Structural Design of Robust Buildings for Extreme Events: Early New Zealand 'Capacity Design'

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This paper provides a brief exposition of the 'Capacity Design' procedures used in New Zealand for the design of reinforced concrete structures to be fully ductile and robust. The author sees this as an issue primarily about rebar detailing. This paper is intended, first of all, for Australian readers and so one must first establish that there is some interest and/or need for such a paper in Australia. Informed comment/discussion may come from New Zealand and that would be welcome.

'Capacity Design' was developed in New Zealand from about 1971 when the Los Angeles (San Fernando) earthquake, coming soon after the Alaskan earthquakes of the late 1960s, finally made it clear that the state of the art then was just not good enough. There is a good deal of conversation New Zealand / USA on earthquake matters and one assumes that ACI-318-2011 is not very different to NZS3101. The author's direct knowledge of NZS3101 ceased from about 1987 after completing the design of a tall Australian building to NZS3101on client instructions.

Australian consulting engineers now design many buildings outside Australia and most other countries in the Asia/Pacific region have levels of earthquake risk that are a good deal higher than Australia. The recent Christchurch earthquake had a return period there of about 200 years. Perhaps the corresponding period for a similar earthquake in an Australian capital is 2000 years but there is no guarantee that a 2000 year, or even a 10,000 year event will not happen tomorrow or next week. Unlike wind, earthquake loads and displacements involve large increases with return period. The extent of damage and death in an earthquake, particularly for earthquakes larger than the 'design earthquake', is quite sensitive to the differences between 'limited ductile', 'moderately ductile' and 'fully ductile' detailing.

Most large, real structures rarely experience anything like full design loads except, perhaps, for water-retaining structures. If we are to continue to improve our understanding of large, real structures then we need to learn from their performance in extreme events including earthquakes but also other extreme events such as Ronan Point 1968, Oklahoma City 1995 and the World Trade Center, NY 2001.

The author has published papers on these other extreme events with AEES (Canberra 2006) and ASCE. These earlier papers were particularly focussed on the FEMA 277 report of 1996 on the Oklahoma City event by a Chicago-based ASCE expert committee. This committee concluded that the extent of damage at Oklahoma City could have been reduced 80% - 85% by changes in rebar detailing that would have cost 1% - 2% of the total cost of the building. It remains true that methods for improving ductility and robustness, especially in rebar detailing, are much cheaper and more effective done during original construction rather than later.

FEMA 277 recommends such detailing for all new US Federal buildings to be built after the report. The death-toll of 167 (no survivors and most crushed by falling concrete rubble) might, perhaps have been reduced to about 25. Who can argue against this extra 1% - 2%? The cost of rebar is about 10% of building total cost so the extra cost is 10%-

20% of rebar costs. But most of this extra rebar would be in places where there might otherwise have been no rebar at all or it would have been too widely spaced to be useful.

The devil is, as the saying goes "all in the detail". The author will not repeat the content of his AEES Canberra Conference 2006 paper but just consider one example of that detail: properly anchored bottom rebars at supports. AS 3600 S8 requires only that 25% of bottom rebars continue past the face of the support, perhaps as little as 50 mm thus effectively zero. The rest can be stopped $0.10*L_n$ into the clear span. This does qualify as 'limited ductile' ($\mu/S_p = 2.6$).

Earthquake horizontal load produces tension-bottom peak moments at alternating beam-ends. If this exceeds the minimum gravity load moment (0.9*G less any upward acceleration) then there will be a nett tension-bottom moment at a section that might have zero effective bottom reinforcement. There is nothing in AS3600 S8 or AS1170.4 to prevent this. Who can be certain that the real earthquake load will be no greater than the 'design earthquake' specified in AS1170.4? How ductile is a section with zero tension rebar?

Then there is the risk of one or more lost columns; 3 adjacent columns were lost at Oklahoma City; one to the terrorist bomb and two to shear failure perhaps due to the larger tie spacing permitted in columns. These three column locations then became mid-span zones over an extended span. Some properly anchored bottom rebar would have been helpful. It might even have prevented the disproportionate/progressive collapse that actually happened by acting as a catenary.

Would one be happy to design, say, a critical-care facility for a major public hospital with 'limited ductile' detailing? Just to save 1% - 2% of total building cost, which, as FEMA 277/ASCE points out, is a good deal less than the scatter between tenders. This author's AEES Canberra paper suggested that minimum standards for ductile detailing be related to the Importance Level as established in BCA. Under BCA, post-disaster recovery buildings are Importance Level 4 (IL4) with a return period of 1500 years. He has made similar suggestions for US buildings in his ASCE papers.

