1. INTRODUCTION:

For thirty years or more structural design has focused on forced-based (FB) design methods, where inertia of the structure generates forces within the structure. Strength capacity of the building compared to the demands of the forced based analysis was the primary basis for assessing structure performance. There is a check on interstorey drifts and overall building lateral deflection in the process. In the last ten years it has been recognised that lateral deflections and material strain capacities may be a better means of assessing building and building element performance (Priestley, 1995).

Therefore, this paper will discuss in general terms what displacements may be expected in a major seismic event in Australia and how the detailing practices of AS 3600 and AS4100 might fare in dealing with these lateral displacements. Some suggestions are made for minor changes to increase the robustness of frames in reinforced concrete frames and walls and structural steel frames.

2. LATERAL DISPLACEMENTS AND ACCELERATIONS OF BUILDINGS DURING TO THE EARTHQUAKE:

The Response Spectra of NZS 1170.5:2004 and Draft AS 1170 Part 4 will be used to make some simple comparisons between New Zealand and Australian force and displacement demands.

2.1 Lateral Accelerations of Buildings due to the Earthquakes

Following a similar discussion to the earlier paper, for Class C – Shallow Soils (see Appendix A, Site Subsoil Classes – Standards Australia, Draft As 1170:4), Figure 1 indicates the relative lateral accelerations of a normal occupancy commercial building with Probability Factor k_p = 1.0, S_p =1.0, μ = 1.0 for an Australian intraplate earthquake in Sydney compared to an interplate event in Wellington. Auckland, which has a portion of its derived hazard based on intraplate events but is artificially made higher, is plotted for comparison.

For short period structures with $T < 0.3$ sec, the Wellington building has 3.6 times the acceleration of the Sydney building. At $T = 1.5$ sec, a building in Wellington is subjected to greater than 5 times that of a building with the same Period, T, in Sydney.

2.2 Lateral Displacements of Buildings due to the Earthquakes

Figure 2 shows earthquake induced lateral displacements of the Centre of Mass (CoM) of a building with, k_p =1.0, for Sydney, Wellington and Auckland. Depending on the type of building (moment resisting frames vs. wall or concentrically braced frame) the effective location of the Centre of Mass is 0.6 to 0.7 times the height of the building. Wilson and Lam (2003, 2005) explain clearly the development of displacement response spectra for Australian application.

