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THE REVISED NEW ZEALAND CONCRETE DESIGN STANDARD

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ABSTRACT

A revised New Zealand concrete design standard, NZS 3101:1995 was published in 1995 to replace the standard that has been in use since 1982. Some of the significant changes made in the revised standard affecting seismic design are the possible use of higher material strengths and new design provisions for confining reinforcement in columns, horizontal and vertical shear reinforcement in beam-column joints, size of reinforcing bars passing through beam-column joints, support of precast floor systems, and structures of limited ductility.

KEYWORDS

Bond strength, concrete design standard, confining reinforcement, ductile design, limited ductility design, material strengths, precast floor support, New Zealand standard, reinforced concrete, shear reinforcement.

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) was published in June 1995. Several changes to the seismic design provisions have been incorporated reflecting the advances made as a result of research and development since the late 1970s. This paper summarises some of the changes made to the seismic provisions.

MATERIAL STRENGTHS

NZS 3101:1995 (Standards New Zealand, 1995) has increased the strength of concrete and steel reinforcement which can be used in seismic design. It is recommended that the specified compressive cylinder strength of the concrete f'_c for ductile and limited ductile elements in seismic design shall not exceed 70 MPa. This limitation is mainly because of the difficulty of providing sufficient transverse reinforcement to confine very high strength concrete to make it adequately ductile.

With regard to steel reinforcement, NZS 3101:1995 requires the lower characteristic (5% percentile value) yield strength f_y of longitudinal reinforcement used in design to be not greater than 500 MPa.

Similarly, in the design of longitudinal reinforcement, and transverse reinforcement for shear, the yield strength used in design is not permitted to be taken as greater than 500 MPa. These limitations are to restrict the widths of flexural and diagonal tension cracks in the concrete at service loads. However, in the design of transverse reinforcement in columns to confine the concrete and to provide restraint against lateral buckling of longitudinal reinforcement, NZS 3101:1995 permits a yield strength as high as 800 MPa to be used in the design calculations. This means that in practice a high strength transverse reinforcement with a lower characteristic yield strength equal to or greater than 800 MPa can be used providing that the value of the yield strength used in design does not exceed 500 MPa for shear calculations and 800 MPa for concrete confinement and bar lateral restraint calculations.

DESIGN SEISMIC FORCES

NZS 3101:1995 (Standards New Zealand, 1995) specifies values for the structural ductility factor μ to be used when determining the design seismic forces at the ultimate limit state. The seismic acceleration coefficients are determined from acceleration response spectra with ordinates reduced appropriately to account for the magnitude of the assumed available structural ductility factor. The structural ductility factor, also known as the displacement ductility factor, is $\mu = \Delta_u/\Delta_y$ where Δ_u = ultimate lateral displacement (defined as that displacement when after four cycles of lateral loading to it in each direction the lateral load strength of the structure has not reduced by more than 20%) and Δ_y = lateral displacement at first yield. The values of μ to be used in design are those shown in Table 1.

Table 1 Maximum available structure ductility factor μ to be assumed in design when determining the design seismic forces at the ultimate limit state

	Type of Structure	Reinforced Concrete
1	Elastically responding structures	$\mu = 1.25$
2	Structures of limited ductility	
	(a) Moment resisting frames	$\mu = 3$
	(b) Walls	3
	(c) Cantilevered face loaded walls (single storey only)	2
3	Ductile structures	
	(a) Moment resisting frames (see also below)	$\mu = 6$
	(b) Walls	
	(i) Two or more cantilevered	$5/Z$
	(ii) Two or more coupled	$\frac{5}{Z} \le \frac{3A + 4}{Z} \le \frac{6}{Z}$
	(iii) Single cantilever	$4/Z$

In Table 1, $1.0 \le Z = 2.5 - 0.5A_r \le 2.0$ and $1/3 \le A = TL/M_o \le 2/3$, where $A_r = h_w/L_w$, h_w = total vertical height of wall from base to top, L_w = horizontal length of wall, T = axial load induced at the bases of coupled walls, L = distance between the vertical reference axes of coupled walls, and M_o = total overturning moment at base of structure comprising coupled structural walls.

