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Seismic lateral stability design to AS Codes

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Abstract

Amendments to the wall design clauses of the AS 3600-20181 Code have resulted in considerable commercial implications. Applying the capacity design principles to safeguard elements against the deemed to comply force based design and drawing parallels from the Christchurch Earthquake appear to be the prime reasons. Comparisons and investigations were carried out using the ACI 318-2019 Code, Australian & international publications, and case studies in this submission. In some cases, the AS Code deemed to comply provisions were found to not necessarily provide satisfactory design. Some of the key areas of the AS 3600 Code such as structural ductility, boundary elements, shear protection and minimum tensile wall reinforcement, are investigated in this paper. This paper raises a number of points for discussion regarding AS 3600-2018 and makes preliminary recommendations, based on limited analysis and scenarios, to serve for discussions and potential additional research to improve the AS Codes. Some other seismic lateral design aspects are to be tabled in future.

Keywords: limited ductile walls, boundary elements, shear protection, tensile reinforcement.

1 Introduction

The amendments to the Australian Standards Concrete Structures Code AS 3600-2018 posed varied understanding among the design engineers. Although the commentary helped to clear several of these, still a few areas are requiring attention. The lateral stability design in Australia utilises shear and core walls. Reinforced concrete façade, internal, and retention walls are also generally present in most buildings. As such the wall design can have significant financial bearing on the projects. The critical seismic lateral design aspects, especially relating to limited ductile structural walls, are investigated in the future.

It should be noted that the AS 3600-2018 Code was released a year earlier than the ACI 318-2019, and may have not had the opportunity to make meaningful comparisons by the Code Committee. Some of the amendments to exclude singly reinforced thin walls and the use of Class-L reinforcement are justified following extensive research and experiments by Menegon et al (2017) and observations in real earthquakes. Hence the adoption of these restrictions should not raise any qualms.

The deemed to comply provisions of the Australian Standards Earthquake Loading Code AS 1170.4 are based on seismic Force Based Design (FBD). Determining the appropriate Ductility (□) and Structural Performance (S_p) factors at the beginning is key to the design. Most structural systems in practice are of mixed nature and difficult to be determined. Section 14.6

of the AS 3600 Code relating to the Limited Ductile Structural Walls (LDSW) appears to be too onerous with respect to the local low seismicity. This is discussed in Section 2 of this paper.

Some aspects of the current confinement requirements require clarification. Although high strength concrete has lower ductility, when appropriately reinforced, and confined it is known to behave satisfactorily moderate axial loads. It appears that the AS Code appears to be too conservative compared to the ACI Code for the LDSW as discussed in Section 3. Moment-shear interaction is included in the AS 3600 Code to protect brittle shear failure in critical areas of the walls and the supporting foundations. The design shear is to account for proportionate increase in flexural over-strength and dynamic amplification due to higher mode effects. The prescribed clause for the LDSW appears again too conservative as discussed in Section 4.

The minimum tensile reinforcement requirements in the critical areas of the shear and core walls and their curtailment up the height of the building also appear to be too conservative, especially for the LDSW, as it demands higher reinforcement than the ACI Code and other publications intended cover more frequent occurrences. The AS 3600 Code require additional verifications and/or experiments in this regard as discussed in Section 5.

2 Ductility

It is intuitive to contemplate that the top lateral displacements of a building using the prescribed FBD in the Code may correlate to the ductility level of the structure. The appropriate force reductions corresponding to the ductility levels become sensible in this case with application of realistic cracked section properties for the lateral analysis. El-Sokkary et al (2018) refer to the Canadian Standards CSA 23.3-14 Code, which gives axial and flexural modification factors of 0.65 & 0.5 for walls for the Australian equivalent ductility factors of μ = 2 & 3 respectively and 0.2 for the slabs. These appear reasonable for the moderate seismicity of Montreal. About 20% stiffer values are appropriate for the Australian designs. The property modifiers in the AS 3600 Code are only for flexure and hence is inadequate for finite element based 3D analysis whereas those in the AS 1170.4 Commentary includes mostly suitable shear & axial modifiers.

