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DESIGN OF RC WALLS POST THE CANTERBURY EARTHQUAKES

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Abstract

The seismic behaviour of reinforced concrete (RC) walls has received significant attention following the Canterbury earthquakes. The poor performance of some RC walls in several modern buildings raised questions regarding the current state of RC wall design and construction practice. In response to the concerns raised, the provisions in the Concrete Structures Standard (NZS 3101:2006) have been re-examined and amendments proposed. Following the earthquakes, both the Canterbury Earthquakes Royal Commission (CERC) and the Structural Engineering Society (SESOC) recommended changes to current design practice and to standards to address the identified deficiencies. Several key considerations for RC wall design are discussed with respect to the issues raised by CERC and SESOC, and these have been addressed in Amendment 3 to NZS 3101:2006. These amendments will result in significant changes to RC wall design provisions in relation to the minimum vertical reinforcement limits, confinement reinforcement, detailing of shear reinforcement, axial load limits, coupled wall systems and the detailing of singly reinforced walls.

Introduction

A number of structural reinforced concrete (RC) walls that were examined by the Canterbury Earthquakes Royal Commission (CERC) performed poorly during the earthquakes with unexpected and/or undesirable behaviour. In some cases the poor performance appeared to be largely due to basic structural mechanics issues not being adequately addressed. To understand the reasons for the observed poor performance and to produce safe designs it is essential to have a good basic understanding of structural engineering concepts.

In response to issues identified by CERC and the Structural Engineering Society (SESOC, 2011; 2013), significant changes are being made to NZ 3101:2006 in the form of Amendment 3, which it is planned to publish in late 2015. In addition to numerous editorial changes (including terminology and clause order), important changes were made to most sections of the Standard, and particularly to Chapter 11 which covers RC walls. Some of the noteworthy changes made to other chapters included:

- Consideration of member elongation, which was the cause of significant damage in the Canterbury earthquakes
- Changes to support details for precast floor, stair units and panels
- Changes in the way that member stiffness values are calculated for seismic analyses.

Further details of the major changes proposed in Amendment 3 can be found in Cook et al. (2014).

Assessment of RC wall performance resulted in significant changes to Chapter 11 of NZS 3101:2006, based on inadequacies found in design and detailing in the following areas:

- Minimum reinforcement limits
- Singly reinforced walls
- Transverse reinforcement and detailing, including an increase in confinement reinforcement
- Global wall buckling and instability – axial load limits and slenderness
- Wall elongation
- Low aspect ratio walls
- Coupled wall systems

Background – Observed Wall Damage and Implications

Two of the RC walls that were assessed by the CERC had been designed or assessed using stiffness for analysis that were based on the assumption that extensive flexural cracking would occur over the whole height of the wall. However, relatively simple calculations, backed by observed crack patterns, indicated that flexural cracking was limited to one or two sections. The consequence was that the calculated fundamental periods were too high and the required design strength was too low. The lack of sufficient vertical reinforcement in both cases led to fracture of the reinforcement due to yield being confined to the immediate vicinity of the cracks. One of these cases, being the Gallery Apartments Building, is described in greater detail below.

Torsional resistance in a beam, a shear core or a column requires that the diagonal compression forces, which resist the torsional shear stresses at a section, are balanced by tension forces in the transverse and longitudinal reinforcement. In one of the buildings, a shear-core, which was eccentric to the centre of mass of the building in plan, provided the lateral seismic strength of the building. For this purpose the shear-core needed to sustain uncoupled flexural and torsional actions in an earthquake. When the longitudinal flexural tension reinforcement yields in tension the capacity of this reinforcement that is required for torsion is lost. This results in a loss of torsional resistance and failure can occur by rotation of the shear core leading to collapse of the building. For this building a flexural torsional failure of the shear core appears to be a likely cause of collapse (PGC building).

In walls subjected bi-axial flexure and shear there are some basic problems in assessing shear strength. Relatively simplistic analyses indicate that under this loading condition there is likely to be a significant decrease in out of plane shear strength of the member when it is subjected to high axial loading. In one multi-storey building a wall that was subjected to relatively high axial loading failed in the earthquake. Initially this was put down the lack of confinement reinforcement in the wall. However, as the wall would also have been subjected to appreciable out of plane displacement due to torsional response of the structure, it is likely that the failure was triggered by a shear failure where the shear resistance was reduced by the axial loading and the in plane flexural and shear forces.

