was approximately that required by NZS 3101:1982, calculated using the actual measured steel and concrete strengths.

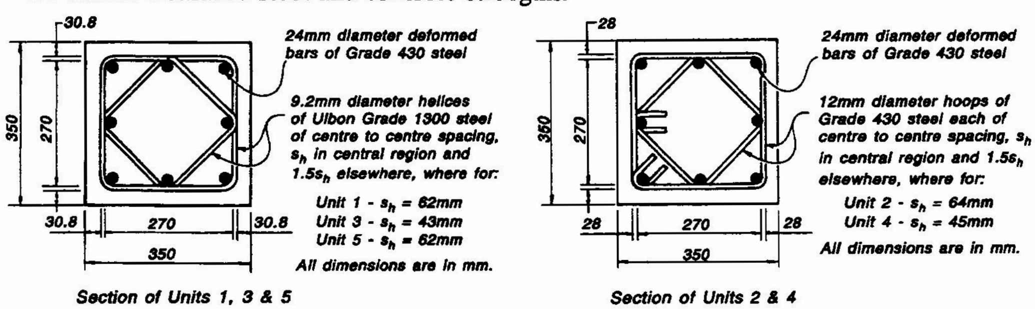


Fig. 8 Cross Sections of High Strength Concrete Columns (Li B et al, 1994)

The columns were tested under a constant axial compressive load of either 0.3 f'_cA_g or 0.6 f'_cA_g and reversible quasi-static lateral loading applied to a stub. The tests indicated that for columns with heavy axial loading, where the compressive force in the concrete made a major contribution to the flexural strength of the section, use of the compressive block parameters recommended by NZS 3101:1982 (which are similar to those in ACI 318-1989) led to an underestimate of the flexural strengths. It was also found that high tensile stresses were not reliably achieved in the transverse confining steel. While the columns with the low axial load level achieved reasonable flexural ductilities, the columns with the high axial load level achieved very limited flexural ductilities.

Concrete Compressive Stress Block Parameters

The parameters of the equivalent rectangular compressive stress block for concrete are obtained by making the area of the actual and the rectangular stress distributions equal and their centroids collinear (see Fig. 9).

The equivalent rectangular compressive stress block for concrete that was recommended by the NZS 3101:1982 is identical to that recommended by ACI 318-89 (American Concrete Institute, 1989). It has a mean stress of $0.85f'_c$ and the ratio $\beta_1 = a/c$ depends on f'_c but is not taken as less than 0.65, where a = depth of the equivalent rectangle and c = neutral axis depth (see Fig. 9a). For $f'_c \geq 55$ MPa, $a/c = 0.65$. While adequate for normal strength concrete these parameters may not properly represent the shape of the actual stress distribution at high concrete strength.

(a) Normal strength concrete (b) High strength concrete

Fig. 9 Equivalent Concrete Compressive Stress Blocks

Assuming that when $f'_c \geq 80$ MPa the actual shape of the concrete compressive stress block at the flexural strength is a triangle with peak stress $k_3f'_c$ (see Fig. 9b), the equivalent rectangle can be shown to have depth a where $\beta_1 = a/c = 2/3$ and mean stress $\alpha_1 f'_c = 0.75\ k_3f'_c$. For high strength concrete k_3 may be less than 1.0, as has been found for normal strength concrete (Park and Paulay, 1975). However, $k_3 = 1.0$ and a triangular stress distribution may be assumed for high strength concrete since although k_3 may in fact be smaller (for example, 0.9) the actual shape of the stress block will be slightly curved rather than triangular with an average stress of about $0.5f'_c$. Therefore $k_3 = 1.0$ is a reasonable approximation and hence $\alpha_1 = 0.75$ may be assumed.

