RECOMMENDATIONS FOR THE DESIGN AND DETAILING OF DUCTILE PRESTRESSED CONCRETE FRAMES FOR SEISMIC LOADING

Prepared by

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TEXT

1.0 SCOPE

This document includes recent research information and suggests minimum requirements for the design of prestressed and partially prestressed concrete frames to resist seismic forces. They are based on ACI 318-71[3] where appropriate and shall be read in conjunction with NZS 4203[2], and are supplementary to NZSR 32[1]. The recommendations are not to be read as a code of practice.

2.0 GENERAL REQUIREMENTS

The philosophy of design shall be as required by NZS 4203[2] which is summarised as follows:

A prestressed concrete ductile moment resisting frame of more than two storeys shall be capable of dissipating seismic energy in a flexural mode by the formation of plastic hinges in beams of adequate ductility. For frames of more than two storeys the flexural strength of columns shall be such that mechanisms of collapse involving plastic hinges forming simultaneously at the upper and lower regions of the columns of one storey cannot occur. All forms of brittle failure shall be prevented. Consideration shall be given to the probable increase of material strengths to above their design values.

COMMENTARY CLAUSES

C1.0 NZSR 32[1] contains no provisions for the seismic design of prestressed concrete structures. These recommendations for the detailing of ductile prestressed and partially prestressed concrete frames for seismic loading are intended to supplement NZSR 32. The recommendations are based on recent research evidence and tend to follow American practice used for reinforced concrete where appropriate. The design principles and loadings of NZS 4203[2] should also be complied with.

C2.0 The object is to design structures capable of behaving in a ductile manner when responding to a major earthquake. Ductile behaviour is best obtained by ensuring that the beams reach flexural capacity before alternative more brittle failure states are reached, and that the beam plastic hinges are capable of undergoing significant inelastic rotation under cyclic loading without appreciable reduction in flexural strength. In general, column sidesway mechanisms involving plastic hinges in the columns of one storey only are undesirable because of the high ductility demand on the plastic hinges in the columns. Hence the flexural strength of columns should be such that mechanisms of collapse involving plastic hinges existing simultaneously in the upper and lower regions of the columns of one storey cannot occur. For one and two storey frames the ductility demand on the column hinges may be not so high and column sidesway mechanisms may be permitted for such frames if it can be shown that the ductility demand can be met. It must also be ensured that all brittle forms of failure, such as due to shear, are prevented. The strength of materials in members will generally be higher than the values specified in design and will result in the actual strengths of the members being greater than calculated, using the specified material strengths. This enhanced member capacity, if unaccounted for, may have the effect of inducing alternative undesirable modes of failure. For example, enhanced beam flexural strength may result in a failure in shear rather than yielding in flexure. A capacity design approach should be adopted whereby the probable overstrength of materials at plastic hinge zones is taken into account in determining levels of strength required to avoid brittle failure states. For prestressing steel the actual stress-strain curves for the prestressing steel should be used in the flexural strength computations. Note that the prestress losses caused by friction, etc. do not need to be considered for the ultimate load case as they will have negligible effect. The major difference between the nonlinear response of prestressed concrete structures and reinforced concrete structures to seismic ground motions is that the narrower load-deflection hysteresis loops of prestressed concrete mean that less energy is dissipated than for reinforced concrete when responding to inelastic loading cycles. Because of this, a greater lateral displacement response may be expected for a prestressed concrete structure than for a reinforced concrete structure of the same strength when subjected to severe earthquake motions[4]. This factor is recognised in NZS 4203[2] which requires a 20% greater materials factor for prestressed concrete than for reinforced concrete, resulting in a 20% greater design horizontal seismic force for a prestressed concrete frame than for an equivalent reinforced concrete frame. Even with this greater materials factor however, resulting in higher member strengths than for an equivalent reinforced concrete frame, there is no guarantee that larger lateral displacements will not occur in a prestressed concrete frame and the importance of this with respect to possible damage to nonstructural elements should be considered by the designer. An alternative to fully prestressed members is to use partially prestressed members, the non-prestressed steel in the plastic hinge zones helping to dissipate energy and to provide additional compressive resistance during major seismic ground motions. The appropriate materials factor for such a "mixed" structure will need attention by the designer. In general it appears reasonable to adopt an intermediate figure depending on the relative amount of prestressed or non-prestressed steel in the plastic hinge zones.

