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Seismic Lateral Stability Design to AS Codes – Further Considerations

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Abstract

The paper on this topic by the author for the 2023 AEES conference discussed some of the aspects requiring attention in the AS 3600-2018 Code deemed to comply provisions relating to the seismic lateral stability design, especially for the limited ductile structural walls. Useful additional analysis and design considerations, which are generally ignored by the designers, were included in the AS 1170.4 Commentary of 2021 and somewhat addressed the said issues. Few key aspects relating to the important aspects in design practice such as link beams, diaphragms, torsion check, drifts, secondary walls, precast construction, retention walls, shear amplification and high-strength concrete are investigated in this submission.

Keywords: link beams; diaphragms; torsion check; drifts; secondary walls; precast walls; retention walls, shear amplification, high-strength concrete.

1 Introduction

As found in the 2023 AEES Conference Paper, the Australian Concrete Structures AS 3600-2018 Code seismic design requires further attention, especially for the Limited Ductile Structural Walls (LDSW) when pitched against the ACI/NZ Codes, international publications and in-house case studies.

The AS Code provisions do not have adequate guidelines with respect to the design of link beams. Hence due care shall be exercised by the designers in the interim as discussed in section 2. Diaphragm design requirements appear conservative in the AS 3600-2018 Code as discussed in section 3. The Australian Standards Earthquake Loading Code AS 1170.4 Commentary-2021 provides good background on the torsion effects while some clarifications are required in determining the torsion governing criteria as discussed in section 4.

The current drift limit in the AS 1170.4 Code should include axial load levels and ductility as discussed in section 5. Modelling and design of secondary walls, precast construction and retention walls are important aspects of the lateral stability design as discussed in sections 6 and 7. Shear amplification and high-strength concrete are revisited in sections 8 and 9 with conclusion in section 10.

2 Link Beams

Firstly, the link (also coupling or header) beams connect walls or cores to achieve enhanced stiffness and integrated lateral structural performance. Secondly, the flexibility of the building provides a source of energy dissipation for seismic cyclic loadings etc. A coupling beam then both strengthens the building as a whole and is designed to yield first to preserve more vital parts of a building.

A conventionally reinforced coupling beam is reinforced with horizontal reinforcement and closely spaced ligatures. They are relatively simple to construct, and as such are very common in low-risk areas. Diagonally reinforced coupling beams are used in the US and NZ for deeper sections due to its excellent energy dissipation capacity. However, there are concerns of susceptibility to brittle failure and detailing issues of installing and maintaining concrete cover.

The link beams typically occur at the passages and lobby areas of the buildings and hence subject to significant services penetrations and are heavily reinforced. Currently there are no specific guidance to the design and detailing of these critical elements in the AS 3600 Code. Although sufficient level of reinforcing is required to achieve the required level of ductility, use of Intermediate Moment Resisting Frame (IMRF) beam shear reinforcement of Cl.14.5.2.2 is appropriate for MDSW (ductility factor μ = 3) and not for LDSW (ductility factor μ = 2).

Since over-reinforced and congested sections can be more harmful, it is appropriate to analyse the link beams with reduced section properties or pins/partial fixity to as illustrated in figure 1 for the FBD designs of low and midrise structures in Australia. Further research is required in this area as noted by Imperatori et al (2019) from their finite element analysis and experiments.

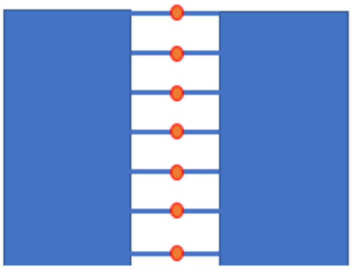

Figure 1. Link beams with pins/releases.

Isakovic et al (2024) found slabs generate considerable coupling between walls to increase overall stiffness and increase the seismic demand. Hence it appears to appropriate to use semi-rigid diaphragms for slabs in lateral analysis.

3 Diaphragms

Diaphragms are recognized as part of the lateral load resisting structure with a design section included in the AS 3600-2018 Code. The functions of the diaphragms may include transfer of imposed, compatibility and inertial forces to the lateral structure, interact with vertical elements/ form part of the moment resisting frame, provide lateral support to the vertical elements, resist thrusts from inclined columns and to resist out of plane gravity loads as illustrated in figure 2.