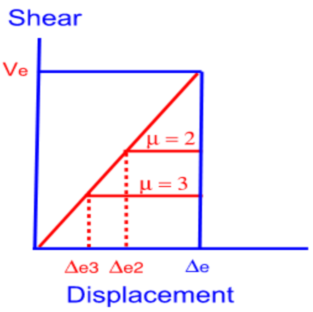


Figure 1. Ductility and seismic force reduction

The inelastic base rotation demand of the wall is given as (1) in El-Sokkary et al;

$$\theta_{id} = [\Delta_{fi}(\mu - 1)/S_p]/(h_w - l_w/2) \text{ with limits of } 0.003 \ \&\ 0.004 \text{ for } \mu = 2 \ \&\ 3 \qquad (1)$$

V_e and Δ_e *are elastic shear and displacement.* (h_w – l_w/2) can be approximated to 0.8h_w for typical 8 level buildings. This gives, => $\theta_{id} = (\Delta_{fi}/h_w)[(1 - 1/\mu)/0.8]$, where in-elastic top displacement $\Delta_{fi} = \Delta_f(\mu/S_p)$ => Δ_{fi}/h_w = 1/200 for μ = 2 & 3 for the above θ_{id} limits. The rotation capacity is $\theta_{ic} = [(\varepsilon_{cu}.l_w)/(2c) - 0.002]$, with upper limit of 0.025. Assuming the neutral axis depth $c = l_w/6$ and ε_{cu} = 0.003 conservatively => θ_{ic} = 0.007 to satisfy the demands as above. Hence, both ductility levels result in the same minimum in-elastic top displacement ratio of 1/200 and complies with the equal displacement principle illustrated in Figure 1. The load reduction (μ/S_p) ratio between μ = 3 & 2 is (3/0.67)/(2/0.77) = 1.7 and the stiffness modifier proportion is 0.65/0.5

= 1.3. Hence their elastic displacement ratio is Δ_{e3}/Δ_{e2} = 1.3/1.7 = 0.75, which is satisfactory for sufficiently stiff structures.

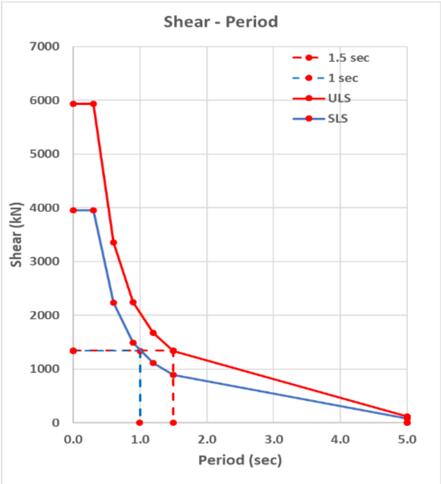


Figure 2. Extra flexible seismic force design

Excess flexibility with inadequate lateral structure or by assuming lower than realistic stiffness to increase the fundamental period of the building and hence reduce the seismic actions in the FBD are not appropriate. For example, a structure designed for a ULS base shear of 1350kN with an over-estimated period of 1.5s may reach this base shear at a hypothetical SLS event if the realistic period is 1.0s as illustrated in Figure 2. The static design method in AS 1170.4 Code requires the design base shear using a fundamental period from a software to be not less than 70% from the empirical formula in the Code. This provides a safety net to the static analysis. However, this is not the case in the dynamic analysis as there is no scaling to the static base shear in the current Code.