For gravity load dominated ductile frames with strong beams it may not be feasible to prevent some relatively weak columns (typically interior columns) from developing plastic hinges simultaneously at the top and bottom ends, while the other columns in that storey remain in the elastic range. In that case the value of μ used in design is not to exceed 12 times the ratio of the shear resisted by the columns remaining in the elastic range in that storey to the total storey shear, but not greater than 6.

Evaluation of Column Actions in Ductile Moment Resisting Frames

The capacity design rules for protecting columns of tall moment resisting frames, by ensuring that as far as possible strong column-weak beam behaviour occurs, that were introduced in NZS 3101:1982 (Standards Association of New Zealand, 1982) have remained practically the same in NZS 3101:1995 (Standards New Zealand, 1995). Those rules involve determining the design column actions by multiplying the column bending moments and shear forces determined from elastic frame analysis, for the load cases involving the ultimate design seismic force, by factors which take into account the beam flexural overstrength, higher mode effects and concurrent seismic forces. The multipliers depend on the frame variables. Also, the design axial loads in columns are found taking into account beam flexural overstrength and gravity loads. Strong beam-weak column design is only permitted for gravity load dominated frames, as indicated previously, or for one or two storey frames.

Equations for Quantities of Transverse Confining Reinforcement in Columns

As a result of recent tests and analytical studies in New Zealand (Watson et al, 1994) the quantities of transverse reinforcement recommended in NZS 3101:1995 for the confinement of concrete in the potential plastic hinge regions of columns of ductile frames have been made more dependent on the level of the axial load, resulting in less confining steel in lightly loaded columns and more confining reinforcement in heavily loaded columns that was recommended in NZS 3101:1982. For lightly loaded columns the requirement for sufficient transverse reinforcement to prevent premature buckling of longitudinal bars is more critical than for confinement of the concrete. The design axial compressive load on columns is not permitted to exceed 0.7 N_o, where N_o = concentric load strength of the column.

For columns with spirals or circular hoops, NZS 3101:1995 requires that the volumetric ratio of transverse reinforcement p_s should not be less than that given by Eq. 1 for confinement of concrete nor Eq. 2 for restraint of longitudinal reinforcement against premature buckling:

$$p_s = \frac{\left(\dfrac{\phi_u}{\phi_y} - 33p_t m + 22\right)}{79} \frac{A_g}{A_c} \frac{f_c'}{f_{yt}} \frac{N^*}{\phi f_c' A_g} - 0.0084 \qquad (1)$$

$$p_s = \frac{A_{st}}{110d''} \frac{f_y}{f_{yt}} \frac{1}{d_b} \qquad (2)$$

where p_s = ratio of volume of spiral or circular hoop reinforcement to volume of concrete core of column, ϕ_u = ultimate curvature of column, ϕ_y = curvature of column at first yield, $p_t = A_{st}/A_g$, A_{st} = total area of longitudinal column reinforcement, $m = f_y/0.85f_c'$, f_y = yield strength of longitudinal steel, A_g = gross area of column, A_c = core area of column measured to outside of spiral or circular hoop, f_{yt} = yield strength of transverse steel (not to exceed 800 MPa), f_c' = concrete compressive cylinder strength, N^* = axial compressive load on column, ϕ = strength reduction factor, d_b = diameter of longitudinal bars, and d'' = diameter of concrete core of column defined as for A_c.

For columns with rectangular hoops and cross ties, NZS 3101:1995 requires that the total area of the transverse bars in the direction under consideration for concrete confinement within vertical spacing s_h is given by $A_{sh} = 0.73\, s_h h'' p_s$ where p_s is given by Eq. 1 and h'' = dimension of concrete core measured perpendicular to the direction of the transverse bars under consideration to the outside of the peripheral hoop. For restraint against lateral buckling the yield force of the transverse bar applying restraint should be at least $s_h/96d_b$ times the yield force of the restrained longitudinal bar or bars, where s_h = vertical spacing of hoop legs or cross ties and d_b = diameter of longitudinal bars.