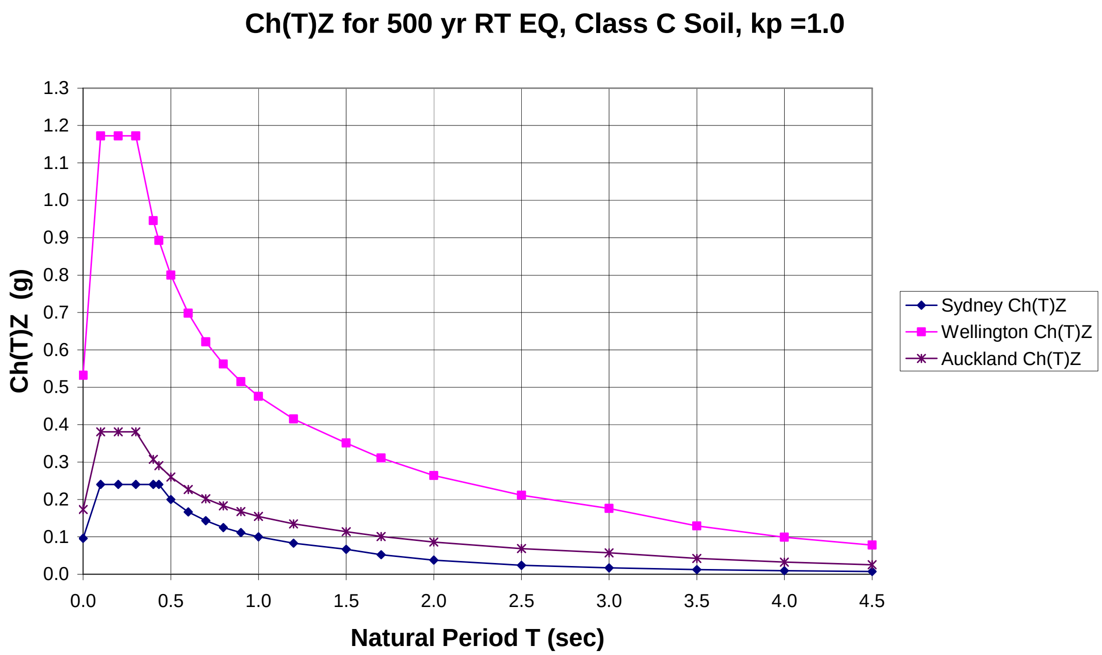


Figure 1: Sydney, Auckland and Wellington Elastic Hazard Response Spectra for Shallow Soils, Class C

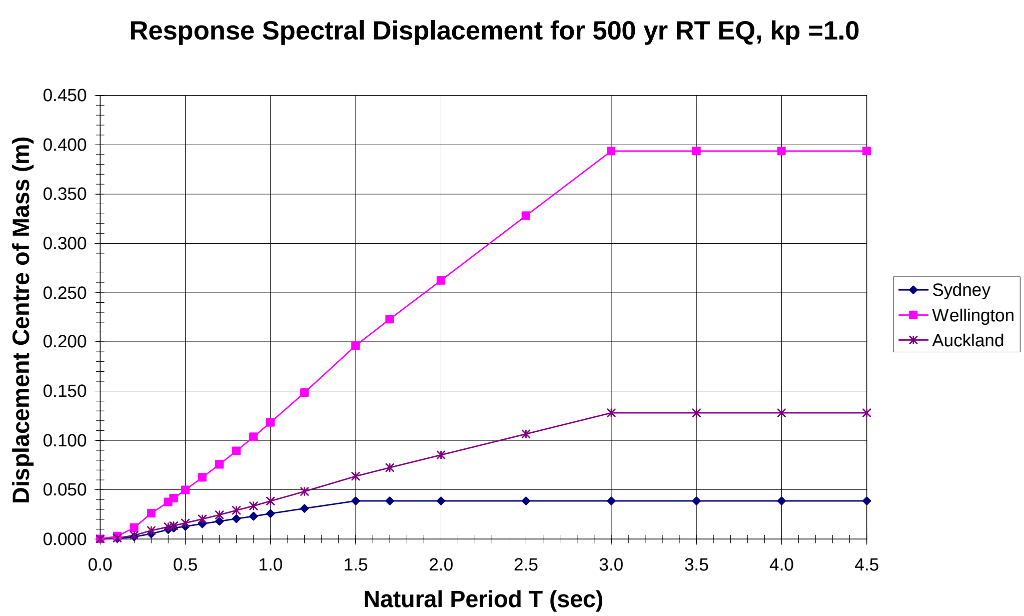


Figure 2: Sydney, Auckland and Wellington Elastic Displacement Response Spectra for Shallow Soils, Class C

Maximum CoM displacements of the building (Class C soil, R =1.0, T> 3.0 s):

Sydney	Displacement at CoM	39 mm	(cf. Rock B: 26 mm)
Auckland	Displacement at CoM	128 mm	(cf. Rock B: 102 mm)
Wellington	Displacement at CoM	387 mm	(cf. Rock B: 313 mm)

However, at short periods, T, from Figure 2, it can be seen that Auckland and Sydney have very similar displacements. The long period values result, in part, due to the method used to predict the Auckland hazard.

Should the same building be founded on soft to very soft soils (Class D and Class E) these displacements will increase by up to 3.5 times, depending on the fundamental Period $T_1$of the building.

2.3 Probable Lateral Displacements and Implications on Structural Types

For the sake of discussion, undertake a very rudimentary sensitivity analysis looking at building type, building height, Period of Vibration, T_1, ULS Displacement, Δ_u, (Return Period *RP*= 500 years) and Peak Interstorey Drift.

The peak Interstorey Drift is assumed to be equal:

- For frames: CoM displacement extrapolated to the roof, averaged over the number of storeys and amplified by allowing for higher mode effects by multiply by 2.

- For walls and braced frames: CoM displacement extrapolated to the roof, averaged over the number of storeys and amplified by allowing for higher mode effects by multiply by 1.5.