Lack of Distributed Flexural Cracking

Assessments of buildings following the Canterbury earthquakes highlighted several examples of RC walls in multi-storey buildings that had formed a single flexural crack in the plastic hinge region as opposed to the expected larger number of distributed cracks (CERC 2012a, Sritharan et al. 2014). After breaking out the surrounding concrete it was found that the vertical reinforcing steel was often fractured due to the inelastic strain demand at the crack location. If too little vertical reinforcement is used in walls, there is insufficient tension transferred across the crack to exceed the tensile strength of the concrete surrounding the reinforcement and as a result secondary cracks cannot form. In this situation tests on bars extracted from other buildings where single major cracks had formed indicated that yielding of the reinforcement was limited to fewer than two bar diameters on each side of the crack. Consequently relatively modest crack widths of the order of a few mm can lead to strain levels that may fracture the bars. Such walls lack ductility and perform poorly during earthquakes. To obtain ductile performance a series of secondary cracks must form so that yielding can extend for some distance along the wall. The Canterbury Earthquake Royal Commission recommended that research be conducted to refine design requirements for crack control in RC walls (CERC 2012a).

As part of the CERC (2012a) a detailed investigation was conducted into the performance of the Gallery Apartments building (Smith and England 2012). The reinforced concrete walls in the Gallery Apartment building contained less vertical reinforcement than is required by the current design standard NZS 3101:2006, but it complied with the 1995 edition of the Standard. The walls were observed to have formed a small number of cracks at the wall base. As highlighted in volume 2 of the CERC report (2012a), the quantity of vertical reinforcement in the Gallery Apartment walls was insufficient to develop the tension force required to form further cracks. Additionally, samples extracted from the Gallery Apartments building following the earthquakes indicated that the concrete strength was significantly higher than the specified concrete strength (Holmes Solutions 2011). The higher concrete strength further increased the likelihood of a single crack formation and fracture of vertical reinforcing steel. Both moment-curvature and detailed finite element analysis conducted of the grid-F wall in the Gallery Apartments building confirmed the observed earthquake performance (Henry 2013, Sritharan et al. 2014). When the as-built reinforcement content and concrete strength were used, a single flexural crack was predicted to form when subjected to lateral load with premature fracture of the vertical reinforcement. It should also be noted that this analysis confirmed that the stiffness of the wall was likely to be closer to the gross section stiffness rather than the reduced cracked section stiffness used during the design.

<u>Minimum Vertical Reinforcement Limits</u>

Historically, minimum requirements for vertical reinforcement in RC walls were governed by shrinkage and temperature effects. More recently, minimum vertical reinforcement limits for RC walls have been increased in design standards worldwide to ensure that ductile behaviour is achieved when yielding of reinforcement is expected. In the 2006 revision of NZS 3101, the minimum required vertical reinforcement in RC walls was increased by over 80% with the adoption of a similar equation to that previously used for RC beams. Because of these recent changes, some of the RC walls in Christchurch that were observed to have only a few flexural cracks and fractured vertical reinforcement had vertical reinforcement contents below the current limit in NZS 3101:2006.

As discussed by Henry (2013), the current minimum vertical reinforcement limit for RC walls may not be appropriate as the margin of separation between cracking and nominal strength would be less than for an equivalent beam unless a significant axial load is applied. A series of large-scale tests were conducted and supported by additional numerical analysis to investigate the seismic behaviour of RC walls with vertical reinforcement in accordance with the current minimum limits in NZS 3101:2006 (including Amendment 2, A2) (Lu et al. 2014, 2015). Based on this research the following conclusions were drawn:

- Current NZS 3101:2006 (A2) requirements for minimum vertical reinforcement are sufficient to prevent a non-ductile response, but deformations are still concentrated at a limited number of flexural cracks and these walls are susceptible to bar buckling and fracture failure at low lateral drifts.
- Placing additional reinforcement at the ends of the wall beyond that required by NZS 3101:2006 (A2) significantly increased the distribution of cracking and ductility of the wall.

NZS 3101 A3 – Nominally Ductile

The existing requirements for minimum vertical reinforcement were retained with two minor changes. The notation was changed from a total vertical reinforcement ratio to a distributed vertical reinforcement ratio (p_l), and the previous clause that allowed the limit to be related when the nominal strength exceeded the design strength by 30% was removed. The performance of these lightly reinforced walls was also improved by refinement of requirements for singly reinforced walls and transverse ties as discussed below.