In NZS 3101:1995 it is recommended that the currently used parameters for the equivalent rectangular concrete compressive stress block are applicable up to $f_c^{'}$ = 55 MPa. For $f_c^{'}$ > 55 MPa it is recommended that β_1 remains at 0.65, and α_1 reduces linearly with increase in $f_c^{'}$ to become a minimum of 0.75 at $f_c^{'}$ = 80 MPa. That is, the equivalent rectangular concrete compressive stress block has a mean stress of $\alpha_1 f_c^{'}$ uniformly distributed over a depth a, where $\beta_1 = a/c$, where

$$\alpha_1 = 0.85 \quad \text{for } f_c^{'} \leq 55 \text{ MPa}$$
$$\text{or} \quad 0.85 - 0.004\,(f_c^{'} - 55) \quad \text{for } f_c^{'} > 55 \text{ MPa} \tag{22}$$
but not less than 0.75, which is reached when $f_c^{'}$ = 80 MPa.

$$\beta_1 = 0.85 \quad \text{for } f_c^{'} \leq 30 \text{ MPa}$$
$$\text{or} \quad 0.85 - 0.008\,(f_c^{'} - 30) \quad \text{for } f_c^{'} > 30 \text{ MPa} \tag{23}$$
but not less than 0.65, which is reached when $f_c^{'}$ = 55 MPa.

Confining Reinforcement for High Strength Concrete Columns

Recent moment-curvature analyses conducted by Li B et al, 1994, using stress-strain models for confined high strength concrete recently derived at the University of Canterbury have indicated that Eqs 1 and 2 may be applicable to high strength concrete columns with $f_c^{'}$ up to 100 MPa. However, a limitation on the attainable stress in the transverse reinforcement is necessary because high strength concrete exhibits less internal microcracking than normal strength concrete at similar high imposed axial compressive strains. The lower lateral expansion of high strength concrete means that the stress in the transverse reinforcement at the peak load of a high strength concrete column may be less than the yield strength if very high strength transverse steel is used. Li B et al, 1994 have used an empirical approach to estimate this stress. An urgent research topic for the future is the accurate determination of the attainable stress in transverse reinforcement when used for the confinement of high strength concrete.

NZS 3101:1995 recommends the use of Eqs 1 and 2 for columns with $f_c^{'}$ up to 70 MPa, but if high strength transverse reinforcement is used the attainable yield strength of that steel is not permitted to exceed 800 MPa.

Use of Mixed Grade Longitudinal Reinforcement

The use of very high strength longitudinal reinforcement mixed with normal strength longitudinal reinforcement in columns has also been tested (Tanaka et al, 1994). The reason for the use of mixed grade longitudinal reinforcement is to achieve gradual attainment of yield of the longitudinal bars, from normal to very high strength, as the column curvature increases near the ultimate limit state, thus ensuring that the post-yield branch of the moment-curvature relation continues to rise. Columns with $f_c^{'}$ of 60 or 72 MPa, transverse reinforcement with yield strength of 1,275 MPa, and mixed longitudinal reinforcement with yield strengths of 430 and 930 MPa, have been tested subjected to simulated seismic loading with very good results.

Tests on High Strength Concrete Beam-Column Joints

In the design of moment resisting reinforced concrete frames. The use of high strength concrete, leading to smaller column sizes, may mean that anchorage of longitudinal beam bars passing through beam-column joints would need to be achieved over smaller lengths.

A series of one-way beam-column joint subassemblages have been tested by Xin et al, 1992. The concrete compressive cylinder strength f'_c ranged between 31 and 61 MPa. The longitudinal steel reinforcement was Grade 430. Quasi-static cyclic lateral loading was applied to the beam-column joint subassemblages. It was found that when plastic hinges formed in the beams at the column races, the permitted beam bar diameter to column depth ratio d_b/h_c could be increased in proportion to $\sqrt{f'_c}$, as expressed by Eqs 17 and 18. For example, the d_b/h_c ratio used for ductile frames could be 1/21 if f'_c = 60 MPa rather than 1/36 if f'_c = 20 MPa.

RETROFITTING OF EXISTING REINFORCED CONCRETE BUILDING FRAMES

Background

The assessment of the seismic risk of older building frames, designed to now sub-standard codes, and retrofitting where necessary, is a very important topic for research (Park, 1992b). Lack of sufficient transverse reinforcement in beams, columns and beam-column joints and lack of column flexural strength are common deficiencies. A typical moment resisting frame of a seven-storey building designed in the late 1950s has assessed and full scale replicas of portions of the frame constructed and tested subjected simulated seismic loading.