For ductile moment resisting frames when the design seismic forces at the ultimate limit state are determined using a displacement ductility factor $\mu = 6$, NZS 3101:1995 requires design for $\phi_u/\phi_y = 10$ for the potential plastic hinge regions of columns above the bottom storey of frames where strong column-weak beam design is used, and $\phi_u/\phi_y = 20$ for the potential plastic hinge regions of the bottom storey columns of frames where strong column-weak beam design is used or of columns of one or two storey frames where strong beam-weak column design is permitted.

The transverse reinforcement must also be checked to ensure that it is satisfactory for shear. The requirement for shear reinforcement may be more critical than for confinement or bar restraint.

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 a square column when $\phi_u/\phi_y = 20$ is required is shown in Fig. 1 for various axial compressive load levels. Note that the requirement for concrete confinement 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 requirements of the 1989 ACI building code (American Concrete Institute, 1989) are also shown.

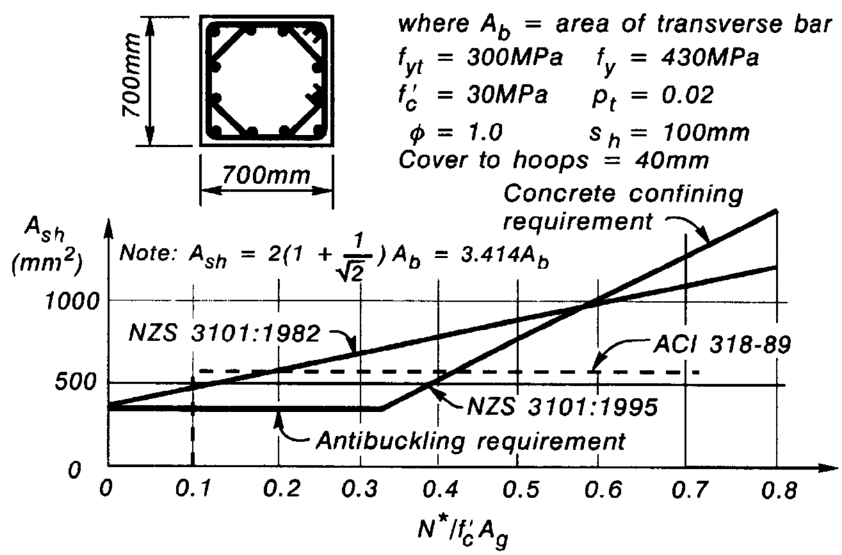

Fig. 1 Example of transverse reinforcement required for a ductile column

Length of End Region of Column to be Confined

The confined length of column adjacent to the section of maximum bending moment (see Fig. 2) 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).

Figure 2 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.

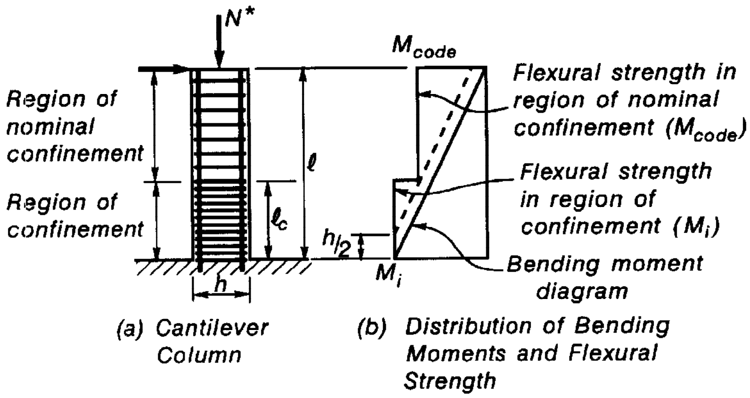

Fig. 2 Determination of the length of the confined end region of column

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:

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

where the notation is the same as for Eq. 1. It is evident that the confined length ℓ_c should 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 the column depth 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.

BEAM-COLUMN JOINTS

The provisions of NZS 3101:1982 (Standards Association of New Zealand, 1982) for shear reinforcement in beam column joints of ductile moment resisting frames were of necessity conservative, due to the limited test information available in the late 1970s when those provisions were drafted. In the light of recent tests and analytical studies (for example: Park and Dai, 1988; Cheung et al, 1991) in NZS 3101:1995 (Standards New Zealand, 1995) the quantities of shear reinforcement required in joint cores of ductile frames is significantly lower (at least 30% less) than that required by NZS 3101:1982, and the equations used to calculate it are simpler than those of NZS 3101:1982.