NZS 3101 A3 – Ductile and Limited Ductile

For ductile and limited ductile hinge regions additional vertical reinforcement is required to ensure the required hinge rotations can be sustained during an earthquake. As per recommendations from research (Henry, 2013; Lu et al. 2014, 2015), it was proposed to increase the reinforcement in the ends of the wall. The initial primary flexural crack in a wall is induced when the tensile stress in the concrete is exceeded. This crack will extend from the extreme tension fibre to close to the zero strain fibre, and generally the crack length will exceed half the wall length. The minimum vertical reinforcement proportion in the end regions of the wall has been determined to ensure that before the reinforcement yields a series of secondary cracks will have also formed adjacent to the primary crack. This is essential to enable yielding to spread over a length of the wall to enable ductile performance to develop. If the series of secondary of cracks cannot form, a brittle failure may occur similar to that observed in the Gallery Apartments building.

To achieve this secondary crack formation, the tension force transmitted across the cracks in the end region must exceed the tensile strength of the concrete in this region. The requirement for minimum vertical reinforcement in the end zone was determined by equating the tensile force provided by the vertical reinforcement in the end zone with the expected maximum long-term tensile strength of the concrete in the end zone to ensure that secondary cracks would form. Consideration was given to the following aspects:

- Concrete strengths being greater than specified and strength increasing with time
- Earthquake strain rates increasing the strength of both concrete and reinforcement
- Relationships between compressive and tensile strength of NZ concrete
- The maximum likely tensile strength of the concrete
- Residual tensile stress in the concrete due to internal restraint of drying shrinkage by the reinforcement

Details of the numerical factors that were used to derive the minimum reinforcement requirements are shown in Cook et al. (2014).

The end zone, requiring additional vertical reinforcement, extends $0.15L_w$ along the wall length at each end of rectangular walls, or from the ends of flanges and webs in non-rectangular walls. The end zone definitions are shown in Figure 1 for different wall geometries.

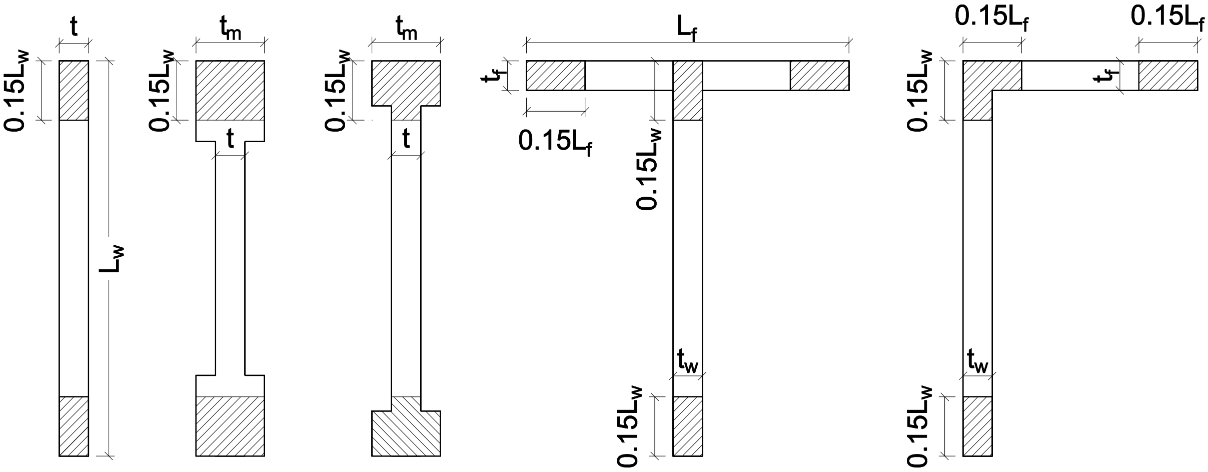


Figure 1: End zone definition for different wall geometries

If the quantity of vertical reinforcement in the end zone ($0.15L_w$) is significantly greater than the quantity of distributed vertical reinforcement in the central region, without distributed reinforcement in the central region of the wall the secondary cracks will not extend into the wall far beyond the end regions, and the wall can be susceptible to widely spaced cracks in the central region which can result in poor shear resistance and higher shear deformations (Sritharan et al 2014). Lumping reinforcement at the ends of the walls ensures the initiation of primary and secondary flexural cracks, and an additional requirement for 30% of the end reinforcement ratio in the central region of the wall (between end zones) ensures that these cracks propagate through the length of the wall. A second requirement for this central reinforcement is to ensure that the bending moment transferred across the crack is sufficient to enable other primary cracks to form higher up the wall.