3.0 MATERIALS

Wires and strands for tendons in prestressed concrete shall conform with the provisions of NZS 1417 or BS 3617 respectively or shall be of equivalent quality.

The specified concrete compressive cylinder strength shall not exceed 55 MPa (8000 psi) unless special transverse reinforcement is provided.

C3.0 It is of particular importance that the prestressing steel complies with the specified requirements for percentage elongation at rupture to ensure that it is adequately ductile. The actual stress-strain curve of the prestressing steel should be available for flexural strength calculations. It is desirable that the concrete strength does not exceed 55 MPa (8000 psi) to ensure that the concrete is not overly brittle. This is because tests indicate that the higher the concrete strength the steeper the falling branch of the concrete stress-strain curve and the smaller the extreme fibre concrete compressive strain when the member reaches ultimate moment capacity[5]. This however is not the case if special transverse reinforcement is provided to confine the concrete, in which case this upper limit of compressive strength may be waived.

4.0 DESIGN OF FLEXURAL MEMBERS

4.1 The content of flexural steel (prestressed plus nonprestressed) shall be such that a/h is not greater than 0.2 at the flexural strength of sections in the plastic hinge zones, where 'a' is the depth of the equivalent rectangular concrete compressive stress block at the flexural strength and 'h' is the overall depth of the member.

4.2 The flexural strength of the section shall be greater than the cracking moment

C4.1 The object is to ensure ductile behaviour in plastic hinge zones. The requirement comes from an examination of theoretical moment-curvature curves[6], and from a comparison of provisions for reinforced concrete. For rectangular sections with the prestressing steel concentrated near the extreme tension fibre it was observed previously[6] that the normal ACI 318-71[3] and NZR 32:1968[1] requirement for preventing over-reinforced members, namely when allowance is made for likely variations in prestress and the strength of materials. In the absence of special studies, it may be assumed that the maximum concrete tensile stress prior to cracking is $1.0\sqrt{f'_c}$ MPa provided allowance is made for a variation of 10% in the calculated level of prestress at the section under consideration.

4.3 The maximum design shear force shall not be less than:

$$v_u = \frac{M_{uA} + M_{uB}}{l} + v_{dg} \qquad \ldots (1)$$

where M_{uA} and M_{uB} are the ultimate moment capacities of opposite sense at the ends of the beam taking into account the probable overstrength of materials, l is the clear span of the beam, and v_{dg} is the shear due to the design gravity loads acting on the beam treated as a simply supported span. In plastic hinge zones, web reinforcement shall be provided to carry all the design shear force and should take the form of closed stirrups of size not less than 10mm (3/8 in) in diameter placed at spacing not exceeding 100 mm (4 in) or d/4, where d is the effective depth of the member which for prestressed concrete need not be taken as less than 0.8 of the overall depth. In plastic hinge zones the transverse steel shall also be capable of providing adequate confinement to the concrete. Elsewhere in the member the spacing of web reinforcement shall not exceed d/2. Longitudinal steel shall be present at each corner of the stirrups.

4.4 The spacing of closed stirrups surrounding nonprestressed compression bars in plastic hinge zones shall not exceed 6 compression steel bar diameters, 100mm (4 in) or d/4, whichever is least. The stirrups shall be so arranged that every compression bar shall have lateral support at not greater than the above spacing provided either by the corner of a stirrup having an included angle of not more than 135° or by the equivalent welded steel arrangement. This does not apply to second (i.e. internal) layers of bars.

$$\frac{A_{ps}}{bd}\,\frac{f_{ps}}{f'_c} \leqslant 0.3$$

was inadequate for prestressed members in seismic resistant structures where the ductility demands could be more significant. It is considered that for seismic design for prestressing steel concentrated near the extreme tension fibre the requirement should be

$$\frac{A_{ps}}{bd}\,\frac{f_{ps}}{f'_c} \leqslant 0.2$$

i.e. $A_{ps} f_{ps} \leqslant 0.2 f'_c bd$.