In this study, the Probability Factor, k_p =1.0 and the site subsoil class is C.

The questions are:

> *"For a range of buildings, what is the CoM lateral displacement for the ULS earthquake (RT=500 years), what is the peak interstorey drift and* ***is the capacity of these structures able to deal with these demands****?"*

One of the suggested methods of estimation the Fundamental Period of Vibration or the "Structural Period", T_1, of a building, is the approximations from the following equations (used with the Equivalent Static Method), at Ultimate Limit State (Design earthquake with Return Period of 500 years) (Standards Australia, 2004).

$T_1 \quad = \quad 1.25 k_t h_n^{0.75}$ for the ultimate limit state

Where

k_t	=	0.11	for moment-resisting steel frames
		0.075	for moment resisting concrete frames
		0.06	for eccentrically braced steel frames
		0.05	for all other frame structures
h_n	=		height from the base of the structure to the uppermost seismic weight or mass

Presented in Table 1 and 2 are the results of the overly simplified sensitivity analysis which was done on an elastically responded buildings.

Trends to note are that for the normal use and subsoil Class C Australian buildings lateral displacements are **not** significant!

"significant" meaning:

- Relative to high seismic zones of New Zealand.
  - Take the 15 storey steel moment resisting frame in Wellington, same fundamental period, essentially, resulting in a ULS displacement at the CoM of 390mm, compared to 39mm for Sydney (see Figure 2).
  - This means that the likely interstorey drift in the lower floors of the 15 storey steel moment resisting frame building, in Wellington is about 50mm or 1.5%. Where in Sydney, the interstorey drift could be in the order of 5mm or 0.15%
- Both reinforced concrete and structural steel moment resisting frames of conventional geometry yield at about **0.6 – 0.9%** interstorey drift: applicable in NZ and Australia.
- This may be interpreted that Australian ***moment resisting frames*** probably do not yield or at low risk of yielding significantly (forming a full plastic mechanism as per the building in Fig 3(a)) with the maximum interstorey drift being less than 0.6%, (see Table 2).
  - Even if a building was designed to be elastic throughout, should the building, by way of the cladding and fit out, result in enhanced strength in the upper storeys, then all the lateral deformation may occur at one level. This is very common in the ground floor of commercial buildings where the ground floor is open space for retail, while the upper floors have significant cladding and multiple partitions (between offices), stiffening and bracing those upper floors.

– Should the overall building displacement occur at one floor (a side sway mechanism - see Figs. 3(b) and 3(c)) then the interstorey drift would be around 1% (see Table 3) that is, just yielding. This could be said of any height building under the k_p =1.0, *RP* =500 years and subsoil class C, as the maximum drift at one floor is limited by the displacement plateau of 39 mm (see Figure 2).

– If the mechanism forms across one floor, then the likely ductility demand will be low, if the interstorey drift is around 1%, for these Sydney buildings (Table 3).

➢ Much the same conclusion can be drawn for cantilever REINFORCED concrete and masonry walls.

➢ Concentric braced frames yield in the braces at around 0.2% interstorey drift.

– The degree of ductility required to accommodate the inelastic demands at this level of drift is relatively low.

Table 1: Period T_1 and Building Type (Australia)

Building*	k_t	Storeys	Height h_n *m*	$T_1 = 1.25 k_t h_n^{0.75}$ *sec*
Walled building (concrete or masonry)	0.05	4	14.5	0.47
moment resisting concrete frames	0.075	6	21.9	0.95
moment-resisting steel frames	0.11	6	21.9	1.39
concentric & eccentrically braced steel frames	0.06	6	21.9	0.76
moment-resisting steel frames	0.11	15	54.75	2.77
Walled building (concrete)	0.05	15	54.75	1.26

* Elastically responding building

Table 2: Building Type, T_1 & Δ_u (Australia)

Building*	T_1 *sec*	Storeys	h_n *m*	ULS Displacement Δ_u *mm*	Peak Interstorey Δ_u *mm*	Peak Interstorey Drift **%**
Walled building (concrete or masonry)	0.47	4	14.5	12	6	0.17
moment resisting concrete frames	0.95	6	21.9	25	9	0.25
moment-resisting steel frames	1.39	6	21.9	36	12	0.37
concentric & eccentrically braced steel frames	0.76	6	21.9	19	5	0.15
moment-resisting steel frames	2.77	15	54.75	39 **	9	0.25
Walled building (concrete)	1.26	15	54.75	32	4	0.12