This does now seem a crucial issue in the light of the Christchurch earthquake of February 2011. There were about 150 buildings with lifts usually implying more than 6-8 storeys including a few up to 25. All but two of these killed nobody at all whether in the building itself or in the surrounding streets. This is, of itself, a great result for New Zealand engineering expertise. One quite old taller building killed a small number of people. One understands that it had been 'tarted up' to look modern without doing significant structural strengthening. One other building built mid-1980s almost tripled the otherwise death-toll being responsible for 115 out of 185 total fatalities. A New Zealand Royal Commission is now spending some months on this particular building and one can only wait for conclusions. Perhaps it is conceivable that this collapse had to do with structural rebar detailing? Building owners, tenants and the public in New Zealand are now demanding that earthquake resistance be drastically improved. This may seem over-stated but who can say that this demand will not extend elsewhere in the Asia/Pacific region where Australian consulting engineers do business? Perhaps even to Australia?

CHECK LIST 'DF' FOR DUCTILE FRAME BUILDINGS

The order of this list and the tags (DF1 etc) is the author's own invention but the ideas had been in circulation in New Zealand and America since a year or more before the author migrated Sydney to Christchurch in July, 1972.

DF1 Strong columns / weak beams
DF1R Revised early 1980s? : some (but not all) weak columns with stronger beams but …Weak columns confined full-height to DF4
DF2 No shear failures in ductile beams
DF3 Bottom rebar through supports
DF4 Confinement of columns
DF5 Shear reinforcement in columns
DF6 Column splices
DF7 Panel zones at joints

DF1 Strong columns / weak beams

In its original form, rule DF1 required that the total column strength M_u (below + above if any) at any specific joint be >, say 130% of the total beam moment strength (left + right) associated with yield both sides. If there are beams in both plan directions then the column biaxial strength should be, say 130% of the total vector beam moment strength north, south, east and west. The beam moment strengths will be negative (tension-top) on one face and positive (tension-bottom) on the opposite face but absolute values are to be added. It will be difficult, probably/perhaps impossible, to achieve this objective if the columns are significantly smaller than the beams. All cases are to be calculated with the factored gravity load during earthquake and the 'over-strength' main rebar yield in the beams but nominal (ϕ=1) yield in the columns. If there are long-span beams with significant gravity loads then the positive (tension-bottom) moment hinge may migrate out into the span. The positive moment at the column face will then be less than the mid-span hinge value.

The purpose of this rule is to prevent "soft storey" failures in which all of the post-elastic seismic energy has to be absorbed within a single storey usually/often the lowest storey. Given that explanation, the revision, from one thinks, the early-1980s, held that a number of strong columns (or maybe even a reasonably central shear core see below) could ensure that lateral load mechanisms involve many storeys and that a minority of smaller columns were then acceptable. These smaller columns would have to retain the function of supporting axial loads and so they should definitely be confined (Rule DF4) over their full height.

DF2 No shear failures in ductile beams

Shear failures in beams and in columns should be prevented even if the bending rebar is over-strength, say 130% of the nominal strength (including strain-hardening). Bending failures should be ductile and, indeed AS3600 implies that maximum ductility is obtained, in part, when the neutral axis depth ratio k_u < 0.20. One wonders just how often this is observed in Australian design practice.

Shear failures are often/usually brittle. The implication is that the shear reinforcement should be calculated so as to prevent shear failure even with "over-strength" main rebars. So the shear design is based on the 'capacity shear' provided by the main rebar at 'over-strength', say 130% yield and this should be bigger than the design shear V^* resulting from the factored loads. The factored gravity loads are unchanged but the earthquake moments are to be increased until 'over-strength' yield occurs and shear failure is to be prevented under these circumstances. However the design of shear reinforcement can reasonably, one thinks, use nominal strength $\phi = 1$.

The treatment of shear in 'ordinary' beams in empirical but alas the scatter in plotted data is large: AS3600 Commentary: 1994. Gurley 2011a,b,c has proposed the use of 'exact' (in-plane) yield-line plastic analysis which should reduce scatter but has not yet been correlated with experimental results.

The maximum spacing of shear reinforcement is important when it rules which it often seems to do. A slight variation on AS3600:2009 S8 would require maximum tie spacing of:
Near supports: MIN(300, 0.50*D_{beam})
Near midspan: MIN(450, 0.75*D_{beam})
Near supports means within, say, 2* D_{beam} of a support face where:
D_{beam} is the overall depth of the beam.