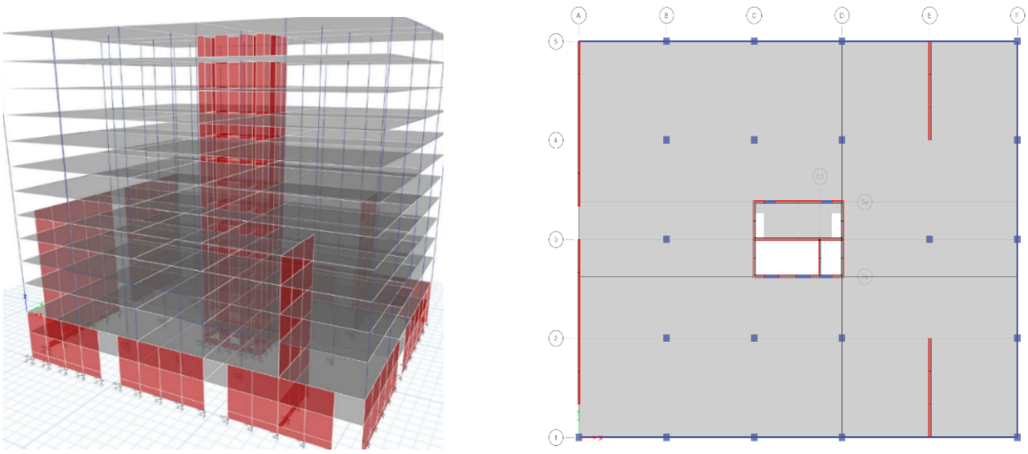


Figure 3. Case study model – 3D view and typical floor plan

One of a few case studies between systems with (a) insitu and (b) precast, core and shear walls with Class N reinforcement to both faces in a building as illustrated in Figure 3. A residential building with ten above ground levels and two basements is used for the lateral stability design against earthquake. The lateral structure comprises of 250 thick Core, 250 thick Shear Walls and 200 thick Precast Boundary Wall. The typical floors are 200 thick PT flat plate with 250 thick PT for the roof plant and ground floors with 600x600 columns on 8mx8m grids. Etabs Ultimate version software was used to carry out to satisfy firstly the FBD deemed to comply provisions of the Code and secondly the Capacity Spectrum Method (CSM) using non-linear static pushover analysis as described in the AS1170.4 Code Commentary 2021 with Importance Level = 2, probability factor k_p = 1, hazard factor z = 0.09 and Site Class = C_e. The precast option assumed three standard stitch plates per level at the vertical connections.

Gravity (G + 0.3Q) and the Code based earthquake imposed (G + 0.3Q + E*) axial stresses on the critical sections of the core and shear walls are about 0.15Fc' and 0.25Fc" respectively. The FBD Code design with ductility factor □ = 2 resulted in similar reinforcement and confinement to the lateral structure for both the insitu and precast options. However, the CSM clearly identified the lack of ductility in the precast option due to the failure of the stitch plates. The insitu option demonstrated ductility factor □ >> 2 with 0.5% vertical & 0.25% horizontal reinforcement and no confinement to the critical areas. These are illustrated in Figure 4. Even the precast option resulted in □ > 2 with improved connections

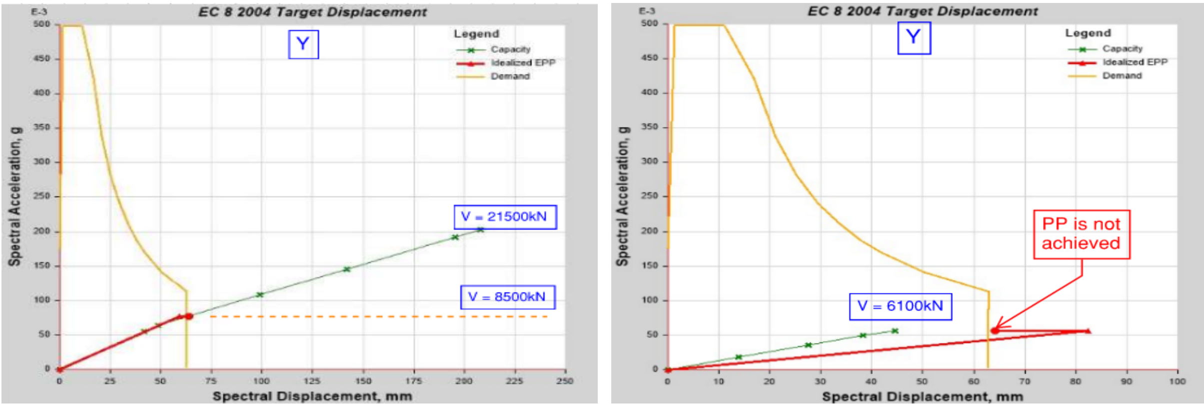


Figure 4. Push-over curves for insitu (LHS) and precast (RHS) walls

3 Boundary elements

Boundary elements are important in the seismic design of walls. Confinement ties may need to be provided to prevent the vertical wall reinforcement from buckling so that plastic hinges can form in these critical areas. However, the ductile behaviour is significantly increased as seen in the CSM Case Studies described in Section 2 when the axial and the extreme open end fibre stresses are not excessive and its slenderness is limited. The ACI Code now includes top storey displacement and inter-storey drift ratios in their boundary element evaluation and detailing for Special Structural Walls (SSW), Australian equivalent of Moderately Ductile Structural Walls (MDSW). However, this appears to be satisfied by the top displacement check described in Section 3 when choosing ductility levels.