* Elastically responding building.
** Maximum lateral displacement from displacement response spectra (RP =500yr)

Table 3: Moment Resisting Frames, Δ_u at one floor (Australia)

Building*	T_1 *sec*	Storeys	h_n *m*	ULS Displacement Δ_u *mm*	Interstorey Drift for Δ_u at one floor **%**
moment resisting concrete frames	0.95	6	21.9	25	0.68
moment-resisting steel frames	1.39	6	21.9	36	1.00
moment-resisting steel frames	2.77	15	54.75	39 **	1.07

** Maximum lateral displacement from displacement response spectra (*RP* =500yr)

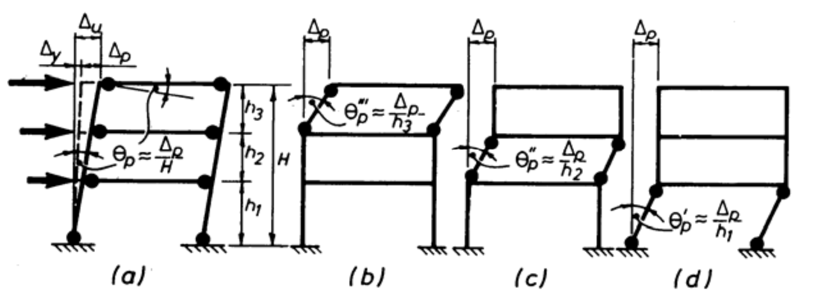


Figure 3: Sway Mechanisms in Frames (Paulay & Priestley, 1992)

It is usually recognised that the "general" detailing of AS 3600 (Standards Australia, 2001) will provide a displacement ductility to a member of 1.5 and for AS 4100 (Standards Australia, 1998); similar to New Zealand general detailing in most respects, while resisting earthquake attack, the structural should remain mostly elastic.

The above discussion was based on a 500 year return period earthquake and site subsoil class C. Two issues need to be considered:

a) "Avoidance of collapse" Limit State would require the displacements to be increased by 1.8, the ratio of the Probability Factor k_p (2500 year) to Probability Factor k_p (500 year). The "2500 year" event is deemed to be the Maximum Credible Event. Bear in mind, the experience of the east coast of North America, another interplate region, has indications that the ratio may be as high as 2.5 or even higher.

   If the frames were at or just yielded at the 500 year event (particularly if the a soft storey or side sway mechanism forms), does the detailing of AS 3600 and AS 4100 provide inelastic capacity associated not only the 500 year event but also the 2500 year event?

b) What is the influence of building the same building on site subsoil Class D and E sites?

   The ratio of lateral displacement from Class C to Class D is up to about 1.5 and Class C to Class E is up to about 3.5.

In the case of long period structures, frame buildings (T > 1.5 seconds) for Class D soil sites, assuming the peak displacement occurs **at one level**, the peak displacement would be 39mm x 1.5 (Class D/Class C soils), divided by the typical interstorey height of 3.65m produces an interstorey drift of 1.6%.

The general detailing of AS 3600 may accommodate this. However the connections in Ordinary Moment Resisting Frame steel buildings may have problems. These connections become the weakest link with very little ability to deal with plastic rotations in weld sites or bolt groups.

For Class E soil, from Figure 2, the maximum interstorey displacement is approximately 39mm x 3.5 = 98mm. **At one level**, if the storey height is around the typical 3.65 m then the interstorey drift would be 3.7%. P-Delta an issue now.

Another way of interpreting this is that if a typical frame has a sway mechanism at 0.7% drift, notational "first yield" of the building, then to displace a further 3.5 times (Class E soil) implies that the displacement ductility μ for the building is about 3.5 (assuming the structure was permitted, by design, to drift this far).

If the stiffness of the structure was tuned to keep the expected drift to 1.5% then the effective displacement ductility μ of the building would be about 2.

This is an issue for the material Standards producers. Does the general detailing prescribed in AS 3600 and AS 4100 provide this order of displacement ductility? Again, maintenance of load paths needs to be considered.