However, the diaphragms are reasonably flexible redundant elements with alternate load paths when designed appropriately. ACI 318-2019 recommends the diaphragm forces be increased by the over-strength amplification factor Ω_0 for higher order ductile systems where $\Omega_0 = \Omega.\omega$, Ω and ω are the over-strength and flexural amplification factors due to the higher mode effects respectively. Hence, use of μ = 1 for the design as required in the AS Code appears too conservative for the LDSW and a lesser magnitude is appropriate as per Thurairajah (2023).

The in-plane compression and tension forces are recommended in the Code to be determined using strut-tie or truss analysis. These are to be combined with the out of-plane gravity load effects to design the floor structure. Discontinuities should be limited as elaborated in the FEMA P-2012 Guidelines when establishing load paths. The typical floors generally do not require any additional considerations for the Australian seismicity. However, detailed design is required at floors where variations to vertical stiffness (ex: ground floor with retention walls, floors at which shear walls appear/disappear etc) or where horizontal stiffness changes (ex: small to large floor plates, large voids in floors etc) occur.

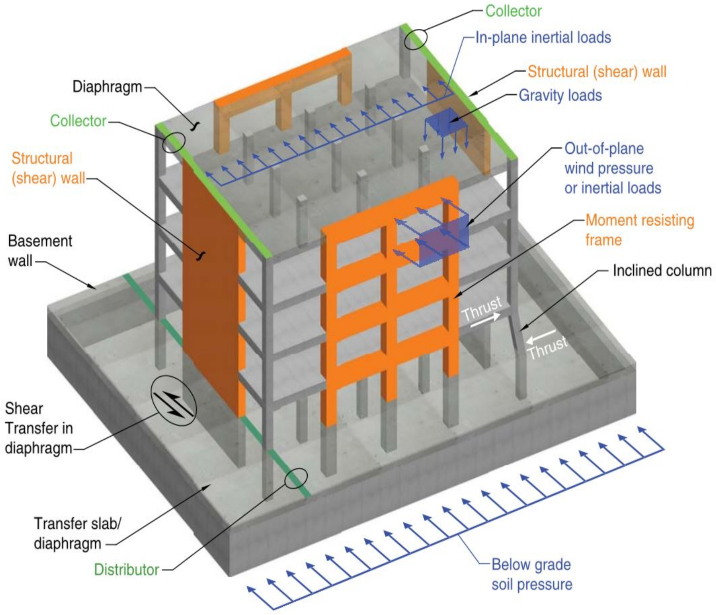

Figure 2. Diaphragm functions.

4 Torsion

Although there are no torsion limitations in either the AS 1170.4 or the AS 3600 Codes, the AS1170.4-2021 Commentary recommends some checks as structural drift instability is known to occur in the presence of torsion. One of the horizontal torsional stiffness check is referenced from FEMA P-2012 (2018) and illustrated in figure 3a. For an example building with maximum drift Δ_{max} = 10mm and minimum drift Δ_{min} = 4mm => average drift Δ_{ave} = 7mm => drift ratio $\Delta_{max}/\Delta_{ave}$ > 1.4 and will be classified as 'extreme' torsional irregularity. However, the smaller magnitudes of drifts imply insignificant consequences in practical terms, and this ratio should be somehow tied to the actual magnitudes as well (ie more concerning if the above values are 20mm, 8mm and 14mm).

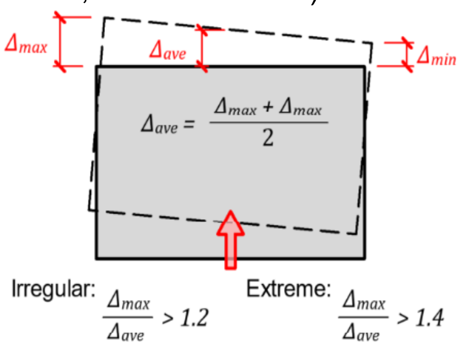

Figure 3a. Horizontal torsional stiffness.