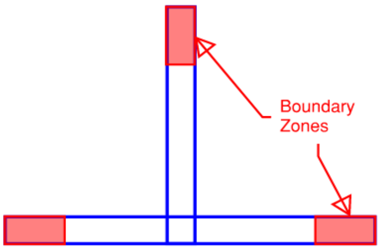


Figure 5. Typical boundary elements in special structural walls

Menegon et al (2019) found that the walls detailed with horizontal U bars at the ends will be sufficient without any additional confinement for LDSW in their preliminary investigation. Section 18.10 of the ACI 318-19 Code defines boundary elements to the SSW and typical zones reproduced in Figure 5. Hence these stipulations are too conservative for LDSW. The study in Section 2 of this paper also indicated that the L and T corner stress limits could also be increased due to the restraint from the return walls.

The lack of ductility in high strength concrete appears to be over-stated for seismic design in low-risk countries like Australia, where seismic design is critical typically for low-rise buildings and occasionally for mid-rise buildings in poor geotechnical conditions. Use of high strength concrete in such situations is unlikely and the FBD stresses due to seismic actions are low in taller buildings. Use of 150MPa concrete is not unusual even in high seismic regions where ductile performance can be achieved with appropriate detailing as per Deng et al.

The axial and extreme fibre stress ratios from a study with 3000mm long 200 and 300mm 50, 65 & 80MPa grade walls with two-way slenderness of 20, axial load of $0.5\phi N_u$, bending moments of $0.3\phi M_u$ & $0.6\phi M_u$ are summarized in Table 1 where H = height, L = length, T = thickness, F_c' = concrete strength, M* & N* are ultimate moment & axial force, A = area and Z = section modulus of the wall. From this, it appears that for the LDSW with horizontal U bars at ends is sufficient without any additional confinement. axial and extreme fibre stress limits for the 50MPa grade to be limited to $0.2F_c$'and $0.3F_c$' respectively. The regions of the walls where the above stress limits are exceeded to be detailed as columns with general confinement in accordance with Section 10 of the AS 3600 Code. The axial stress to be limited to $0.15F_c$', and slenderness limited to 16 for the MDSW. The regions exceeding $0.2F_c$' stress are to be detailed as boundary elements with general confinement and those regions exceeding $0.3F_c$' stress at the open ends with special confinement to Section 10 of the Code. The L and T corner stress limits to be increased by up to 35% and the above stress limits for the 50MPa grade to be reduced at 10% per higher grade.

Table 1. Axial and extreme fibre stress

H	L	T	H/T	Fc'	M* = 0.3ϕMu	M* = 0.6ϕMu	N* = 0.5ϕNu	Axial Stress Ratio	Extreme Stress Ratio
mm	mm	mm		MPa	kNm	kNm	kN	(N*/A)/(0.2Fc')	(N*/A + M*/Z)/(0.3Fc')
4000	3000	200	20	50	2000		7500	1.25	1.28
4000	3000	200	20	50		4000	6500	1.08	1.61
4000	3000	200	20	65	2400		9000	1.15	1.18
4000	3000	200	20	65		4800	8000	1.03	1.50
4000	3000	200	20	80	2700		10500	1.09	1.10
4000	3000	200	20	80		5400	9500	0.99	1.41
6000	3000	300	20	50	3000		11000	1.22	1.26
6000	3000	300	20	50		6000	10000	1.11	1.63
6000	3000	300	20	65	3500		13500	1.15	1.17
6000	3000	300	20	65		7000	12000	1.03	1.48
6000	3000	300	20	80	4100		16000	1.11	1.12
6000	3000	300	20	80		8200	14500	1.01	1.43

One of the misinterpreted clauses in the AS 3600-2018 Code is highlighted below, where several designers detail the entire long walls with confinement ties because Cl.14.6.3 states "throughout". It should be noted that this is not required in the ACI and NZ practice even for the Australian equivalent MDSW. The AS 3600 Commentary is also not clear on this.

> **14.6.3 Confinement of the wall core**
>
> For structural walls where $f'_c > 50$ MPa confinement of the wall core shall be provided throughout by fitments in accordance with Clause 14.5.4.