This means that the maximum possible tensile force in the tendons at the flexural strength is $0.2f'_c bd$ and hence that the maximum possible depth of the rectangular stress block is

$$a = \frac{0.2f'_c bd}{0.85f'_c b} = 0.235d$$

But since d is approximately 0.8h the requirement may be written as

$$a \leqslant 0.2h$$

For sections with tendons at various positions down the depth of the member it is difficult to set a limiting value for $A_{ps}f_{ps}/bdf'_c$ because tendons at various levels result in sections with different moment-curvature characteristics from the case when all the tendons are placed near the extreme tension fibre. Rather than stipulating different limiting $A_{ps}f_{ps}/bdf'_c$ values for various steel arrangements it is more convenient to require that $a/h \leqslant 0.2$ for all sections. This achieves the same end result since the ultimate curvature will always be at least equal to that of the section with all tendons placed near the extreme tension fibre. Nevertheless tendons should not be concentrated only in the mid-depth of the beam section. Ideally tendons or nonprestressed reinforcement should also exist at the top and bottom of the section if centrally placed tendons are used. Sections with greater a/h ratios may need more confining steel than specified in Section 4.3 to reach the required ultimate curvature. Equations relating content of confining steel to maximum concrete compression strain are available elsewhere[5].

It has not been made mandatory to have nonprestressed longitudinal bars present at the plastic hinge sections because the web reinforcement which additionally provides some degree of concrete confinement, and the small depth of compressed concrete, should result in ductile behaviour, even at high concrete strains. Nonprestressed steel will improve the compression zone behaviour and allow greater energy dissipation but such steel is only effective if the bond throughout the beam-column joint core does not deteriorate and allow the bars to slip through the joint core. The total beam bar force to be transferred to the joint core by bond when lateral loading is applied to the frame is approximately twice the yield force of the bar and the resulting bond stresses can be very high.

If slip of steel occurs through the core there will be a loss of compression steel because the bars will actually be in tension in the compression zone and lead to a reduction in the available curvature ductility. This points to the need for relatively deep members and/or relatively small diameter longitudinal bars. For example in the Canterbury tests[7] on beam-column assemblies involving partially prestressed beams framing into a 406 mm (16 in) deep column, 28.6 mm diameter (1 1/8 in) deformed nonprestressed mild steel beam bars eventually slipped through the joint core after several cycles of loading whereas 19.1 mm diameter (¾ in) deformed mild steel beam bars did not. Note that at sections subjected to moment reversals prestressing tendons will exist at the top and bottom of the member and these will act as compression steel in the event of very large curvatures being enforced after crushing of the concrete during catastrophic loading.

C4.2 The section should crack before the flexural strength is reached, otherwise a brittle failure may result. Allowance has been made for the case of a high modulus of rupture (for example $0.83\sqrt{f'_c}$ MPa or higher is occasionally measured in tests) and the case of concrete cylinder strength f'_c being greater than specified.

C4.3 Shear failure caused by cyclic loads are generally non-ductile and must therefore be avoided by a capacity design approach. The design shear force is calculated using the design gravity loads and the beam plastic hinge moments. To ensure that the greatest probable shear force is calculated, the beam plastic hinge capacities used are those computed assuming a capacity reduction factor ϕ of 1.0 and including an allowance for probable overstrength materials. For example, in the Canterbury tests[7] the maximum moment capacities measured in prestressed concrete beams at the column faces were up to 16% greater than the theoretical values calculated using $\phi = 1$ and the measured concrete cylinder strength and the measured steel stress-strain characteristics. This strength increase was due to maximum moment occurring at an extreme fibre concrete strain of greater than 0.003 and due to the extra confinement given to the beam concrete by the proximity of the column. It should be noted that strength increase may also result from the contribution of the floor slab and Bauschinger effects in the nonprestressed steel. A capacity reduction factor of $\phi = 0.85$ should be used when calculating the capacity of the shear reinforcement. The stirrup spacing specified in plastic hinge zones is such that the stirrups are close enough to act as confining steel for the concrete as well as shear reinforcement. In plastic hinge zones the stirrups are to be closed and, to confine the concrete effectively across the width of the section, the maximum unsupported width of stirrup measured between perpendicular legs of the stirrup or supplementary crossties should be limited. 150mm between longitudinal bars supported by the corners of ties is acceptable. Concrete cover to the stirrups should be as small as possible so as to avoid a large loss of moment capacity if the cover concrete is shed at high strain levels during seismic response. Reversed loading effects are included by ignoring the shear carried by the concrete shear resisting mechanism in plastic hinge zones. For the purposes of Sections 4.3 and 4.4 plastic hinge zones may be regarded as the end regions of the beam extending over a distance of at least twice the member depth from the face of the columns, and wherever else in the member ultimate moment capacity may be developed during inelastic lateral displacement of the frame.