Vertical soft, weak and discontinuous storeys should be eliminated or minimized at the early design stages. However, soft and weak storey guidance in FEMA appear to be too onerous for our local low seismicity. Lapped vertical elements (shown in blue of figure 3a) to enable continuous load paths between floors above and below as indicated should be sufficient when drifts are limited. In the vertical torsion irregularity of Figure 3b, V_i & K_i are shear & stiffness at the i^{th} floor respectively and K_θ is the torsion stiffness of the structure.

$$b_r = \frac{1}{r}\sqrt{\frac{K_\theta}{K}} \qquad b_r = \frac{1}{r}\sqrt{\frac{\Delta_{CR}\cdot(e+e_{acc})\cdot L}{\Delta_{max}-\Delta_{min}}}$$

A simplified torsional stability verification from Khatiwada et al (2020 & 2021) is included in the AS 1170.4 Commentary, where the elastic radius ratio (measure of lateral resistance against mass distribution) $b_r > 1$ is to be satisfied. In-house case studies found to satisfy this generally (appears adequate for local designs) but not the drift ratio check as above. Xing et al (2021) found performance within the displacement-controlled region of ADRS and buildings with well separated lateral elements on plan exhibit better torsion resistance.

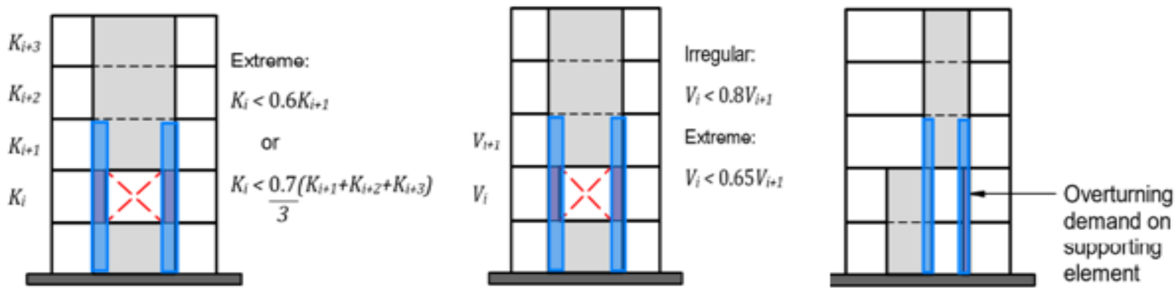

Figure 3b. Horizontal torsional stiffness.

5 Drifts

The floor plates around the perimeter of the building undergo lateral sway as well as twist due to torsional response as illustrated in figure 4 by Menegon et al (2019), forcing the columns to move along. The differential lateral sways between adjacent floor plates result in significant inter-storey drifts. The AS 1170.4 Code stipulates inter-storey drift limit of 1.5%. It was established that for satisfactory drift performance the axial loads should be kept low.

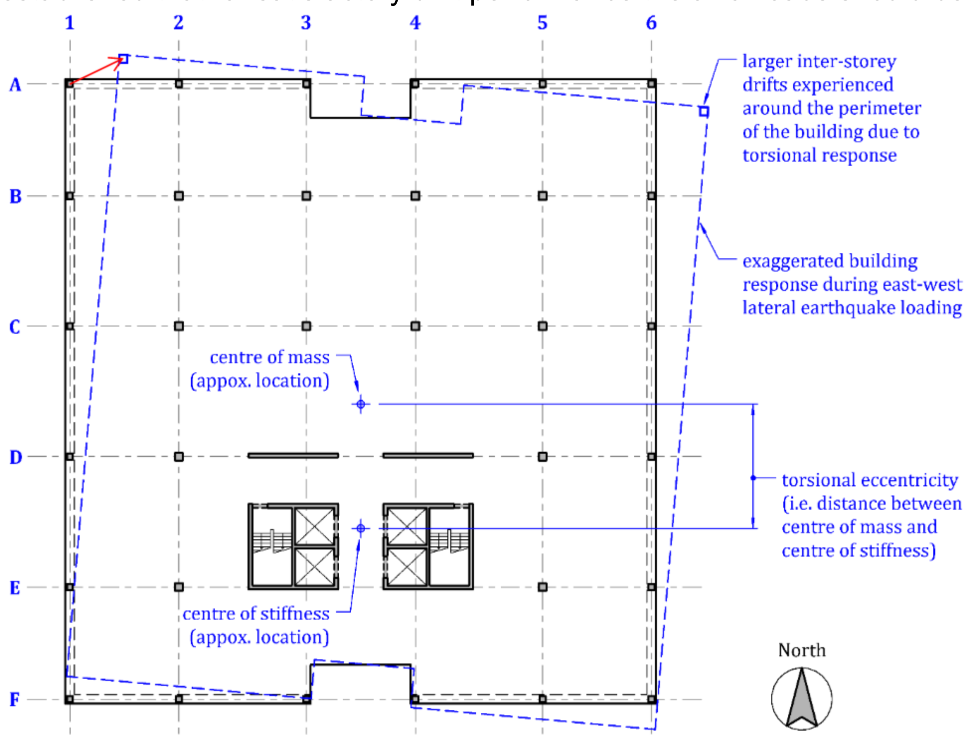

Figure 4a. Torsional drift response.