4 Shear protection

Lateral loads in short shear walls are transferred by strut-tie action and hence there is no need for ductility. However, taller shear and core walls resist lateral loads by cantilever or propped-cantilever actions and hence are subject to high bending moments and shear forces at the same locations. Plastic hinges are to be formed at these critical areas and as such shear failure should be protected. In the ACI 318 Code Section 18.10 the amplified design shear force V_d for SSW (Australian MDSW), where Ω is the flexural over-strength factor (M_u/M*), ω is the dynamic amplification due to higher mode effects and V* is the critical inelastic reduced shear force. Ω and ω in both the ACI Code and Priestley et al are about 1.6 for less than ten levels high buildings and hence correlate to those in the AS 3600 Code. The shear and moment distribution in the ACI Code and the shear amplification to ACI and AS Codes are summarised in Figure 6.

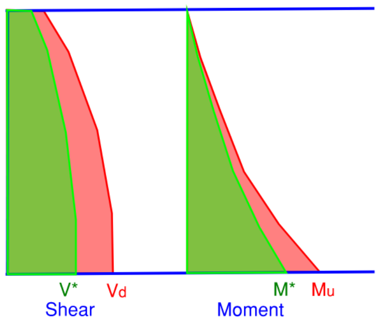


Figure 6. Flexural over-strength and dynamic amplification

The amplified shear to the AS and ACI Codes are given in (2), where S_p is the structural performance factor.

$$\boxed{\phi V_u \geq \min\left[\left(\frac{1.6M_u}{M^*}\right)V^*;\left(\frac{\mu}{S_p}\right)V^*\right]} \qquad \boxed{V_d = \Omega\ \omega\ V^* \leq 3V^*} \qquad (2)$$

The AS 3600 Commentary states that the factor 1.6 is to account for the characteristic material strengths instead of the average values and allowance for strain hardening instead of the dynamic amplification as discussed above. However, such effects should be included in the M_u/M^* ratio as in the ACI Code. The Commentary also describes how M_u is to be determined as illustrated in Figure 7. If $M^* < 0.6\phi M_u$ and setting $\phi = 1$ conservatively as in the ACI Code, then $M_u/M^* = 1.6$ should satisfy the intention of the Code.

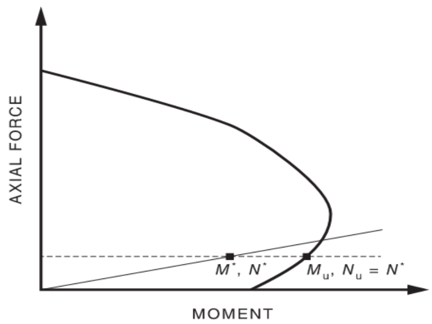


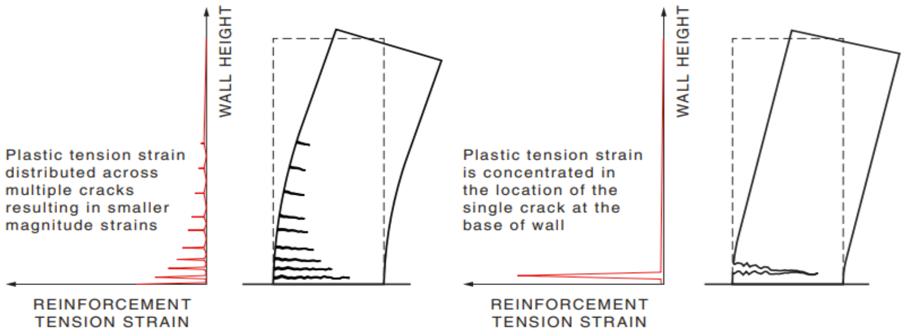
Figure 7. Over-strength factor - AS 3600 Commentary

Menegon (2022) obtained the over-strength factor of 1.4 from the section's moment-curvature for an example case as illustrated during a Concrete Institute of Australia Session. However, it appears acceptable to consider the above as an indicative mean of Ω = 1/0.77 = 1.3 for LDSW and Ω = 1/0.67 = 1.5 for MDSW. Also, 1.6 factor for dynamic amplification is too high for Australian LDSW. Investigation of Priestley et al (2007) included comparison of the CSM and time history comparison. This indicates the dynamic amplification factor ω < 1.4 for the structural fundamental natural period T < 1s and ductility factor □ = 2 are appropriate. Hence it is reasonable to adopt ω = 1.3 for LDSW and ω = 1.6 for MDSW.