This demand for ductility can not be mitigated by simply increasing the strength of a conventional frame building. Increasing the strength can increase the stiffness and hence shorten the Period, T_1. This in turn reduces the lateral displacement. However, the change of stiffness with increased strength is not marked for conventional structural forms and is ineffective if the Period of the building is greater than 1.5 seconds in Australia: the constant displacement zone of the displacement response spectra.

The old adage of "double the strength: halve the ductility" does not apply. The issue is for a resulting lateral displacement, for a given earthquake, do the structural elements yield or not and how much plastic deformation is needed.

Similarly, for collapse avoidance in a Maximum Credible Earthquake (MCE), the ductility $\boldsymbol{\mu}$ of the side sway mechanism would be between 2 and 3 for Class C soils or better. This increasing ductility demand compounds when the soil sites are Class D or Class E. Though there is a debate on what material properties are used at MCE.

3. POSSIBLE SUGGESTIONS FOR AS 3600 AND AS 4100 WHEN DUCTILITY MAY BE NECESSARY.

3.1 Irrespective of seismic actions causing ductility demands, there are other sources of ductility (settlement of the building, shrinkage, thermal effects, redistribution of internal actions) such that only N class steel should be used (or E Class) (AS/NZS 4671:2001; Bull 2003) in all reinforced concrete elements.

3.2 In order to take advantage of the low lateral displacement demands typical of the Australian earthquakes, with respect to frames (moment resisting) one option would to be make the sum of the column flexural strengths above and below a floor level, larger by a suitable margin, than the sum of the flexural strengths of the framing in to the columns at that level. Note, it is flexural strengths described here, and not simply the strength demand determined from a structural analysis. The summation is done at the centroids of the beam-column joints (Priestley, 1995).

A sway potential index S_i can then be determined:

$$S_i = \frac{\sum(M_{bl} + M_{br})}{\sum(M_{ca} + M_{cb})} \qquad \text{....Eq. 1}$$

Where M_{bl} and M_{br} are the expected maximum flexural strengths either side of a joint and M_{ca} and M_{cb} are the minimum expected column strengths above and below the joints; all determined at the centroids of the joints. These are summed for all joints across the level being investigated.

If $S_i < 0.85$ then it should be assumed that column plastic hinges will form. A target $S_i \geq 1.15$.

This is a rudimentary approach for reducing the risk of a column side sway mechanism forming (see Figure 3(b) and 3(c)), maintaining with some degree of certainty for the desirable weak beam/strong column mechanism – see Figure 3(a).

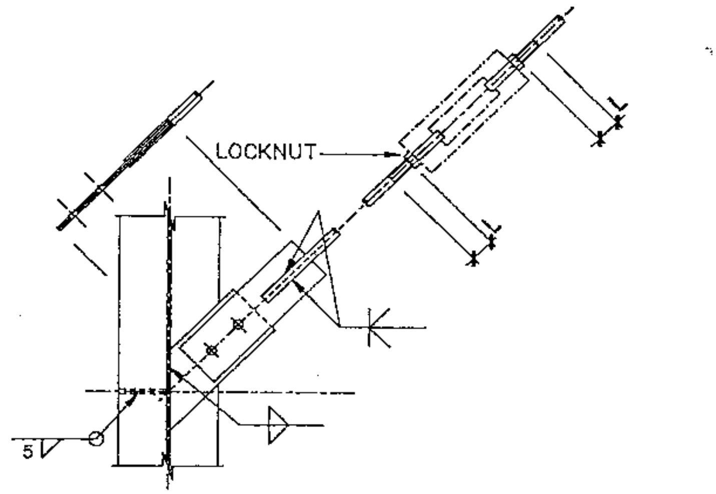
3.3 The general provisions of AS 4100 would satisfy for lateral displacement demands of normal use, RT = 500 years, Class C or better soils, if the structure remained elastic (Category 4, NZS 3404:1997). It is suspected that this is very common as wind would govern the strength of many buildings as it does in Auckland, NZ.

In New Zealand, the NZS 3404 is based on the AS 4100 provisions for bare steel member design. However, in New Zealand as soon as a member forms part of the primary lateral force resisting system, Section 12 of NZS 3404 governs: the *seismic provisions*.