From Al-Sheik (2019), ASCE 7-16 recommends drift limits according to the risk category and type of structure. It highlights the importance of torsional drift control. The Code suggests torsional amplification for the higher seismic design categories but not for the lower ones. Raza et al (2020) outline an approximate simplification of column drift as below, where β = bi-directionality factor, n = axial load ratio and ρ_h is the horizontal reinforcement ratio.

$$\delta_{af} = \beta\left[5(1-2n) + p_h\sqrt{\frac{f_{sy.f}}{f_c'}}\right]$$

6 Secondary Walls

Reinforced concrete, generally precast, and masonry internal walls are commonly used as load bearing and non-load walls in Australia and often excluded in the lateral load resisting structure. Façade walls may also be of similar design and construction. However, the details required to prevent vertical and lateral loads transferred to these elements are generally inadequate. Non-load bearing walls should be detailed similar to the illustration in figure 5, where Details A and B demonstrate "rocking" and "sliding" types of connections with the latter with least resistance to the lateral loads.

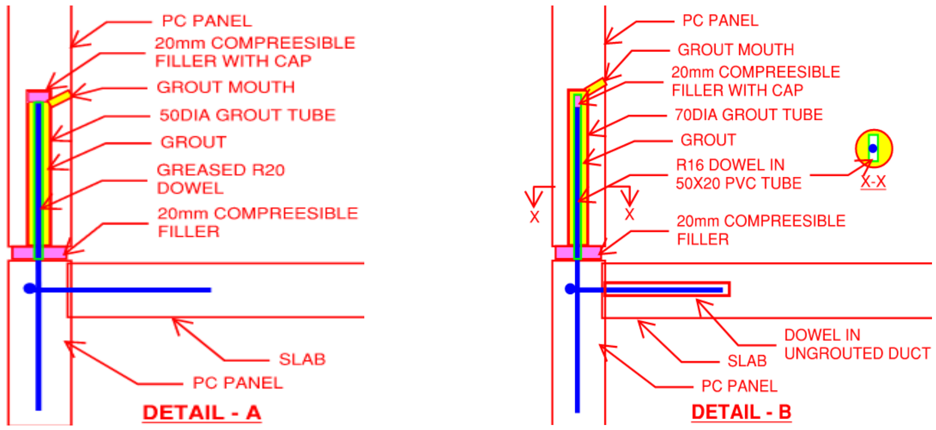

Figure 5. Non-loadbearing precast wall detail.

Other walls not detailed as above and longer than the floor-to-floor height ($L_w > h_s$) should preferably be included in the lateral model as a wall element. Very long walls may be sub-divided into lengths not exceeding twice the floor heights ($L_w < 2h_s$) with gaps in between, if local failure will not result in disproportionate collapse of the building. Precast load bearing walls not stitched together can be modelled as individual column elements and checked for drift capacity. Long blade columns with lengths greater than 6 x thickness but less than the floor height ($6t < L_w < h_s$) may be modelled as columns and checked for drift capacity.

Whichever the modelling approach is, it is important to assess with sensitivity checks to allow for adequate load transfer to the primary lateral structure or build in redundancies to allow for alternative lateral load paths.

7 Retention Walls

Please note. Retention Walls in basements generally result in significant shear reversal in the core and shear walls, hence high diaphragm forces as illustrated in Figure 6. As such upper-lower bound sensitivity checks for alternative structure rigidities are advisable to understand possible variations in load paths. Lateral stability modelling discussed for the secondary walls are applicable in this case too.

Establishment of seismic site classification is not straight forward when the basements are through different soil layers. As an example, although the earthquake excitation for both cases in figure 7a is different, both are typically classified as C_e by most Geotechnical Consultants.