The CSM Case Study in Section 2 demonstrated that the ductility demand in plastic hinge region for □ = 2 is quite easily met and the AS Code specified shear protection is not warranted for the LDSW. It should be noted that both the US and NZ requirements are intended for higher ductility (Australian equivalent of □□= 3) and seismic conditions. Hence for □ = 2, the design shear force V_d = (1.3*1.3)V* = 1.69(V_e/2.6) = 0.65V_e, which is 35% less than the elastic shear force V_e as in the AS 3600 Code. Also for □ = 3, V_d = (1.5*1.6/4.5)V_e = 0.53V_e. These design shears are appropriate for the critical regions of the walls and the foundations. Foundations such as piles where potential plastic hinges could form are generally flexible enough not to warrant full elastic shear resistance. Piles and caps may benefit from additional confinement detailing when supporting MDSW.

5 Minimum tensile reinforcement

Minimum tensile reinforcement in the critical region of the wall is essential in developing distributed cracking and to form plasticity as illustrated in Figure 8, which is reproduced from the AS 3600 Commentary. Excessive premature yielding of the reinforcement will not only lead to brittle tensile failure but also compressive failure as the axial capacity of the concrete reduces and the energy dissipation in the hysteresis loop narrows. The minimum vertical reinforcement ratio is given in the Commentary as $p_w > f_t/f_{sy}$. However, it is not clear whether f_t is the direct tensile strength ($f_{ct} = 0.36\sqrt{f_c'}$) or the flexural tensile strength ($f_{ct}.f' = 0.6\sqrt{f_c'}$) of concrete and f_{sy} is the reinforcement yield strength. The methods available are discussed here.



(a) Desirable distributed cracking (b) Undesirable sparse cracking

Figure 8. Strain distribution in wall – AS 3600 Commentary

Section 18.10 of ACI 318-2019 Code recommends $(0.5\sqrt{f_c'})/f_{sy}$ as the minimum reinforcement ratio to the critical regions of the SSW and so does Lu & Henry for Ductile Walls, both for Australian equivalent of □□≥ 3 in regions including higher seismicity. The AS 3600 requirement appears to have stemmed from the research of Hoult (2017) utilising the flexural tensile strength of concrete. His working for a 300mm thick (t_w) wall as (3) include 1.08 factor for ultimate to yield reinforcement strength ratio and 1.1 factor for increased tensile strength due to dynamic loading, 1.32 factor to account for expected to characteristic concrete strengths and 1.4 factor for ageing. Note that without the 1.4 factor this would match the overseas ones.

$$\boxed{\rho_{wv.\min} = \frac{(t_w - n_t d_{bt}) f_{ct.fl}}{f_u t_w} \qquad \rho_{wv} = \frac{(300-(2\times 16))(1.4\times 0.6\sqrt{1.32 f_c'})}{(1.1)(1.08 f_y)(300)} = \frac{0.7\sqrt{f_c'}}{f_y}} \quad (3)$$

Menegon et al (2018) approach was as in the AS 3600 Commentary, utilising the direct tensile strength of concrete with a factor of 1.8 for lower to upper characteristic tensile strength as in (4). This will match the overseas requirements if a factor of 1.4 for average values is adopted.

$$\boxed{f_t = 1.8 \times 0.36\sqrt{f_c'}} \quad (4)$$

Both the Australian versions are conservative for the less demanding LDSW in lower seismicity. It appears investigation of the actual curvature behaviour is more appropriate as the walls are not likely to conform to the Bernoulli's principle of plane sections remain plane. Rather it is closer to the deep column/beam behaviour. A simpler alternative may be to match the strain energy in the concrete and reinforcement prior and post cracking in lieu of the tensile forces. By limiting the neutral axis depth to the confined region of $0.1l_w$ as in the strain diagram of Figure 9a and assuming $\varepsilon_{cu} = 0.003$, the maximum steel strain $\varepsilon_{sr} = \varepsilon_{cu}*(0.9l_w/0.1l_w) = 0.027 < 0.05$ for the Class N 500 steel => $\varepsilon_{sr}/\varepsilon_{sy} = 0.027/0.0025 = 10.8$, adopt 10. ε_{cu} is the ultimate concrete strain and ε_{sy} is the steel yield strain.