It is not being suggested that the same full comprehensive philosophy be employed in Australia. However, it is suggested that the load paths through primary structure, particularly the connections may be enhanced.

- If a frame (moment resisting or braced) is to have any ductility in the members, then this plasticity should be restricted to the members and not the connections (welds or bolted details). In order that the connections do not become the weakest link, primary members resisting the earthquake forces should have connections at least as strong as the members (in NZ, the connections are at least 1.15 time the strength of the members).

- If the structure remains elastic then the connections can be detailed to meet the design actions determined from analysis (maintaining the minimum connections strength of Cl. 9.1.4: Minimum design actions on connections – AS 4100: 1998.).



To achieve member category 2 ductility, length L must be at least 100 mm but not less than three times the bar diameter. The length L is measured to the loaded face of the nut.

Figure 4: End connection detail for tension braces using round bar with threaded ends (Feeney and Clifton, 1995)

- Braces constructed of threaded rods and turn buckles are a concern in New Zealand. These typically have little ductility with the rod fracturing at the base of the thread. The almost non-existent ductility will be relevant to Australian detailing. It is recommended that when rod tensions braces are the primary lateral force resisting system then notched braces be employed (Ref. 6 and 9). If a thread rod is to be used with a turnbuckle then at least 100 mm of thread each side of the turnbuckle should be available and the rod should be Grade 300 N or E.

3.4 Review of the Section 13: EARTHQUAKE of AS 4100 would appear appropriate for "limited ductility" design for Intermediate Moment Resisting Frames (IMRF) where the local demands in members or elements would either have a local displacement ductility of 3 or less, or a curvature ductility of 10 or less.

3.5 Concentric Braced Frames (CBF) would be useful and practical (elastically responding up to 32 storeys and nominally elastic, $\mu \leq 1.25$, up to 24 storeys.

In the case of the nominally ductile CBFs, the connections would need to match the strength of the members. Interstorey drift beyond the elastic capacity of the CBFs can cause localised failures of connections in CBFs

(Wallace et al, 2002). These studies produced good suggestions of avoiding connections as the weakest links in the load paths.

3.6 The general provisions of AS 3600 would most likely satisfy the displacement ductility demands of normal use, RT = 500 years, Class C or better soils. For columns, it is suggested that the following be considered for all classes of frame (OMRF, IMRF and SMRF) and for boundary elements of walls, as minima:

- The spacing of ties and helices is deceased to 10 d_b or $D_c/2$ – the smaller over the full height of the column or the height of the wall boundary element equal to the length of the wall (potential zone of yielding). The spacing is currently the smaller of D_c or 15 d_b (Cl. 10.7.3.3 AS 3600-2001). The tighter spacing for bundled bars is adequate.
    - Note: Those R10 stirrup-ties at 10 d_b would probably cover the prevention of longitudinal bar buckling and provide a reasonable level of confinement of the concrete core of the element in all cases.
    - A spiral should be terminated by either welding to the adjacent turn or by a 135° hook. Lapping in the cover concrete should be discouraged.
- This would bring the OMRF up to a reasonable level of robustness for columns and walls for nominal cost: to a level very similar to IMRFs.
- Beam-column joint design is based on providing shear capacity for all classes of frames (OMRF, IMRF and SMRF).

For beams, it is suggested that all classes of frames (OMRF, IMRF and SMRF) have stirrup ties on most longitudinal bars in the order of R10s at 10 d_b spacing for $2D_b$ from the face of the supporting columns (NZS 3101:1995).

Lapping of plain round bars as fitments (stirrup-ties) across members might be discouraged. Lapping ties, that are not the perimeter stirrup, can be done if the bars have a deformed/ribbed profile.

Where lapping of longitudinal bars occurs and the stresses in the bars exceed 0.6 f_y in tension or compression then the transverse reinforcement should be at least (NZS 3101:1995):

$$\frac{A_{tr}}{s} \geq \frac{d_b f_y}{48 f_{yt}} \qquad \text{....Eq. 2}$$

Where A_{tr} = area of the tie

s = spacing of tie $\leq 10d_b$

d_b = diameter of longitudinal bar

f_y = yield strength of longitudinal bar

f_{yt} = yield strength of tie or stirrup

For example: R12 at 225 mm spacing should be sufficient to restrain lapping D25 bars. Typically this will be 3 or 4 sets of stirrup ties along the lap length.