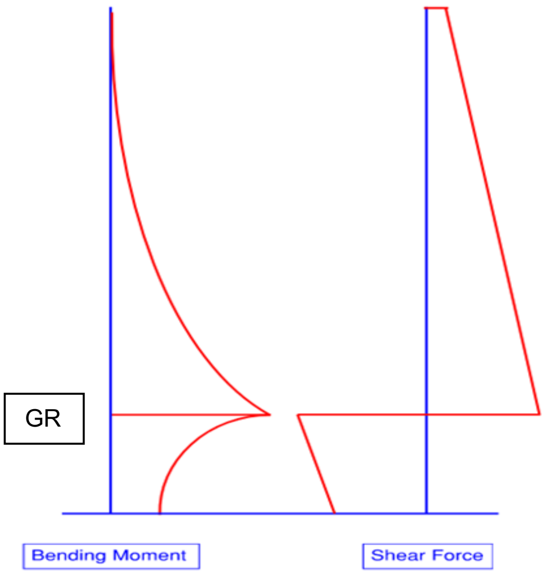

Figure 6. Core bending moment and shear force diagram with basements.

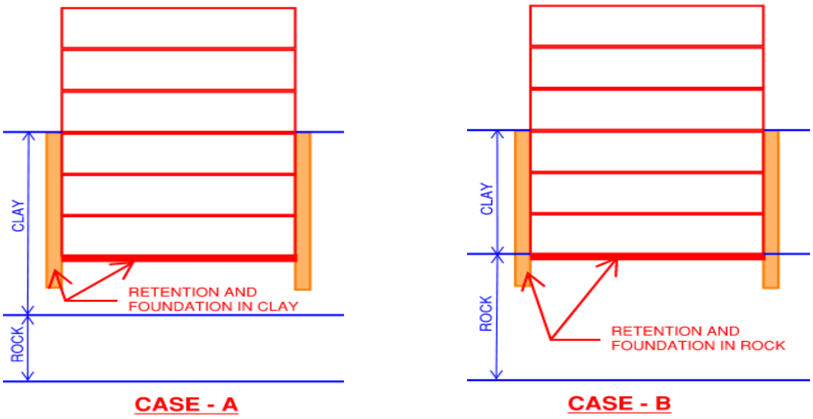

Figure 7a. Site conditions.

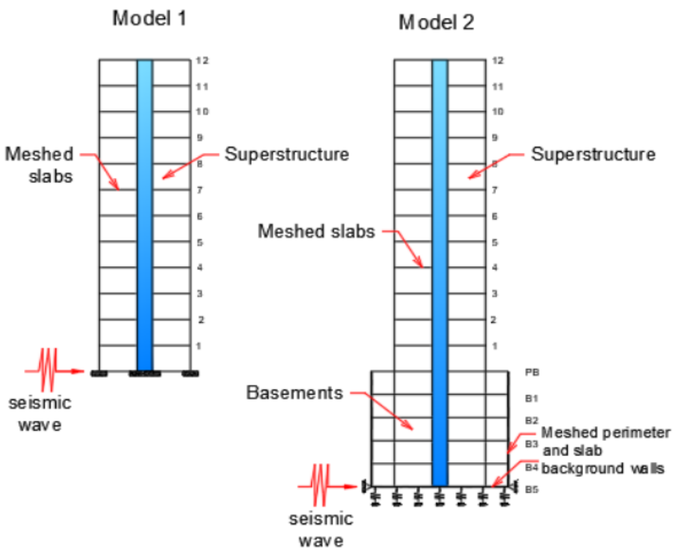

Figure 7b. Seismic input.

The soil-structure interaction SSI) consists of a kinematic component (differential stiffness of the foundation and soil) and inertial component (deformation of the soil resulting in more flexibility). Bapir et al (2023) report that base flexibility and structural yielding is beneficial for slender structures on stiff soils with natural period longer than the site period and not for shorter building on flexible soils. Hence it is appropriate to assume the seismic energy input is primarily transferred to the structure at the foundation for structures including basements as illustrated by Verdugo (2024) in figure 7b, with appropriate spring stiffness.

A school of thought is to consider seismic input through the retention with potential impact on the site classification. Figure 7c is an example with basements to evaluate this approximately. 3000mm typical floor-to-floor height, 200mm thick basement slabs, 600mm diameter retention piles at 2000mm c/c within 8m deep clay. 50% effective load transfer from the shotcrete is also assumed. 20m long retention wall, 20m wide building and foundation covering 25% of floor area are assumed. Weighted average effective contact areas between the soldier pile retention wall to the floor structure and the corresponding shear velocities are used.