Singly Reinforced Walls

In Amendment 3 to NZS 3101:2006, singly reinforced walls are no longer permitted to be designed for ductile or limited ductile regions. Whilst research and some experiences in the Canterbury Earthquake sequence demonstrated ductility of singly reinforced walls, these walls are thought to lack the robustness and ductility to sustain significant damage while retaining lateral stability, particularly when considering multi-directional actions. Included in the lack of robustness were uncertainties relating to:

- Connections and bar anchorage
- Shear and stability
- Fracture of reinforcement
- Reliance being placed upon extensive yielding to develop ductile behaviour

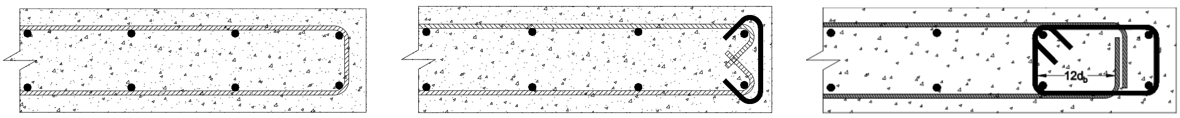
Consequently, to allow for their uncertain performance in earthquakes, singly reinforced walls which form part of the primary lateral load resisting system are to be designed for nominal ductility (μ=1.25) combined with a strength reduction factor of 0.7. The strength reduction factor of 0.7 is a result of combining the "usual" wall strength reduction factor of ϕ = 0.85 with an additional factor of 0.8. The 0.8 factor is the ratio of the flexural strength at first yield to the ultimate strength. The product of 0.85 and 0.8 gives 0.68 which was rounded to 0.7. With the reduced strength reduction factor the wall should resist the ultimate strength actions with negligible inelastic deformation and sustain only relatively minor inelastic deformation in the maximum considered earthquake.

The maximum vertical reinforcement content in singly reinforced walls is limited by the requirement that they satisfy the condition for nominal ductility for out of plane bending. For this condition the neutral axis depth must be equal to or less than 0.75 times the corresponding depth at balanced conditions (see clause 7.4.2.8), that is $c > 0.75c_b$. Hence, for a singly reinforced wall reinforced with Grade 500 reinforcement, the limiting vertical reinforcement proportions for concrete strengths of 30 MPa and 50 MPa are 1.18% and 1.6% respectively. To be consistent with other equations for walls these percentages are expressed in terms of the total width of the wall.

Anchorage of Horizontal Reinforcement

SESOC (in the Interim Design Guidance) (2013) noted that reinforcement confinement was important for the performance of doubly reinforced walls in the Canterbury Earthquakes. Amendment 3 to NZS 3101 now explicitly requires the ends of wall segments to be locally confined to provide adequate anchorage and development of horizontal reinforcement, and to transfer loads through a diagonal compression strut into the end vertical (longitudinal) bars. This can be achieved by using one of the following alternatives;

- bending the horizontal reinforcement around both of the corner end bars to form a continuous U shaped bar (Figure 2(a));
- bending the horizontal reinforcement around a corner bar at the ends of the wall with a 135º hook; with those two corner bars at the end of the wall then linked with an effective tie (Figure 2(b));
- anchoring the horizontal reinforcement as close as practical to the end of the wall with horizontal 90º or semi-circular hooks, with these hooks terminating within a horizontal closed cage which encloses at least 4 vertical bars at the end of the wall (Figure 2(c)).



(a) (b) (c)

Figure 2: Anchorage of horizontal reinforcement (dimensions exaggerated for illustrative purposes)

Similar anchoring details are required for horizontal reinforcement at the intersections in C- or T-shaped walls. Because singly reinforced walls will be effectively designed to behave elastically, no such detailing requirements are included in Amendment 3.