There is a problem for wider beams. AS3600 c8.2.12.2 requires that *"The maximum transverse spacing across the width of the member shall not exceed the lesser of 600 mm and D"*. Yield-line (in-plane) theory for shear wants to use the vertical ties to collect the vertical component of the arch re-actions that want to be hung back up to the top chord. The main rebars carry the horizontal component of arch compression. Gurley 2011c. AS3600 seems to permit excessive cross-section spacing. NZS3101 and/or ACI-318, as one remembers, require vertical tie-legs on every second main rebar. This does seem more appropriate at least near supports!

The minimum smeared yield strength of the shear reinforcement in AS3600 S8 is $rf_y = \left(\frac{A_v * f_{sy}}{b_w * s} \right) \geq 0.35$ MPa which rarely governs in Australia because the smallest available rebar with 'Normal' Grade ductility is N10. New Zealand provides smaller 'E' Grade bars. Be aware that 0.35 MPa is a very low tensile strength and should be higher!

DF3 Bottom rebar through supports

Properly anchored and/or lapped bottom rebar through supports is useful as compression rebar for negative (tension top) moments (and thereby improves ductility) but essential if support/a column is lost due to an earthquake or other extreme event. The bottom rebar is then in tension and may support catenary action after the support is lost, perhaps thereby avoiding a catastrophic collapse that is 'progressive' and 'disproportionate'. The ASCE expert committee on the Oklahoma City bombing identified the lack of bottom rebar at supports as one of a few principal causes for that catastrophic collapse. See also the note about diagonal trimmer bars under DF7.

DF4 Confinement of columns

The concept of confinement is most easily explained by reference to a circular column with spiral reinforcement. The cover shell outside the (outside face of the) spiral spalls at strains above about $e_{cu} = 0.004$. If the spiral is properly anchored (with a hook into the core embracing a longitudinal rebar plus some extra turns) then tensile strains in the spiral increase rapidly after spalling thus increasing the concrete axial strength inside the core. Park & Paulay 1976 attribute their equation (2.3): $f'_{ccore} = f'_c + 4.1 * f_{lateral}$ to researchers at the University of Illinois. $f_{lateral}$ is the effective confining pressure given by the obvious equation: $f_{lateral} = \dfrac{2 * A_{sspiral} * f_{sy}}{D_{core} * s_{spiral}}$ exerted by the spiral on the core and s_{spiral} is the pitch or spacing of the spiral. This should make it possible for the increased core-strength to replace the squash load strength lost from the spalled cover-shell and this, one understands, is the current basis for the NZS3101 requirements for spirally reinforced circular columns.

There is no doubt that spirally reinforced circular columns can be very ductile. See the photograph in Park & Paulay 1976 page 565. The then, brand-new, 5-storey Olive-View Hospital, San Fernando 1971 had a soft bottom-storey with permanent sway after the earthquake clearly exceeding the column diameter but had not collapsed. As one remembers, the only (one or two) deaths in this building were caused by the loss of electric power to medical equipment.

The pitch of spirals normally varies 40 – 100 mm which is, indeed, close. The lower limit is to do with preventing segregation of the concrete as the aggregate moves through the spiral from the core to the shell during casting. The use of hard-drawn wire for spirals is definitely not appropriate. Strains in the spiral are expected to increase dramatically immediately after spalling of the cover shell and to exceed yield strain $\boldsymbol{e_{sy}}$.

For rectangular columns, the confinement provided by rectangular ties does not, according to research, significantly increase the strength f'_c of the core. Such columns are much more frequent than spirally-reinforced circular columns although, alas much less robust. In this case Park & Paulay Fig 2.19 suggest that confinement ties be determined so as to provide a shallower slope to the falling branch of the stress:strain curve. NZS3101 still seems to be following that approach.

AS3600 S10 did not require confinement until AS3600:2009 and then only for $f'_c \geq 60$ MPa. High-strength concrete is much more brittle than lower strength concrete. There is a case for high-strength concrete columns in Australia because rents are charged on nett lettable area but, then, one would have thought that there is also an equal case for structural steel columns.

AS3600:2009 has sketches and the obvious static equilibrium equations for calculating the effective confining pressure. This is good! However it does seem unaware of the distinction between spirally-reinforced circular columns on the one-hand and rectangular columns with rectangular ties on the other. The AS3600 notation is $f_{lateral} = f_{r.eff}$. AS3600 requires lateral reinforcement to provide a confining pressure of $\dfrac{f'_c}{100}$ which does, indeed seem quite low. The spacing of confinement ties can often be less than 100 mm usually just for confined regions of about 1.5*D at the top and bottom of each storey but should be full clear-height for columns that are weaker than attached beams as described in DF1.