Effective velocity $V_e = (V_c A_s + V_f A_f)/(A_s + A_f)$,
where seismic wave velocity in clay V_c = 240m/s and in rock V_r = 360m/s.
Number of piles = 20m/2m =10.
Effective slab contact area A_s = (10*0.6m*0.2m + (20m - 10*0.6m)*0.2m*50%)*2 = 8.0m^2.
Effective foundation contact area A_f = 20m*20m*25% = 100.0m^2.
Total contact area ($A_s + A_f$) = 8.0 + 100.0 = 108.0m^2 => V_e = (240*8 + 360*100)/108 = 351m/s.
Effective Site Period $T_e = 4h_e/V_e$ => T_e = 4*8/351 = 0.1sec < 0.15sec limit for Class B_e.
Therefore, Site Class B_e is still ok.

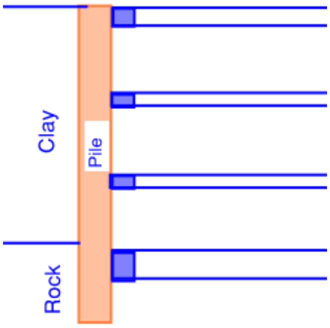

Figure 7c. Seismic excitation example.

8 Shear amplification

This topic was discussed by Thurairajah (2023) and revisited with reference to the work of Rivard et al (2022) of the inelastic higher mode effects of reinforced concrete shear walls. Amplified probable shear force demand at base of a cantilever wall is given below, where $(M_p/M_f) = 1/\phi = 1.2$ (say) for the Australian content.

$$V_{dp}^{a} = \omega_v V_{dp} = \omega_v \left(\frac{M_p}{M_f}\right) V_f$$

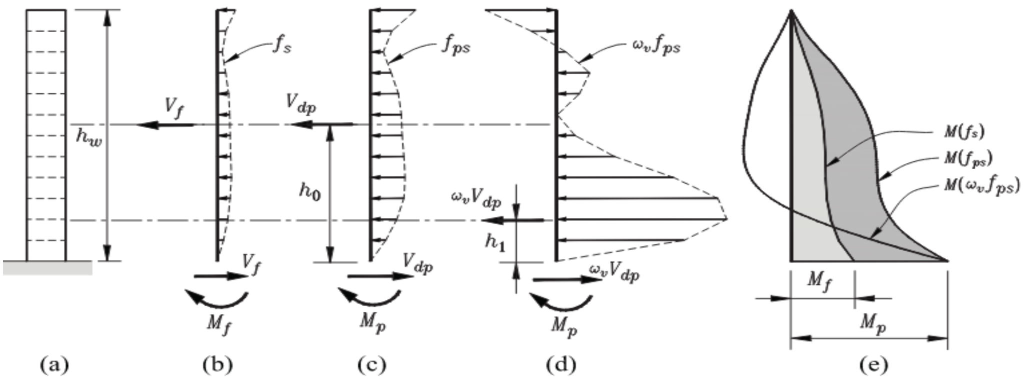

Figure 8a. Seismic inelastic shear distribution.

Figure 8a illustrates (a) shear wall, (b) factored lateral seismic forces, (c) amplified lateral seismic force for capacity design, (d) lateral seismic forces at maximum base shear with non-linear time history analysis, and (e) the corresponding moment envelopes of these cases. The greater participation of the higher modes changes the distribution of the seismic forces, and the resultant height h_1 is much lower than the original h_0.

The dynamic amplification in the Canadian Code is given below. Other formats for coupled shear walls etc are also recommended in their paper.

$$\omega_v = \begin{cases} 1.0 & \text{if } T_1 \geqslant T_L \\ 1.0 + 0.25\left(\dfrac{R_d R_o}{\gamma_w} - 1\right) \leqslant 1.5 & \text{if } T_1 \geqslant T_U \end{cases}$$

If applied conservatively for LDSW with R_d = 2, R_o = 1.3 and γ_w = 1.2 => ω_v = 1.0 for less than five levels and 1.3 for the taller buildings. Hence, total over-strength and dynamic amplification = 1.2*1.0 = 1.2 and 1.2*1.3 < 1.6 respectively appears appropriate.