Local Buckling of Bars in Mid Regions of Walls

In the Interim Design Guidance, SESOC (2013) noted that buckling of vertical reinforcing steel in concrete walls with confined boundary regions was observed in several buildings in Christchurch. Bar buckling was not limited to the ends of the wall, but it also occurred along the wall length and led to lack of containment of large sections of concrete in the web region. When subjected to in-plane actions, a wall with distributed vertical reinforcement and a low axial load ratio would exhibit a strain profile at ULS such that most of the wall (and reinforcing bars) would be in tension, with a comparatively shorter length of the wall in compression. When the earthquake force reverses the bars that had previously yielded in tension would now be subjected to compression, and those bars which had sustained an appreciable level of tension strain would then have the potential to buckle in compression. Whilst those bars in the compression zone on the ULS strain profile (within a length of c, the neutral axis depth, from the ends of the wall) would be provided with anti-buckling and confinement reinforcement, there could be a portion of the middle part of the wall where the tension strain was such that the bars when subjected to compression would be susceptible to buckling. In developing the design criteria it has been assumed that bars not constrained by ties, which sustain a tensile strain in the ultimate limit state greater than $8\varepsilon_y$ are susceptible to buckling when subjected to compression. Further research on this critical strain limit is required, and consequently the following design criteria are tentative. In limited ductile walls the strain limit ($8\varepsilon_y$) is seldom reached due to the limited material strain (curvature) permitted in the plastic hinge region. Consequently for limited ductile plastic regions no additional ties are required.

For ductile plastic regions the permissible curvature limits are considerably higher, which enables appreciably greater strains than $8\varepsilon_y$ to be induced in the reinforcement in the mid region of wall. In this situation, unless it can be shown that the concrete surrounding the reinforcement is intact, so that it can provide lateral restraint to the reinforcement, containment reinforcement, which consists of ties that hold the two sides of the wall together, is required over the length of the effective plastic hinge. It is assumed that the concrete surrounding the vertical reinforcement will be intact where the three criteria given below are all satisfied. Where this is the case no containment reinforcement is required.

The three criteria are:

1. The spacing of the vertical bars is greater than $5d_b$. A close spacing of bars can result in a crack forming in the plane of the bars, which would increase the tendency for spalling.
2. The clear cover to the vertical bars is equal to or greater than $1.5d_b$. Thin cover encourages tensile splitting cracks to form along the bars and the lateral resistance that can be provided to restrain buckling is reduced.
3. The shear force acting on the wall is less than $0.075f'_c\ 0.8L_w\ t$, where L_w is the length of the wall and t is the thickness. This limit was set on the basis that;
    - the diagonal compression stresses associated with shear would be confined to half the wall thickness due to out of plane bending,
    - the shear stresses in the mid region of the wall are greater than the average shear stress.

Based on these assumptions the diagonal compression stresses are close to $0.4f'_c$, which is close to the limit that can be sustained in a region containing diagonal cracks.

Global Wall Buckling and Instability

For walls with high axial loads, ($N > 0.2\phi f'_c A_g$), the ratio of effective height to thickness ($k_e L_n/t$) has been reduced in Amendment 3 to NZS 3101 from 30 to 20 to reduce the effects of lateral flexural torsional buckling. The reason for this reduction is to simply increase the level of robustness, noting that walls may be subjected to both in-plane and out-of-plane actions during an earthquake. In a similar manner, the effective height to thickness ratio of flanges is also now 20 when the overhang length of the flange on the side of the web is of an appreciable length that buckling could be a concern (flange overhang $\geq 3t_f$), and when the flange contributes to the flexural resistance of the wall. In plastic hinges of fully ductile wall structures, this limit is reduced to 15, and again is to safeguard against out-of-plane buckling of thin and wide flanges.

Global wall buckling and instability are not simple phenomena, and for this reason the above reductions in effective height to thickness have been implemented for robustness purposes primarily. By this means allowance has been made to reduce the potential adverse effects associated with global wall buckling.