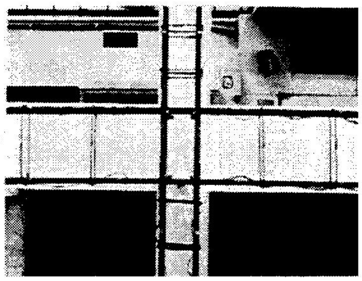
Tests on Building Columns

Four full-scale replicas of the columns of the frame have been tested to establish the performance of the frame as-built and then after retrofitting by jacketing with reinforced concrete (Rodriguez and Park, 1994). The column units as tested had a 350 mm square cross section and the eight 20 mm diameter plain round longitudinal bars were tied by 6 mm diameter hoops at 265 mm centres. A stub was present at the midheight of each test column to represent a portion of the two-way beams and slab at the beam-column joint. The column units were loaded by constant axial compression and quasi-static cyclic lateral loading was applied to the stub. The tests on the as-built columns indicated that, because of the very small quantity of transverse reinforcement present, the available displacement ductility factor was poor, in the order of 2.

Retrofitting was conducted by first roughening the surface of the existing concrete column by chipping to a depth of 2 or 3 mm. Also, in the case of the previously tested and damaged columns, all loose concrete was removed. The new longitudinal reinforcement was passed through holes made in the floor slab. The new transverse reinforcing steel was placed around the existing columns, including the region of the beam column-joint. The thickness of new concrete placed around the existing column was 100 mm. Excellent ductility as achieved from the retrofitted columns. The performance of all retrofitted columns was quite similar, indicating that the previous damage to the as-built columns had little influence on the behaviour of the retrofitted columns.

Tests on Beam-Column Joints

Several replicas of the beam-interior column joint regions of the building have also been tested under quasi-static lateral cyclic loading (Hakuto et al, 1994 and 1995). The beam cross sections were 460 mm deep x 300 mm wide and the column cross sections were 300 mm deep in the direction of the beam x 500 mm wide. There was very little transverse reinforcement in the beams and columns and there was none or very little in the joint core (see Fig. 10a). The test on the as-built subassemblage confirmed that the performance of the beam-interior column joint region of the as-built frame would be poor in a major earthquake. The maximum horizontal nominal joint shear stress, calculated from the horizontal shear force imposed on the joint divided by the column area, was $0.61\sqrt{f'_c}$ MPa, where f'_c was 41 MPa. The longitudinal reinforcement was from deformed bars with a measured yield strength of 325 MPa. The ratio of diameter of longitudinal beam bar to column depth was 1/12.5. The theoretical flexural strength of the columns was less than that of the beams. The measured storey shear versus horizontal displacement relationships are shown in Fig. 11. The horizontal dashed lines show the theoretical strength based on the flexural strength



(a) Reinforcement

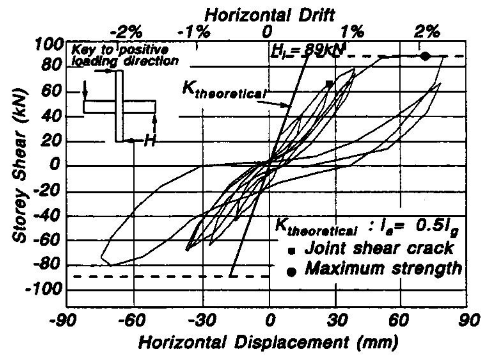
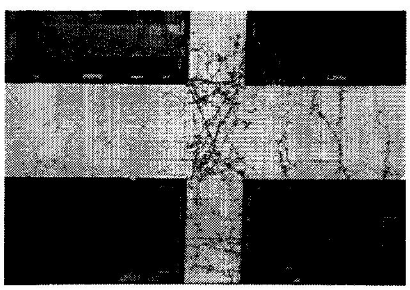


Fig. 11 Storey shear versus horizontal displacement for the as-built interior beam-column joint (Hakuto et al, 1994)