C4.4 The stirrups spacing should be close enough to restrain buckling of compression steel.

5.0 DESIGN OF COLUMNS

5.1 The flexural strength of a column section shall be greater than the maximum likely cracking moment as calculated in section 4.2 except that the effect of axial load shall be allowed for.

5.2 Transverse reinforcement in columns shall be provided to ensure that the shear capacity of the member is at least equal to the shear forces applied at the formation of plastic hinges in the structure, due to the combination of design lateral and gravity loadings, taking into account the probable overstrength of the materials. In the case of space frames, for the calculation of shear forces, the simultaneous development of plastic hinges in the beams in both principal directions of the frame due to earthquake loading acting in a general direction shall be considered. The transverse reinforcement for shear shall be detailed as for flexural members and end regions of columns shall be regarded as plastic hinge zones when detailing shear reinforcement. For calculation of maximum design shear force refer to 4.3.

5.3 The spacing of hoops in the end regions of columns when nonprestressed longitudinal bars are present shall not exceed six longitudinal bar diameters, 100mm (4 in) or d/4, whichever is least. The hoops shall be arranged so that every compression bar shall have lateral support provided either by the corner of a hoop having an included angle of not more than 135^{o} or by the equivalent welded steel arrangements.

5.4 Special transverse reinforcement shall be provided in the end regions of columns if the design load on the column exceeds $0.1P_o$ or if a/h is greater than 0.2 at the flexural strength or if column sidesway mechanisms are permitted in one or two storey frames, where P_o is the axial load strength of the column without flexure making allowance for the effect of prestress, 'a' is the depth of the equivalent rectangular concrete compressive stress block at the flexural strength and 'h' is the overall depth of the member.

Where a spiral (circular) is used as special transverse reinforcement, the ratio of the volume of spiral reinforcement to volume of concrete core (measured to outside of spirals) shall not be less than

$$p_s = 0.45 \left[\frac{A_g}{A_c} - 1\right] \frac{f'_c}{f_y} \qquad \ldots (2)$$

C5.1 The intention is the same as that stated in C4.2 for flexural members. The effect of axial load on the cracking moment is to be included.

C5.2 The design shear force is calculated using a capacity design approach to determine the maximum probable shear force using a capacity reduction factor $\phi = 1$ for the member strengths and making allowance for overstrength materials. The column shear force is equal to the sum of the column moments at the top and bottom of the column divided by the column height. The maximum probable shear force in the column will be when the top and bottom column moments are a maximum. In the limit this would be when the maximum probable ultimate moment capacities develop top and bottom in the column. However, generally a beam sidesway mechanism occurs and the top and bottom column moments to be used are the maximum probable moments there when plastic hinges form in the beams. Shear reinforcement is provided as for beams. For space frames the resultant design shear force on the columns should be calculated for the general case of seismic loading acting simultaneously in the directions of both principal axes of the frame. For that loading case, the column should be capable of carrying the resultant design shear force. For the purposes of Sections 5.2, 5.3 and 5.4, the end regions of columns shall be regarded as the regions of the columns above and below beam-column connections over a minimum length from the faces of the connection at least equal to the greatest column cross sectional dimension, 450mm (18 in) and one-sixth of the clear height of the column.

C5.3 Buckling of non-prestressed column bars is to be restrained.

C5.4 The ductility of columns reduces with increase in axial load level and with increase in longitudinal steel content. ACI 318-71[3] has been taken as a guide for the axial load level above which special transverse steel is required, a column load of $0.1P_o$ being an approximation for the $0.4P_b$ specified in ACI 318-71. For columns with steel which does not have a definite yield point, or with steel around the perimeter of the section, the definition of P_b loses its meaning and the specification of column load in terms of P_o is preferable. The a/h limit specified for beams in Section 4.1 is also used as a limit for columns because deeper compressive stress blocks than 0.2h may require special transverse steel to adequately confine the concrete. The amount of special transverse steel is but not less than $0.12f'_c/f_y$, where A_g is the gross area of the section, A_c is the area of the core measured to the outside of the spiral, f_y is the yield strength of the spiral steel, and f'_c is the concrete compressive cylinder strength.