Axial Load Limitations

The axial load is another contributing factor to global buckling of RC walls. There is currently insufficient evidence and data to support a more comprehensive analysis of the effect of axial load, but it was considered that axial load limitations should be introduced. The exact calculation of axial loads on RC walls during earthquakes is complex and involves consideration of building interactions. The formation of a plastic hinge in a wall can cause significant elongation to develop. The resultant vertical displacement can be partly restrained by surrounding structural elements, potentially significantly increasing the axial load on the wall. As an interim solution, the ultimate axial load for all walls has been limited to $0.3f'_cA_g$ when considering earthquake actions. This limit excludes axial load induced by elongation. The corresponding limit for non-seismic load combinations is $0.4f'_cA_g$. The imposed actions due to earthquake loading are less certain than for gravity loading and it is impractical to calculate the additional axial loading associated with elongation which is discussed below in more detail.

Elongation

Flexural cracking in most RC members causes elongation to occur as the tensile strains are greater than the corresponding compression strains. Generally in the serviceability limit state the effects of elongation are minor. However, when plastic hinges form, the magnitude of elongation increases, and significant interactions can occur between structural elements. A lot of structural damage in the Canterbury Earthquakes was caused by elongation. A major difficulty in dealing with the adverse effects of elongation arises from:

- Most currently used methods of structural analysis do not include elongation
- Much of our design criteria for structural elements has been developed from structural tests and analyses of individual structural elements in which the interactions with other structural elements have not been considered.

Elongation of coupling beams pushes the coupled walls apart. This relative movement will be partly restrained by any floor slabs that are connected to the walls and by the stiffness of the foundation beam. The tension force resisted by the slabs is balanced by a compression force in the coupling beams, which can significantly increase their shear strengths. This action prevented the coupled shear walls in the CTV building behaving in the intended ductile manner (CERC, 2012b). The enhanced strength of the coupling beams, due to the restraint provided by the floor slabs, increased the strength of the beams to the extent that inelastic deformation was induced in the walls causing a premature limited ductile failure. Clauses detailing how strength enhancement of coupling can be assessed have been added to NZS 3101:2006 in Amendment 3, together with design criteria to ensure inelastic deformation is confined to the chosen locations that have been detailed to sustain inelastic deformation in the coupled walls.

When a plastic hinge forms at the base of a wall, or other location, elongation occurs. The increase in height of the wall can be partly restrained by forces that are transferred to columns or other walls by the flexural, torsional and tensile membrane stiffness and strengths of the floors and beams. This action can increase the axial load on the wall and induce tension in columns and other walls in the structure. Elongation can have a major effect on the seismic performance of a building and it is difficult to assess in design (Wight, 1985). As it is impractical to expect designers to determine the potential increase in axial load due to elongation, the limiting design axial load has been reduced and the confinement requirements for the flexural compression zone of walls have been increased. By this means allowance has been made to reduce the potential adverse effects associated with this action, though there may still be potential problems in the foundations.

Squat Shear Walls

In structural walls, where the aspect ratio (M^*/V^* or h_w/L_w) is low, such as can occur in podium type structures where high shears are transferred from the tower to the perimeter walls, the horizontal shear reinforcement becomes less effective than vertical reinforcement for shear resistance. In such walls, the vertical reinforcement provided to resist flexure may be insufficient to maintain shear resistance. Additionally, the flexural design principle of plane sections remaining plane breaks down, and a strut-and-tie analysis may be a more suitable design method.

With a strut-and-tie approach the shear stress resisted by concrete, v_c, should be taken as zero, and the strut angles of the compression forces to the horizontal plane, which are used to resist shear and flexure, must equal or exceed 30°. The limit of 25° given in Appendix A of NZS 3101:2006 is increased to 30° to allow for the inelastic cyclic loading that occurs in earthquakes.

An alternative to the strut-and-tie method of analysis and design is to design for flexure but provide a minimum amount of distributed vertical reinforcement (p_{lm}) to ensure that the shear forces can be adequately resisted. The reinforcement proportion p_{lm} corresponds to a minimum area of distributed reinforcement for shear. If the distributed reinforcement provided for flexure exceeds this value then no additional vertical reinforcement would be required for shear.

In NZS 3101:2006 Amendment 3 a new clause has been introduced so that where the M^*/V^* ratio is less than 0.75, or where the h_w/L_w aspect ratio is equal to or less than 0.5, the minimum proportion of vertical reinforcement, p_{lm}, for shear resistance, at any location along the length of the wall is either based on the strut-and-tie method, with the stipulations given above, or determined by the following equation:

$$p_{lm} \geq \frac{v_s}{f_y} \qquad \text{Equation 1}$$

In Equation 1, v_s is the shear stress resisted by horizontal web reinforcement, which is given by:

$$v_s = \frac{V_s}{0.8 L_w t} \qquad \text{Equation 2}$$

and the minimum proportion of vertical reinforcement at any section along the wall is given by:

$$p_{lm} = \frac{A_{sb}}{s_h t} \qquad \text{Equation 3}$$

where, A_{sb} is the area of vertical reinforcement within a spacing of s_h and t is the width of the wall at the location being considered.