(b) Observed cracking at 2% storey drift

Fig. 10 The as-built interior beam-column joint (Hakuto et al, 1994)

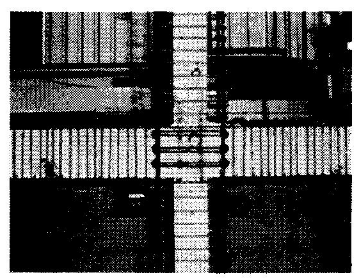


Fig. 12 Reinforcement in jacket of the beam column subassemblage (Hakuto et al, 1994)

of the columns. The columns did not reach their theoretical flexural strength in one direction of loading due to shear failure of the joint core (see Fig. 10b). The joint core region became extremely flexible, which would lessen the response of the building to a typical major earthquake, but the structural damage would be significant.

The damaged and an undamaged as-built subassemblage were retrofitted by jacketing the beams and columns (see Fig. 12). The maximum horizontal nominal shear stress in the two enlarged joint cores was $0.29\sqrt{f_c'^*}$ and $0.27\sqrt{f_c'^*}$, respectively, where $f_c'^*$ is the weighted average compressive strength of the two concretes (existing and added) of the joint core. This joint shear stress was evidently low enough for joint shear failure not to occur. When tested the joints behaved satisfactory, and almost similarly, with ductile plastic hinge behaviour in the beams in spite of the fact that one of the retrofitted undamaged subassemblage had no joint core hoops.

A further undamaged subassemblage was retrofitted by jacketing the column only. The maximum horizontal nominal shear stress in the enlarged joint was a $0.23\sqrt{f_c'^*}$. The joint behaviour was satisfactory, in spite of the absence of joint core hoops, but the beams did not behave in a ductile manner since shear failure of the beams occurred. The shear reinforcement in the beam was nominal, being capable of carrying by truss action less than 20% of the shear force acting at the flexural strength of the beam. Shear failure in the beams occurred when, with beam negative moment, the maximum nominal shear stress in the beams reached about $0.18\sqrt{f_c'}$ MPa. With beam positive moment the maximum nominal shear stress in the beams was $0.11\sqrt{f_c'}$ MPa and shear failure for that direction of shear force did not occur.

Beam-exterior column specimens from the 1950s designed building have also been tested. The specimens had a negligible amount of joint shear reinforcement. In one specimen the ends of longitudinal beam bars were bent into the joint core and the straight extension of the tail of the hooks was twelve times bar diameter, as is currently required. In the other specimen the 90° hooks at the ends of the longitudinal beam bars were not bent into the joint core (the ends of the top bars were bent up and the ends of the bottom bars were bent down) and the straight extension of the tail of the hooks was four times bar diameter, as was required by the New Zealand standard of the 1950s. The specimen with bar ends bent into the joint core performed quite well during simulated seismic loading, but the specimen with bar ends bent out of the joint core performed very poorly

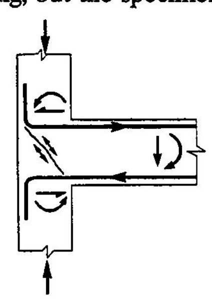


Fig. 13 Diagonal compression strut in joint core not properly engaging beam bar anchorages (Hakuto et al, 1995)

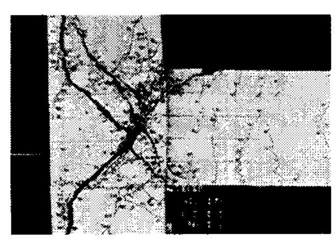


Fig. 14 Observed cracking of specimen with beam bars tails bent out of the joint at second cycle to DF of -8 (Hakuto et al, 1995)

and did not reach its flexural strength. This is because when the beam bar hooks are not bent intothe joint core the beam bars cannot properly engage the corner to corner diagonal compression strut within the joint core (see Fig. 13). As a result, in the test the diagonal compression strut pushed the longitudinal column steel off the columns (see Fig. 14).