The assessment of the shear strength of beam-column joints is based on the contributions of two generally recognised mechanisms; one consisting of a single diagonal compression concrete strut assumed to be capable of transferring both horizontal and vertical joint shear forces without the aid of reinforcement, the other a truss mechanism utilising horizontal and vertical joint shear reinforcement and concrete struts. In NZS 3101:1982 it was considered that when plastic hinges occurred in the beams at the faces of columns carrying small axial loads, the truss mechanism had to transfer all the joint shear. NZS 3101:1995 it is recognised that part of the total force in the longitudinal beam bars passing through the joint core will be transferred to the diagonal compression strut by bond over a length where significant vertical concrete compression stresses are introduced by the column. Thus a significant part of the joint shear can be transferred by the diagonal compression strut mechanism leaving the remainder to be transferred by truss action of the joint core shear reinforcement.

NZS 3101:1995 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 $v_{jh} = V_{jh}/(b_j h_c)$ where V_{jh} is the design horizontal joint shear force acting on the joint core calculated using the overstrength longitudinal steel forces and the design column shear force, 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.

For horizontal joint shear NZS 3101:1995 requires that the area of total effective horizontal joint shear reinforcement corresponding to each direction of horizontal joint shear force shall at least be:

For interior joints

$$A_{jh} = \frac{6v_{jh}}{f_c'}\left(1.4 - 1.6\,\frac{C_j N^*}{f_c' A_g}\right)\frac{f_y}{f_{yh}}\,A_s^* \tag{4}$$

For exterior joints

$$A_{jh} = \frac{6v_{jh}}{f_c'}\,\beta\left(0.7 - \frac{C_j N^*}{f_c' A_g}\right)\frac{f_y}{f_{yh}}\,A_s \tag{5}$$

where v_{jh} = nominal horizontal shear stress in joint core, f_c' = compressive cylinder strength of concrete, N^* = axial load on column, taken as negative when causing tension, $C_j = V_{jh}/(V_{jx} + V_{jx})$, 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), A_s^* = greater of area of top or bottom beam reinforcement passing through the joint excluding the area of bars in effective tension flanges, 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 cannot form at the column face f_y in Eqs 4 and 5 can be replaced by 0.8 f_s where f_s = computed tensile stress of longitudinal beam steel at the column face.

For vertical joint shear NZS 3101:1995 requires that the total area of effective vertical joint shear reinforcement, with columns that are expected to remain essentially in the elastic range, corresponding to each of the two directions of joint actions, shall be:

$$A_{jv} = \left(\frac{0.7}{1 + (N^*/f_c' A_g)}\right)\frac{h_b}{h_c}\,A_{jh}\,\frac{f_{yh}}{f_y} \tag{6}$$

where h_b = depth of beam and h_c = depth of column. The vertical joint shear reinforcement will 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

Recent studies (Park and Dai, 1988 and Cheung et al, 1991) have indicated that a number of factors need to be taken into account when determining anchorage lengths for longitudinal reinforcement passing through interior beam-column joints. Based on their considerations, the requirements of NZS 3101:1995 for the anchorage of longitudinal bars passing through interior beam-column joints are:

For longitudinal beam bars the ratio of longitudinal beam bar diameter to column depth shall satisfy:

$$\frac{d_b}{h_c} \le 3.3\,\alpha_f \frac{\sqrt{f_c'}}{\alpha_o f_y} \tag{7}$$

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. NZS 3101:1995 also recommends as alternative to Eq. 7 a further equation involving more parameters which gives some relaxation in the d_b/h_c ratio required by Eq. 7.

For longitudinal column bars when columns are designed to develop plastic hinges in the end regions, the ratio the longitudinal column bar diameter to beam depth shall satisfy:

$$\frac{d_b}{h_b} \le 3.2 \frac{\sqrt{f_c'}}{f_y} \tag{8}$$

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. 8 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 7 and 8 will generally allow larger d_b/h_c and d_b/h_b ratios than were permitted by NZS 3101:1982, particularly for high strength concrete.