The above Equation 1 effectively ensures that in those walls with low aspect ratios, as described above, the amount of vertical shear reinforcement is equal to or greater than the amount of horizontal shear reinforcement. This new approach in Amendment 3 for Chapter 11 of NZS 3101 for shear reinforcement in low aspect ratio walls is similar to the approach given in ACI 318-14.

Coupled Wall Systems

Several different methods of modelling coupling beams in coupled shear walls have been proposed. Santhakumar (1974) initially developed an equivalent stiffness based on a truss mechanism and Paulay (1981) developed equations in which the shear deformation that developed in coupling beams was allowed for by modifying the flexural stiffness of the coupling beams. This approach was included in the Paulay and Priestley (1992) textbook. However, neither the book nor the report (1981), or the references in either text, gave the basis of the approach. In a later paper Paulay (2003) further developed the approach suggested by Santhakumar (1974) in which the effective stiffness of diagonally reinforced coupling beams was found from calculated deformation of the diagonal reinforcement Paulay (2003). While this approach is theoretically attractive there are difficulties with it in that the stiffness at first yield is calculated assuming that there is appreciable yield penetration of the diagonal reinforcement into the walls, which is unlikely at first yield, and that tension stiffening of concrete surrounding the diagonal bars is neglected. In terms of practical use, this approach has the added though minor disadvantage of requiring the quantity of diagonal reinforcement to be known before the beam stiffness can be determined. When checking the two approaches against each other it was found that there were major differences in the predicted stiffness values of identical beams. In Amendment 3 for NZS 3101:2006 it was decided to base stiffness of coupling beams on the approach given in the Paulay and Priestley book (1992). However, it should be noted that this method does not allow for the change in stiffness associated with different proportions of diagonal reinforcement.

In the Paulay and Priestley book (1992) the effective stiffness, I_e, of diagonally reinforced coupling beams is given by the following equation:

$$I_e = 0.4\, I_g / \left(1 + 3\left(h_b / l_n\right)^3\right) \qquad \text{Equation 4}$$

where I_g is the gross section stiffness of the coupling beam, h_b is the overall beam depth and l_n is the clear span. The $0.4I_g$ is a basic stiffness for rectangular beams used at the time when Grade 300 reinforcement was used. To enable the approach to be used with different reinforcement grades, and in cases other than coupling beams with equal moment capacities at each end, the corresponding values in the proposed Amendment is given as a basic stiffness for rectangular beams (see Table C6.5 in NZS 3101) multiplied by a coefficient given for different M/Vh_b values in a table. The resultant stiffness values are essentially identical to those referenced in the Paulay and Priestley book (1992) except for the basic flexural stiffness of rectangular beams being $0.33I_g$ for beams with Grade 500 reinforcement and $0.4I_g$ for Grade 300 reinforcement.

For coupling beams with conventional reinforcement, the corresponding stiffness values are taken as half the values used for beams with diagonal reinforcement. The shear stress limit, v, for conventionally reinforced coupling beams given in (Paulay and Priestley, 1992) is equal to:

$$v \le 0.1\left(l_n / h_b\right)\sqrt{f_c'} \qquad \text{Equation 5}$$

This limit was introduced to ensure that excessive plastic rotations were not induced in the beam. However, as the section rotation limits are directly controlled elsewhere in the Standard this limit is not used. The requirement that diagonal reinforcement is required where the shear stress exceeds $0.25\sqrt{f_c'}$ given in Chapter 9 of NZS 3101 still applies.

Conclusions

This paper has highlighted some of the current challenges for RC wall design resulting from the Canterbury earthquakes, and the significant changes that have been incorporated into NZS 3101:2006 through Amendment 3. Whilst most of the changes are credibly supported by mature research, several changes have been incorporated to provide an increased level of robustness to RC wall structures, and allowance has been made to reduce the potential adverse effects associated with actions such as elongation and bi-axial bending. Further research may be required to refine these elements.

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