There are also provisions relating to buckling-restraint for the main vertical rebars. Hysteretic yield of the main rebar will result in rounding of the hysteresis loops due to the ‘Bauschinger effect’ which implies a reduction in Young’s modulus and the buckling capacity of the main rebar. Typically, in New Zealand this maximum spacing is about 6 to 8 * the diameter of main rebars and this limit may apply over the full clear height.

DF5 Shear reinforcement in columns

The maximum spacing of shear reinforcement in columns will be more than the spacing for confinement and so this issue is unlikely to arise for weaker columns that are confined full-height. For strong columns it may arise in the unconfined region at the mid-height of each storey.

This writer does not see any reason why the maximum spacing of ties in columns should exceed that permitted as shear reinforcement in beams. See DF2. The ASCE expert committee identified excessively wide tie-spacing in columns as another of the key few items responsible for the catastrophic collapse at Oklahoma City.

This author has not (yet?) given thought to the exact plastic (in-plane) yield-line analysis of columns in shear. It will be different from that for beams in that the shear usually remains pretty-well constant over the full storey-height.

DF6 Column splices

Typical column splices in New Zealand are generally at mid-clear storey height while those in Australia are immediately above floor-level.

Also column splices in New Zealand are always full-tension splices whereas splices in Australia may be full-compression splices which are typically about 60% of full-tension splices. If a column below has been lost for any reason then the splices above that level will want to be full-tension splices. This was also one of the key issues in the Oklahoma collapse.

Note also that Australian practice does use proprietary ‘splice sleeves’ (AS3600 c10.7.4.4) that rely on direct compression bearing and such sleeves will have zero tensile strength.

DF7 Panel zones at joints

The ‘panel-zone’ is American terminology for the volume where beam and column outlines overlap. If the beams are in both (N-S and E-W) directions then this overlap is three-dimensional. Shear forces in panel-zones are high because the beam rebars will yield in tension on one column face and, in compression on the other. The difference is a horizontal force acting as shear to the panel-zone. Australian structural steel designers have long known about this but Australian concrete designers may have tended to ignore it.

AS3600 S10 has little mention of panel-zones; indeed it takes the view that beams to all four sides might confine the panel zone but, of course, under major horizontal loads, these same beams deliver the large shear forces. For joints at which moment is transferred to the column, AS3600 requires only shear reinforcement with a smeared yield strength as above: $rf_y \geq 0.35$ MPa which is very low. It does seem that that AS3600 c10.7.3.5 has not adequately considered major horizontal loads: wind and earthquake.

NZS3101 and ACI-318 have extensive coverage of shear in panel zones, which, as for shear elsewhere, is empirical. NZS3101 and ACI-318 often require closely spaced horizontal ties and that may, in many/most cases, be the most effective method. It is often true that the shear in the joint panel-zone will be a good deal heavier than that in the clear height of the column and require heavier ties. It is however significant to remember that the principal tensions and principal compressions in the panel-zones are of diagonal orientation at about 45 degrees reversing under hysteretic loads.

Fig 1 shows a first approximation to a panel-zone mechanism taken from the author's lecture notes and is, so far as he remembers, original. It is a first approximation in that, it assumes that beams and columns have the same cross-section and that the axial load on the column is negligible. The author believes that it would be useful to show such mechanisms to civil engineering drafting apprentices as a contribution to their qualitative understanding of rebar detailing issues. It does suggest that diagonal trimmers at 45 degrees through the panel-zone corners tying beams to columns might be somewhat more effective than some proportion of the panel-zone ties but this is not yet perceived wisdom in New Zealand or America.