Menegon (2024) pointed out the AS Code's μ/S_p (= 2.6) amplification is a deemed to comply provision only and m is sufficient if considered appropriately in a recent technical talk on behalf of the Concrete Institute of Australia (CIA).

$$\text{Shear capacity} \geq \left(\frac{\psi_o \psi_1 M_u}{M^*}\right) V^* \text{ or } \mu V^*$$

ψ_o = overstrength factor
ψ_1 = higher mode factor

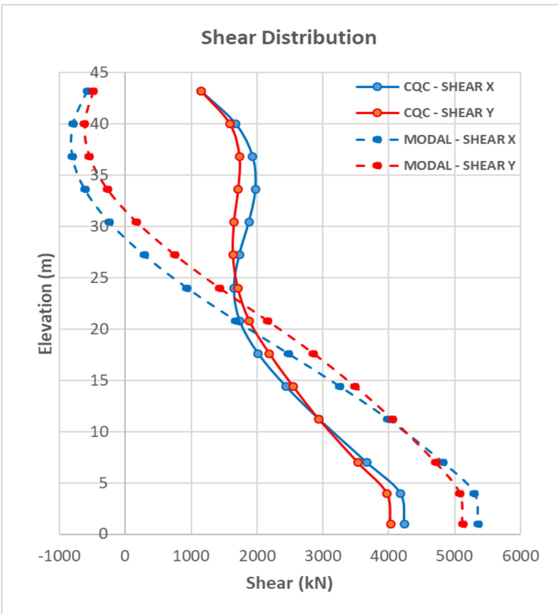

Figure 8b. Response spectra shear distribution with and without CQC.

Typical FBD designs are generally carried out using linear response spectra analysis. The process takes into account of higher mode effects with modal combinations such as CQC or SRSS with statistical weighing. The example LDSW structure with 10 above ground levels and 2 basements that was presented in the previous paper (2023) is used to illustrate the CQC based shear distribution and the modal response shear distribution in X and Y directions without any statistical weighing in figure 8b. The dynamic amplification is around 1.3. Also, the LDSW amplification appears appropriate at the lower levels only where ductility demand is required as stated in the AS3600-2018 Commentary (2021) rather than to reduced scale to the full height as recommended by Priestley et al (2007).

9 High-strength Concrete

Thurairajah (2023) discussed the concern over the ductility of high-strength concrete and the conservative application of confinement in the AS Code for LDSW. It appears to have stemmed from the high confinement required to achieve the <u>full</u> potential stress-strain behaviour of <u>plain</u> concrete cylinders under triaxial testing as referenced by Mendis (2022) in a CIA technical talk and reproduced in figure 9.

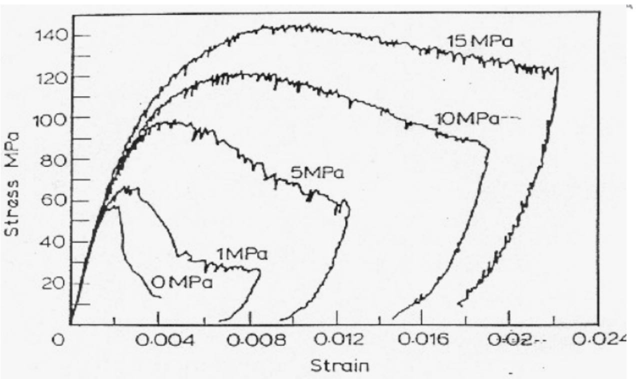

Figure 9. Stress-strain behaviour of high-strength concrete with confinement.

It is not clear why higher strength grades of concrete require full ductility utilization at quite high strain levels with high confinements when the 50MPa grade concrete with 1MPa confinement shows strength degradation beyond 0.003 strain anyway! Menegon (2019) found horizontal lapped U bars at ends is adequate generally for the 50MPa grade LDSW and dramatic increase in confinement is not expected for higher strength concrete. If no confinement is required for an axial stress of 10MPa for 50MPa grade (0.2f_c') then the same stress level is not expected demand it for the 80MPa grade (0.125fc') due to lower stress ratio from a FBD view. Limited DBD examples by the author with 80MPa concrete supports this as reported in the 2023 paper.

A stress based approximate approach awaiting an elaborate evaluation follows. 0.25MPa nominal confinement can be assumed for lapped U horizontal reinforcement at ends for 0.3f_c' compression without additional confinement (0.01f_c' pressure for columns with μ = 2 in AS3600 Commentary C.10.7.3.3 = 0.01*50 = 0.5MPa corresponding to ϕN_u then 0.25MPa for 0.5ϕN_u). Splitting tension is proportional to $(f_t')^{1/2}$ = $(f_c)^{1/4}$ should be overcome by the confinement pressure available. Hence the residual confinement available with reference to 50MPa is [1.3-$(f_c'/50)^{1/3}$]. The axial stress without confinement is **$f_c'[1.3-(f_c'/50)^{1/4}]$**, which is 0.14$f_c$' for 80MPa.