PRECAST CONCRETE

Cast-in-Place Concrete Connections Between Precast Concrete Elements of Moment Resisting Frames

The seismic design and construction of moment resisting frames incorporating precast concrete elements requires satisfactory methods for connecting the precast elements together. If the connections between the precast elements are placed in potential plastic hinge regions, the design approach in New Zealand is to ensure that the behaviour of the connection region approaches that of a monolithic cast in place concrete structure. Two common arrangements of precast concrete members and cast in place concrete, forming ductile moment resisting multi-storey reinforced concrete frames, commonly used for strong column-weak beam designs in New Zealand, are shown in Fig. 15.

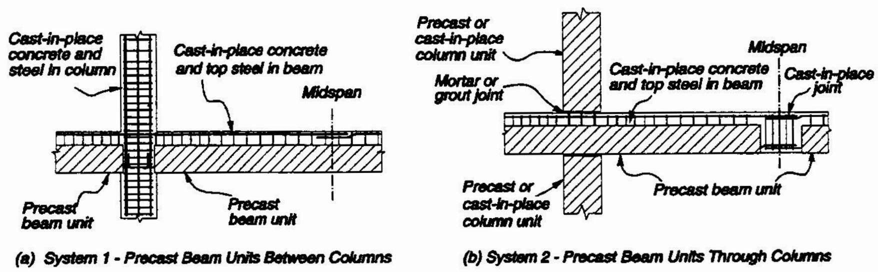


Fig. 15 Two common arrangements of precast members and cast in place concrete for constructing equivalent monolithic moment resisting concrete frames

Many of the currently used connection details have had experimental verification (Restrepo et al, 1992). Simulated seismic loading tests have been conducted on connections between beams and columns, to determine the performance of hooked bar anchorage of the bottom bars of the beam in the cast in place concrete joint core in System 1 of Fig. 15, and the performance of the vertical column bars which passed through vertical ducts in the precast beam and were grouted in System 2. Simulated seismic loading tests have also been conducted to determine the performance of cast in place mid-span connections between precast beam elements. The points of interest were the type of spliced connection of longitudinal beam bars (straight splice, hooked splice or diagonal reinforcement) and the distance of the splice from the column face. It was found that behaviour equivalent to totally cast in place concrete construction could be achieved by properly designed connections. The results of the experimental studies had led to design provisions in NZS 3101:1995.

Floors

Commonly in New Zealand floors are constructed from precast concrete units. The precast units generally act compositively with a cast-in-place concrete topping slab of 65 mm thickness which contains at least the minimum reinforcement required for slabs, in order to transfer the seismic shear forces to the supporting structure through diaphragm action. Concern has been expressed in New Zealand that there were cases in construction where the support provided for precast floors was inadequate due to too narrow seating. In such cases floor systems could become dislodged and collapse as the result of imposed movements caused by volume change or earthquake effects which reduce or eliminate the seating length. For example, beams of ductile moment resisting frames tend to elongate when forming plastic hinges which could increase the distance spanned by precast concrete floor units. Special reinforcement at the ends of the precast units can be designed to be capable of providing an alternative loading-carrying mechanism which will permit the precast concrete floor units to remain suspended in the event of loss of end bearing.

Recent tests conducted by Mejia-McMaster and Park, 1994 have investigated three types of special reinforcement, in the form of tie bars placed in cores filled with cast in place concrete at the ends of 200 mm deep hollow-core units. The tie bars pass over precast supporting beams, as shown in Fig. 16. In one test (Test A) the vertical load was applied to the floor when the support seating was zero but when no significant horizontal displacement occurred. All three types of tie bar were found to be able to support the ultimate design load of the floor in this test by shear-friction at the ends of the floor units. In a second test (Test B) the vertical load was applied to the floor after the hollow-core unit was pulled 55 mm horizontally off its seating. Tie bars Types 1 and 3 in Fig. 16 were able to support at least the service loads of the floor even under that extreme condition. The plain round tie bars with ends hooks (Types 1 and 3) were found to perform better