3.7 Review of the Appendix A: Additional Requirements...Earthquake Actions of AS 3600 would appear appropriate for "limited ductility" design where the local demands in members or elements would either have a local displacement ductility of 3 or less, or a curvature ductility of 10 or less. In places, aspects akin to ACI 318 (Ref. 8) practices may be conservative.

With respect to Intermediate Moment Resisting Frames (IMRF):

In beams, laps in longitudinal reinforcement should be located away from the faces of columns by at least D_b and bound by three sets of stirrups-ties (not the two of A12.3.2.1 (d)).

An additional check on the stirrups-ties to restrain the potential buckling of longitudinal bars in the zones 2D from the face of the supports may be included. It is suspected that the shear reinforcement may be adequate in any case.

Current detailing of IMRFs should satisfy the "limited ductility" cases.

It is suspected that Special Moment Resisting Frames (SMRF) will be very rare.

3.8 One recommendation of Appendix A, A9.2 (AS 3600-2001) with regard to detailing the connections of exterior panels to accommodate relative interstorey displacements should be applied to all classes of building that resist earthquakes.

4. CONCLUDING COMMENTS

Determination of the locations and amounts of inelasticity in a building system can be complex. This depends on the geometry of the members in the frame, material characteristics (yield strength, modulus of elasticity, shear modulus and post-elastic characteristics). So the generalities described above should be interpreted as indicative and provocative. "Provocative" in the sense of suggesting strongly that calibration /sensitivity analysis of significantly more depth and sophistication be undertaken to confirm the comments made here with respect to Australian reinforced concrete and structural steel structures.

In summary:

- Conventional steel frames that remain elastic and reinforced concrete frames that are nominally ductile ($\mu \leq 1.25$), other than on subsoil Class E sites, are probably satisfactorily detailed in accordance with GENERAL PROVISIONS of AS 3600 and AS 4100.

- **Reinforced** concrete (and masonry) cantilever structural walls are likely to be satisfactory on sites other than on subsoil Class E sites,

- Minor modifications to existing detailing would add a degree of robustness to the structures at what should be minimal cost penalties.

- On the very rare occasions where ductility is required, the demand will probably not exceed a local displacement ductility of 3 (or a curvature ductility of 10). This ductility demand is denoted as "limited ductility" - this discussed in depth by Paulay and Priestley (1992) and relatively easy to achieve without cost penalties.

  It would appear that the Section 13 of AS 4100 and Appendix A of AS 3600 would be satisfactory for "limited ductility" design for IMRF and SMRF.

- Connection strength matching the member strength in OMRF in structural steel is recommended and attention of stirrup-tie details (e.g. closing up spacing from $15d_b$ to $10d_b$) and beam-column joint design in reinforced concrete, then OMRFs would be satisfactory in the cases of unanticipated ductility demands (bigger than expected earthquake) or on soils Class E.

- Concentric braced frames would be useful and practical (elastically responding up to 32 storeys and nominally elastic, $\mu \leq 1.25$, up to 24 storeys).

- It is recommended that the Technical Committees of AS 3600 and AS 4100 review the sections incorporating additional requirements for seismic effects to ensure that the observations discussed here are correct. If there are any short comings in these sections, the detailing practices and incorporation of "capacity" design associated with New Zealand practices for "limited ductility" might be appropriate (NZS 3404, NZS 3101, Feeney and Clifton, and Paulay and Priestley) or the special seismic provisions of ACI 318 (ACI 318M-05) may be a viable alternative.

- The issue of avoidance of collapse then becomes dominated by maintenance of load paths through and out of the structures. "The Devil's in the detailing" and in some cases the tying of structures together may require some attention. It is believed that this will not be technically or cost prohibitive to achieve.

  Further, if this collapse limit state is consider important in Australia, then the materials Code producers will need to bear this in mind in order to provide added robustness to the detailing employed for the design event (RT = 500 years).

5. ACKNOWLEDGEMENTS

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