10 Conclusion

Link Beam design in Australia require more attention with guidelines for both Limited and Moderately Ductile structures. Elastic design of Diaphragm appears conservative in the AS 3600-2018 Code and may benefit from consideration of flexural amplification instead. Torsion irregularity check by Khatiwada et al (2020) is recommended for even LDSW structures with significant vertical or horizontal irregularity. Column drift capacity is dependent on the axial load ratio as found by Menegon et al (2019) and the approximate drift capacity could be determined by the recommendations of Raza et al (2020).

Modelling considerations and detailing of the load bearing and non-load bearing secondary walls, in addition to the primary lateral structure such as Cores and Shear Walls, are essential. Retention Walls in basements impose design considerations stemming from significant shear reversal and diaphragm forces. An approximate determination of seismic soil classification is also included in this submission. Current Code provisions of shear amplification is quite conservative as stated by Thurairajah (2023) and reinstated here for the LDSW structures and should only be applicable to areas of significant ductility demand. Ductility concerns of high-strength reinforced and nominally confined concrete appears unwarranted for LDSW when axial stress is limited. Well-proportioned Australian LDSW structures appear to easily satisfy the capacity design requirements other than flexible soil conditions. More focus on “macro” engineering will be beneficial in solving most issues.

11 Reference

ACI 318-2019, ACI 318 Building Code and Commentary-2019. Concrete Structures, American Concrete Institute.

Al-Sheik, A. (2019). ASCE 7-16 Provision for lateral drift determination. AEC Australia, Structure Magazine, USA.

AS 1170.4-2007, Australian Standard AS 1170.4-2007 and AS 1170.4-2007 Sup 1-2021. Earthquake Actions in Australia Code and Commentary, Standards Association of Australia.

AS 3600-2018, Australian Standard 3600-2018 and AS 3600-2018 Supplemet 1-2022. Concrete Structures Code and Commentary, Standards Association of Australia.

ATC, FEMA P-2012. (2018). Assessing seismic performance of buildings with configuration irregularities, California, USA.

Bapir, B. Abrahamczyk, L Wichtmann, T and Prada-Sarmiento, L. (2023). Soil-structure interaction: A state-of the-art review of modelling techniques and studies on seismic response of building structures, Frontiers in Built Environment, 10.3389/fbuil.2023.1120351.

Imperatori, L. Hoult, R. Lumantarna, L. and Beyer, K. (2019). Modelling reinforced concrete coupling beams designed for low-to-moderate seismic regions. AEES 2019 Conference, Australian Earthquake Engineering Society, Newcastle, Australia.

Isakovic, T. and Janevski, A. (2024). Analysis of interaction between reinforced concrete walls and slabs tested on a shaking table. Bulletin of Earthquake Engineering, The European Association for Earthquake Engineering.

Khatiwada, P. Lumantarna, E. Lam, N. and Loo, D. (2020). Fast checking of drift demand in multi-storey buildings with asymmetry. Buildings Journal, MDPI.

Menegon, S. Wilson, J. Lam, N. and Gad, E. (2019). Review of confinement requirements for the seismic design of rectangular walls in Australia, Australian Earthquake Engineering Society, Newcastle, Australia.

Priestley, N. Calvi, G. and Kowalsky, M. (2007). Direct displacement based seismic design of structures, 2007 NZSEE Conference.

Raza, S. Menegon, S. Tsang, H. and Wilson, J. (2020). Force-displacement behaviour of limited ductile high-strength RC columns under bi-directional earthquake actions, Engineering Structures, Vol. 208, Article Number 110278.

Rivard, G. Ambroise, S. and Paulture, P. (2022). Inelastic seismic shear amplification due to higher mode effects, EERI Earthquake Spectra, Vol. 38(2) 1357-1381.

Thurairajah, A. (2023). Seismic lateral stability design to AS Codes. Australian Earthquake Engineering Society 2023 National Conference, Brisbane, Australia.

Verdugo, J. and Davila, D. (2024). Seismic analysis of buildings with basements, MATEC Web of Conferences 396, 03004 (2024), WMCAUS 2023.