Where rectangular hoop reinforcement is used as special transverse reinforcement the required area of hoop bar shall be computed by:

$$A_{sh} = \frac{l_h p_s s_h}{2} \qquad \ldots (3)$$

where l_h is the maximum unsupported length of rectangular hoop measured between perpendicular legs of the hoop or supplementary crossties, s_h is the hoop spacing, and p_s is given by Eq. (2) with the area of rectangular core of column to outside the hoops substituted for A_c and hoop yield strength substituted for f_y. The minimum size of such reinforcement shall be 10mm diameter (3/8 in) and the centre to centre spacing shall not exceed 100mm (4 in).

Special transverse reinforcement may also be considered to act as shear reinforcement. Longitudinal steel shall be present around the perimeter of spirals and in the corners of rectangular hoops.

that required by ACI 318-71[3] or other authoritative document. Columns with design load exceeding $0.1P_o$ or with $a/h > 0.2$ require special transverse steel to increase the available curvature ductility in the end regions in case plastic hinges form there due to circumstances unforeseen by the designer. For example, in flexible structures points of contraflexure in columns can move well away from the mid-height region and may result in plastic hinges forming at one end of the columns.

6.0 DESIGN OF BEAM-COLUMN CONNECTIONS

6.1 Anchorages for post-tensioned tendons shall not be placed within beam-column joint cores.

6.2 Transverse reinforcement around the longitudinal column bars shall be provided through the joint core for shear. The design shear force V_j shall be the maximum horizontal shear force in the joint core determined from the column shear and the shear developed from the steel and concrete forces in the beams at the formation of plastic hinges due to the combination of design lateral and gravity loadings, taking into account the probable overstrength of the materials. In the joint core of a plane frame, the area of shear reinforcement per layer shall not be less than

$$A_v = \frac{V_s}{n f_y \phi} \qquad \ldots (4)$$

where V_s is the design shear force to be carried by the shear reinforcement in the joint core (equal to V_j if no allowance is made for the shear carried by the concrete shear resisting mechanism), f_y is the yield strength of the shear reinforcement, n is the number of layers of shear reinforcement bars effectively crossing the corner to corner crack of the joint core, and ϕ is the capacity reduction factor for shear equal to 0.85. For space frames allowance shall be made for the resultant shear force acting on the joint core due to earthquake loading acting simultaneously in the directions of both principal axes of the frame. For this loading case the shear force to be carried by the shear reinforcement shall not exceed the total component of force in the shear reinforcement acting in the direction of the resultant shear force crossing the plane of the corner to corner diagonal tension crack. At least three prestressing tendons

C6.1 Anchorages are kept out of beam-column joint cores in order to avoid tensile bursting stresses in a region already subjected to severe diagonal tension from beam and column forces. At exterior joints anchorages can be placed in stubs outside the joint core region.

C6.2 The design shear force to be carried by the shear reinforcement in the joint core is $V_s = V_j - V_c$, where V_j is the maximum horizontal shear force acting across the joint core and V_c is the shear carried by the concrete shear resisting mechanisms. The Canterbury tests[7] have shown that, although in the first cycle of loading in the inelastic range, the joint core concrete shear resisting mechanism can carry considerable shear force, further inelastic loading cycles can result in a degradation of the concrete shear resisting mechanism due to a breakdown of the joint core concrete caused by alternating bond forces and diagonal tension cracking. The presence of a central prestressing tendon in the beam was shown in those tests to be effective in helping to prevent joint core shear failure, because of the better control of diagonal tension cracks. Nevertheless the presence of a central tendon did not prevent degradation in the shear carried by the concrete in all joints tests[7]. It may be possible to make some allowance for the shear carried by the concrete in cases where the joint core is well confined by a high axial compressive column load, or by high beam prestress and some central tendons or by longitudinal column steel placed around the perimeter of the column section, or by the presence of beams entering all four sides of the column. However, a conservative approach to joint core shear design should be adopted and generally V_c should be assumed to be zero unless there is evidence to show that some shear can be carried by the concrete shear resisting mechanism during seismic type loading. The should normally be present of which one is located in the mid-depth region of beams. It is desirable that longitudinal column steel be present around the perimeter of column sections. No allowance shall be made for shear carried by the concrete shear resisting mechanisms unless further research shows that the concrete shear resisting mechanism can be adequately maintained during seismic load reversals.