Figure 9b illustrates the stress-strain diagrams in concrete and reinforcement in unit volume of the critical tensile zone of the wall before (SE_1) and after (SE_2) the crack formation, assuming elastic-plastic behaviour.

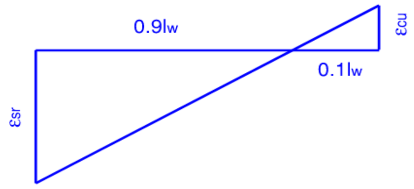


Figure 9a. Strain Diagram of the Wall at Limiting Tensile Strain

$SE_1 = (0.6\sqrt{f_c'})^2/(2E_c) + (p_w f_{sy})^2/(2E_s)$
$SE_2 = (10 - 1)(p_w f_{sy})^2/E_s$
Ec and and Es are elatic modulus of concrete and steel.
With $SE_1 = SE_2$, $p_w = (0.6\sqrt{f_c'})[E_s/(19E_c]^{1/2}/(f_{sy})$, and reduces to 50MPa concrete as (5), which is only 50% of the current AS Code requirement.

$$p_w = (0.35\sqrt{f_c'})/(f_{sy}) \qquad (5)$$

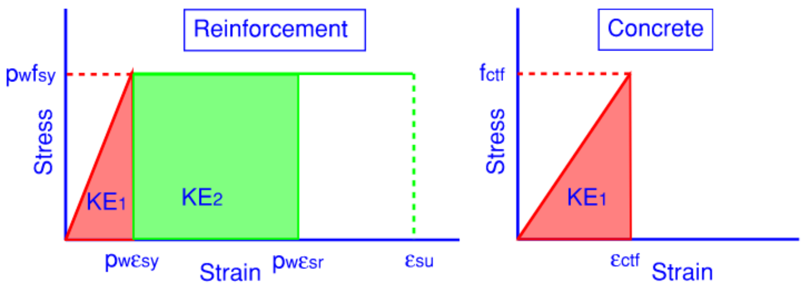


Figure 9b. Stress-strain in the critical tensile zone of the wall

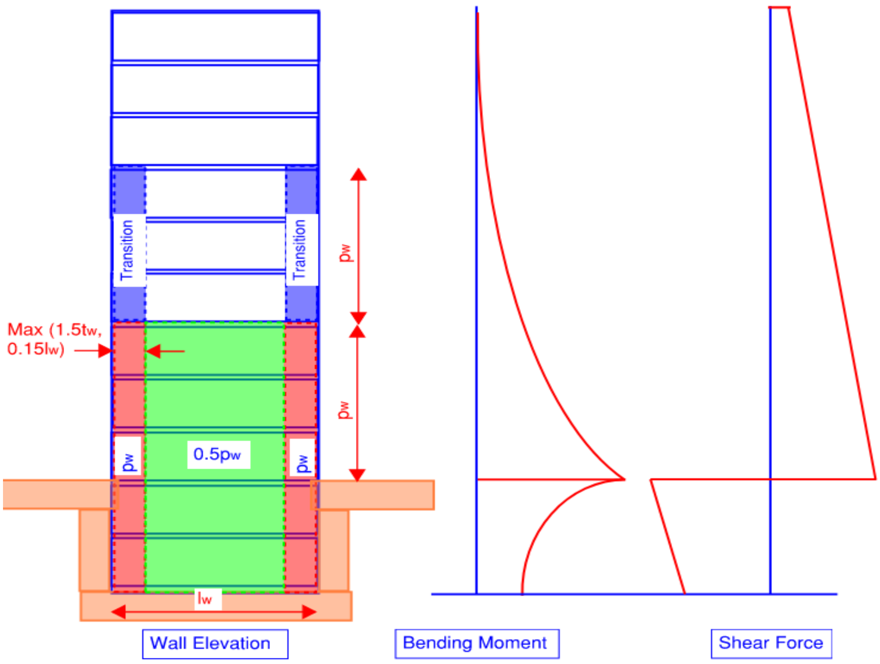


Figure 10. Minimum reinforcing, bending moment and shear diagram

The structure in the case studies of Section 2 exhibited good ductile behaviour with the above recommended reinforcement. Hence, it is adequate to provide the minimum reinforcement of $p_w = (0.35\sqrt{f_c'})/(f_{sy})$ within the critical tensile zone of maximum $(1.5t_w, 0.15l_w)$ at the open ends and $0.5p_w$ in between within the zone extending l_w (instead of the $2l_w$ in the AS Code) above and below the critical design section for the LDSW as illustrated in Figure 10. The reinforcement could be reduced by 35% in the L and T corners. A further height of l_w shall be the transition zone. Minimum reinforcement of pw = (0.5√fc')/(fsy) is appropriate for the MDSW as the ACI and other international guidelines.