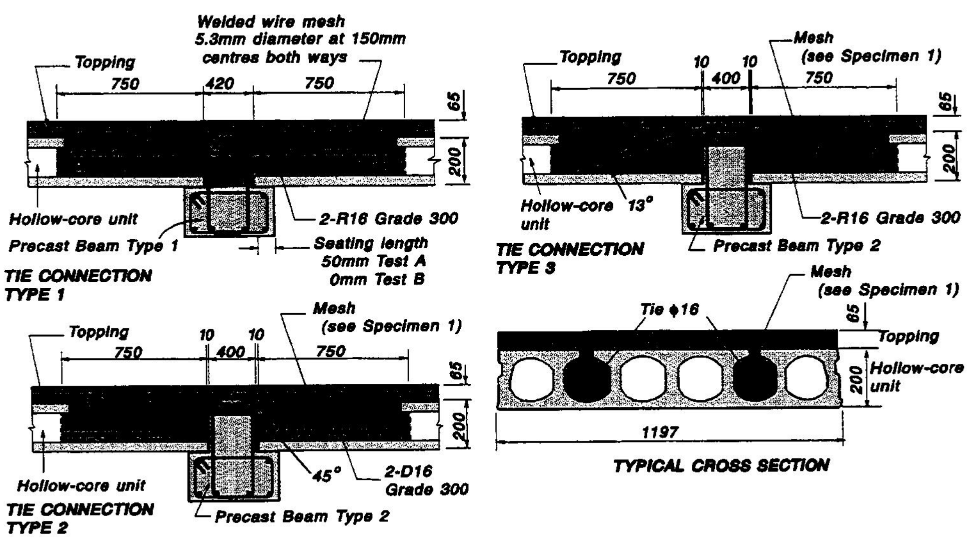


Fig. 16 Details of tie connections investigated by Mejia-McMaster and Park, 1994

than the deformed bars (Type 2), since bond failure propagating along the plain round bars allowed extensive yielding along the bar, therefore permitting substantial plastic elongation before bar fracture. It is also evident that the portions of the tie bars in the filled voids should be straight, since transverse forces caused by bends (Type 2) led to longitudinal splitting of the webs of the hollow-core units since they are without vertical shear reinforcement.

CONCLUSIONS

Considerable further experimental research into the behaviour of reinforced concrete elements and structural subassemblages subjected to be simulated seismic loading needs to be carried out to obtain a better understanding of structural behaviour. The most urgent experimental research to be conducted into aspects of seismic behaviour and design would appear to be in the areas of structural ductility and damage, the use of high strength steel and concrete, the methods for retrofitting existing structures, and the development of appropriate connections between precast concrete elements.

ACKNOWLEDGEMENTS

The considerable efforts of the academic, technical and secretarial staff of the Department of Civil Engineering of the University of Canterbury are gratefully acknowledged. Also, the New Zealand earthquake engineering profession, the New Zealand National Society for Earthquake Engineering and the Earthquake Commission of New Zealand are gratefully thanked for their encouragement and financial support.

REFERENCES

American Concrete Institute, 1989. Building Code Requirements for Reinforced Concrete. ACI 318-89. Detroit.

Cheung, P.C., Paulay T., and Park, R. 1991. Seismic Design of Reinforced Concrete Beam-Column Joints with Floor Slab, "Research Report 91-4, Department of Civil Engineering, University of Canterbury, New Zealand.

Hakuto, S., Park, R. and Tanaka, H. 1994. Behaviour of As-Built and Retrofitted Beam-Column Joints of a 1950s Designed Reinforced Concrete Building Frame, Proceedings of Technical Conference of New Zealand National Society for Earthquake Engineering, Wairakei, 16-27.

Hakuto, S., Park, R. and Tanaka, H. 1995. Seismic Performance of Existing Reinforced Concrete Building Frames, Proceedings of Technical Conference of the New Zealand National Society for Earthquake Engineering, Rotorua.

Li B., Park, R. and Tanaka, H. 1994. Strength and Ductility of Reinforced Concrete Members and Frames Constructed Using High Strength Concrete, Research Report 94-5, Department of Civil Engineering, University of Canterbury, New Zealand.

Li, X.R., Park, R. and Tanaka, H. 1991. Effects of Variations in Axial Load Levels on the Strength and Ductility of Reinforced Concrete Columns. Proceedings of Pacific Conference on Earthquake Engineering. Auckland, vol. 1, 1983-194.