6.3 The transverse reinforcement provided through the joint core shall be not less than that specified in Section 5.4 regardless of the column load.

6.4 Connections between precast members at beam-column joints shall be acceptable provided that the jointing material (mortar, epoxy, or cast-in-place concrete) has sufficient strength to withstand the compressive and transverse forces to which it may be subjected. The interfaces shall be roughened to ensure good shear transfer and the retention of the jointing material after cracking.

6.5 It is desirable that ducts for post-tensioned grouted tendons through beam-column joints should be corrugated.

details of some beam-column joint designs where V_c was maintained during test loading may be seen elsewhere[7].

The shear reinforcement is placed to carry V_s across the corner to corner crack of the joint core, tests having shown that this is the critical failure crack[7]. Only the shear reinforcement within the top and bottom beam steel should be taken as being effective; that is, V_s is not to exceed the total force in the shear reinforcement between the top and bottom beam bars multiplied by the capacity reduction factor. Strain readings taken on hoops in joint cores[7] have indicated that the hoops in the central region of the core are more effective than the hoops near the top and bottom of the core. Thus the top and bottom hoops could be disregarded and the hoops concentrated more in the central region of the core. In cases where there is not a prestressing tendon in the mid-depth region of the beam, the area of shear reinforcement given by Eq. 4 may be inadequate, even when the shear carried by the concrete is ignored, and some increase in shear reinforcement may be necessary. Ideally, for reversed beam flexure and joint core shear, the tendons should be distributed down the beam section and not all placed at the section extremities or all at mid-depth. Longitudinal column bars around the perimeter of the column are also helpful in controlling diagonal tension cracks in the joint core. Hence a number of smaller diameter bars around the column perimeter is preferable to bars placed just at the corners of the section. Eq. 4 is not applicable when the joint core is subjected to biaxial shear forces due to earthquake loading acting simultaneously along both axes of a space frame. For that loading case the joint core should be capable of carrying the resultant shear force due to the biaxial shear forces induced when plastic hinges form in the beams in both principal directions of the frame simultaneously due to earthquake loading acting in a general direction.

C6.3 The minimum transverse reinforcement content permitted in the joint core is based on the requirements of ACI 318-71[3].

C6.4 Although only limited testing has been performed[8] it is considered that precast joints at the faces of columns can function effectively with no other connection through the jointing material other than grouted tendons. Some form of mechanical interlock is desirable to hold the jointing material in place. Where possible it is preferable to locate the jointing faces away from plastic hinge zones by use of cruciform columns.

C6.5 Corrugated ducts enable better bond between grout and concrete which is desirable in regions of high bond stresses.

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NOTATION

a = depth of equivalent rectangular concrete compressive stress block at flexural stength

A_c = area of concrete core measured to outside of transverse steel

A_g = gross area of concrete section

A_{sh} = area of transverse hoop bar, one leg

A_{ps} = area of prestressing steel in tension zone

A_v = area of transverse shear reinforcement per layer

b = width of section

d = depth from extreme compression fibre to centroid of flexural steel, but need not be taken as less than 0.8h

f'_c = 28 day specified concrete compressive cylinder strength

f_{ps} = stress in prestressing steel at design load

f_y = yield strength of transverse steel

h = overall depth of member

l = clear span of beam

l_h = maximum unsupported length of rectangular hoop side measured between perpendicular legs of the hoop or supplementary crossties

M_{uA} = probable ultimate positive moment capacity at one end of beam

M_{uB} = probable ultimate negative moment capacity at other end of beam

n = number of layers of shear reinforcement

P_b = axial load capacity of column at simultaneous attainment of assumed ultimate strain of concrete and yielding of tension steel

P_o = axial load capacity of column without flexure

s_h = centre to centre spacing of hoops

V_c = shear force carried by concrete shear resisting mechanism

V_{dg} = shear force due to design gravity loads acting on beam treated as a simply supported span

V_j = maximum horizontal shear force in joint core determined from column shear force and shear force developed from steel and concrete forces in beams at formation of plastic hinges due to combination of lateral loading and design gravity loading

V_s = design shear force to be carried by shear reinforcement in joint core

V_u = maximum design shear force

ϕ = capacity reduction factor

ρ_s = ratio of volume of spiral reinforcement to volume of concrete core measured to outside of spirals.