SUPPORT OF PRECAST CONCRETE FLOORS

Adequate support of precast concrete floor units is one of the most basic requirements for a safe structure. In the design of the length of the seating in the direction of the span, allowances must be made for tolerances and for the effects of volume changes due to concrete shrinkage, creep and temperature affects, and due to deformations due to flexure. NZS 3101:1995 (Standards New Zealand, 1995) requires that for precast concrete floor or roof members, with or without the presence of a cast-in-place concrete topping slab and/or continuity reinforcement, that unless shown by analysis or test that the performance of alternative details will be acceptable, each member and its supporting systems shall have design dimensions selected so that, under a reasonable combination of unfavourable construction tolerances, the distance from the edge of the support to the end of the precast member in the direction of its span is at least 1/180 of the clear span but not less than: 50 mm for solid and hollow-core slabs or 75 mm for beams or ribbed members. One method of permitting a reduction in the distance from the end of the support to the end of the precast member by test or analysis is to use special reinforcement to carry the vertical loading in the event of loss of bearing at the seating of the precast concrete elements, due to bearing failure or to the precast elements being dislodged. Hanger bars or saddle bars, or horizontal or draped reinforcement, can be used, anchored in the precast elements (for example, grouted into the cores of hollow-core units which have been broken out at the top) and passing over the top of the supporting beam (see Mejia-McMaster and Park, 1994).

SEISMIC REQUIREMENTS FOR STRUCTURES OF LIMITED DUCTILITY

Structures of limited ductility possess strength sufficient to resist, or are designed for, seismic forces between the levels corresponding to elastic response (μ = 1.25) and fully ductile response (μ = 6), where μ is the structural ductility factor. According to NZS 3101:1995 (Standards New Zealand, 1995) structures or structural elements of limited ductility are assigned structural ductility factors μ of 2 or 3. The design provisions for structures of limited ductility have been expanded and rewritten in

NZS 3101:1995 to enable the design of a range of structures of limited ductility. For example, structures may have large inherent strength, such as some walls of small buildings, and hence need only be designed for the limited ductility associated with the available high lateral load strength. Other structures which because of their irregular configuration, such as walls with badly positioned openings, cannot be reliably designed as ductile structures and need to be designed for limited ductility and the associated higher design seismic forces. Also, the columns of moment resisting frames in structures containing also relatively stiff structural walls will typically be required to deform only to the extent of requiring limited ductility. Tall structures in zones of low seismicity, which have lateral load strength governed by wind rather than seismic actions, may need to be designed only for limited ductility.

The design approach for structures of limited ductility recommended by NZS 3101:1995, as for ductile structures, uses capacity design to ensure that an appropriate mechanism of plastic deformation develops in a severe earthquake and that shear failures are avoided. The detailing of structures of limited ductility is made less onerous than for ductile structures where appropriate.

CONCLUSIONS

Several changes have been made to the seismic provisions of the recently published New Zealand concrete design standard NZS 3101:1995 as a result of the research and development that has been conducted since the 1982 edition was written. These modifications include the permitted use of higher strength concrete and steel reinforcement and changes in the quantities of confining reinforcement required for columns which may result in less confining steel in lightly loaded columns, some relaxation in the provisions for beam-column joints of frames, new provisions for precast concrete, and new provisions for the design of structures of limited ductility.

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REFERENCES

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

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

Mejia-McMaster, J.C. and R. Park (1994). Tests on special reinforcement for the end support of hollow-core slabs. Journal of the Precast/Prestressed Concrete Institute, 39, 5, 90-105.

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, 21, 4, 255-278.

Priestley, M.J.N. and R. Park (1987). Strength and ductility of bridge columns under seismic loading. Structural Journal of the American Concrete Institute 84, 1, 61-76.

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

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

Watson, S., F.A. Zahn and R. Park (1994). Confining reinforcement for concrete columns. Journal of Structural Engineering of the American Society of Civil Engineers, 120, 6, 1798-1824.

Watson, S. and R. Park (1994). Simulated seismic load tests on reinforced concrete Columns. Journal of Structural Engineering of the American Society of Civil Engineers, 120, 6, 1825-1849.