Li, X.R., Park, R. and Tanaka, H. 1994. Shear Strength of Reinforced Concrete Columns Under Varying Axial Load. Proceedings of Technical Conference of the New Zealand National Society for Earthquake Engineering. Wairakei, 60-69.

Mander, J.B., Priestley, MJN and Park, R. 1988a. Theoretical Stress-Strain Model for Confined Concrete. Journal of Structural Engineering, American Society of Civil Engineers, vol. 114(8), 1804-1826.

Mander, J.B., Priestley, MJN and Park, R. 1988b. Observed Stress-Strain Behaviour of Confined Concrete. Journal of Structural Engineering, American Society of Civil Engineers, vol. 114(8), 1827-1849.

Mejia-McMaster, J.C. and Park, R. 1994. Tests on Special Reinforcement for the End Support of Hollow-Core Slabs. Journal of the Precast/Prestressed Concrete Institute, vol. 39(5), 90-105.

Park, R. and Paulay, T. 1975. Reinforced Concrete Structures, John Wiley and Sons. New York.

Park, R. and Dai Ruitong. 1988. A Comparison of the Behaviour of Reinforced Concrete Beam-Column Joints Designed for Ductility and Limited Ductility. Bulletin of the New Zealand National Society for Earthquake Engineering, vol. 21(4), 255-278.

Park, R. 1989. Evaluation of Ductility of Structures and Structural Assemblages From Laboratory Testing. Bulletin of the New Zealand National Society for Earthquake Engineering, vol. 23(3), 155-166.

Park, R. 1992a. Capacity Design of Ductile Reinforced Concrete Building Structures for Seismic Resistance. The Structural Engineer, vol. 70(16), 279-289.

Park, R. 1992b. Seismic Assessment and Retrofit of Concrete Structures: United States and New Zealand Developments. Proceedings of Annual Technical Conference of New Zealand Concrete Society, Wairakei, 18-25.

Paulay, T. and Priestley, MJN 1992. Seismic Design of Reinforced Concrete and Masonry Buildings, John Wiley, New York.

Priestley, MJN and Park, R. 1987. Strength and Ductility of Concrete Bridge Columns Under Seismic Loading. Structural Journal of American Concrete Institute, vol. 84(1), 61-76.

Restrepo, J.I., Park, R. and Buchanan, A.H. 1992. Seismic Behaviour of Connections Between Precast Concrete Elements, Research Report 93-3, Department of Civil Engineering, University of Canterbury, New Zealand, 1993.

Rodriguez, M. and Park, R. 1994. Seismic Load Tests of Reinforced Concrete Columns Strengthened by Jacketing. Structural Journal of American Concrete Institute, vol. 91(2), 150-159.

Standards Association of New Zealand, 1982. Design of Concrete Structures, NZS 3101:1982, Wellington.

Standards New Zealand, 1995. Design of Concrete Structures, NZS 3101:1995, Wellington.

Tanaka, H., Sato, Y., Park, R. and Kani, N. 1994. High Strength Concrete Columns With Longitudinal Reinforcement of Mixed Grades. Proceedings of ACI International Conference on High Performance Concrete, Singapore, 391-411.

Watson, S., Zahn, F.A. and Park, R. 1994. Confining Reinforcement for Concrete Columns. Journal of Structural Engineering, American Society of Civil Engineers, vol. 120(6), 1798-1824.

Watson, S. and Park, R. 1994. Simulated Seismic Load Tests on Reinforced Concrete Columns. Journal of Structural Engineering, American Society of Civil Engineers, vol. 120(6), 1825-1849.

Xin, Z.X., Park, R. and Tanaka, H. 1992. Behaviour of Reinforced Concrete Interior Beam-Column Joints Designed Using High Strength Concrete and Steel. Research Report 92-3. Department of Civil Engineering, University of Canterbury.

Zahn, F.A., Park, R. and Priestley, MJN 1986. Design of Reinforced Concrete Bridge Columns for Strength and Ductility. Research Report 86-7, Department of Civil Engineering, University of Canterbury.