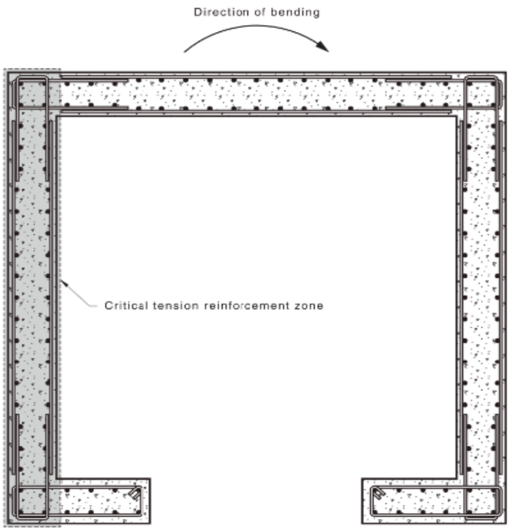


Figure 11. Critical tension reinforcement zone in a core AS 3600-2018

Figure 14.6.7(C) of AS 3600-2018 reproduced in Figure 11 is often interpreted as requiring minimum reinforcement to the entire wall length of a core in all situations. However, it should be only applicable when there is net tension in the entire wall length under a given design load combination when analysed with the shear protection amplification discussed in Section 4. The AS 1170.4 2021 Commentary recommends that the minimum requirement shall be adhered to the CSM design, even after the demand curve from the AS 1170.4 Code is increased by 50%. This appears conservative and it will be an incentive for the designers to use CSM if the reinforcing is reduced as suggested above.

6 Conclusion

Thin and slender walls and singly or Class L reinforced walls shall be excluded in the lateral load resisting structures as stated in the AS 3600-2018 Code due to its brittle nature. Precast construction with stitch plate connections within critical zones or where congested brittle areas at the dowels shall also be avoided. CSM design is found to be insightful in the case studies. However, in addition to the recent criticisms, torsion and lack of dominant fundamental mode will limit its application and hence FBD is still useful design method. Appropriate ductility assumption (top displacement-based as in the Canadian Code or similar) and cracked section properties are essential in the FBD deemed to comply design to AS 1170.4-2007. It is useful to set a minimum proportion of the base shear from the static analysis to be adopted in the dynamic analysis to prevent too flexible structures. It is evident that the AS 3600-2018 Code is too conservative in its requirements for the LDSW.

Confinement to the boundary elements as in the AS 3600-2018 is not required other than horizontal U bars at the open ends for the LDSW when slenderness, axial and the extreme fibre stresses at the open ends of the walls are limited. The limiting stresses of 50MPa concrete grade to be reduced for higher grades and increased at the L and T corners. MDSW are to be designed with less slenderness and stress limits than the LDSW but would require boundary elements nevertheless. Regions where the stresses exceed the limits to be designed with general and special confinement to LDSW and MDSW respectively. Shear Protection to the critical regions of the walls and their supporting foundation as in the AS 3600-2018 Code is also conservative. The Flexural over-strength due to expected material properties over the characteristic values and the dynamic amplification due to the higher mode effects can be reduced to 1.3 for LDSW and 1.6 for MDSW.

The minimum tensile vertical reinforcement requirement in the AS 3600-2018 Code is higher than those for more ductile demand in higher seismic regions. This can be reduced to about 50% of that stipulated by AS 3600 Code for the LDSW at the open ends with further reduction by 35% at the L and T corners. A further height of l_w shall be the transition reinforcement zone. Minimum reinforcement of 50% more than the LDSW is appropriate for the MDSW, which is similar to that in the ACI and other international guidelines. 25% less reinforcement requirements may be applicable to the CSM based design.

Other aspects such as link beams, diaphragms, torsion, drifts, secondary/retention walls and vulnerability are to be included in future. This paper raises a number of points for discussion regarding AS 3600-2018 and makes preliminary recommendations, based on limited analysis and scenarios, to serve for discussions and potential additional research to improve the AS Codes.

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