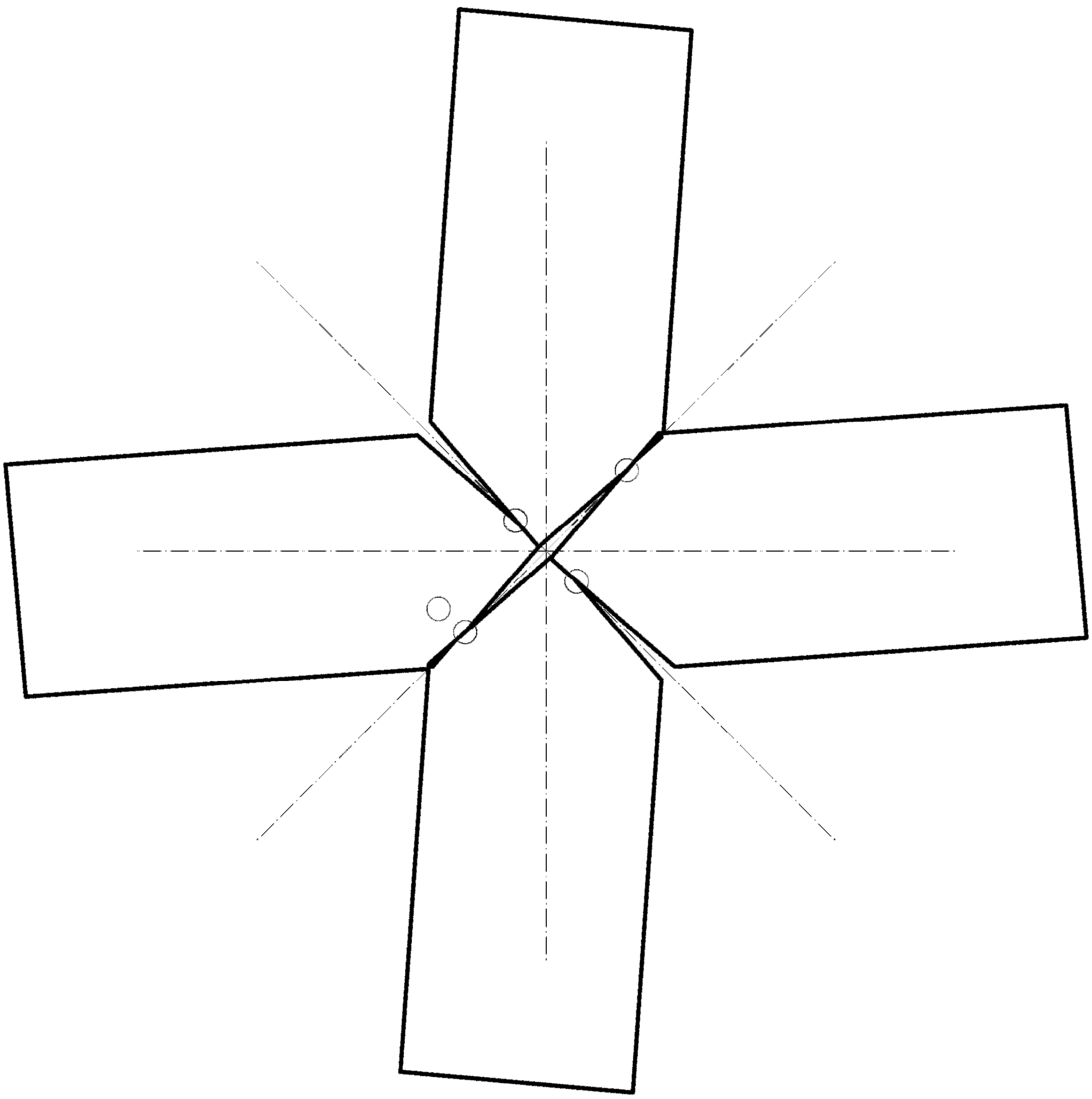

Fig 1: Panel-zone mechanism

Perceptions of earthquake risk in New Zealand and Australia

The previous large-town event in New Zealand comparable to Christchurch 2011 was the Hawkes Bay earthquake of 1931. One is not quite sure how advanced telegraphy was at that time but, certainly, it would have taken some weeks for sea-mail to reach Australia and, by then, it would have been 'old' news. Now live telecasts of the damage in Christchurch were on our Australian screens within minutes. This instant news may change the perceptions of our clients and of their tenants.

It is certainly true that clients, tenants and the public in New Zealand are now much more interested in earthquake resistance than they were just before the recent Christchurch events. Who can say that this will have no effect on the perceptions of Australian clients and tenants or, indeed, on those of other countries around the Pacific 'Rim of Fire' for whom Australian consulting engineers design many buildings?

Differences between practice in New Zealand and Australia

If Australia is to take advantage of New Zealand experience then it needs to be aware of historic differences in practice between the two countries. The most obvious of these is that, relative to the size of each country and its building industry, New Zealand is much more prone to and comfortable with precast concrete as compared with in-situ concrete. This became clear after the author went there in 1972 and it probably had not changed much prior to the recent Christchurch events.

Australia, in the building boom of the 1960s, went the other way and invested heavily in sophisticated formwork systems. It still seems to do so. Australia does have a significant precasting industry but it seems, in relative terms, smaller than that in New Zealand.

Precast concrete can, perhaps/probably, be as earthquake resistant as in-situ concrete but it is certainly more difficult to make that happen. Two examples from the recent Christchurch earthquake:

(1) New Zealand precast floors with pre-tensioned planks factory precast in long beds and cut to length, did not (at least until recently) require any bottom starter rebars. The pretension cables have just a bond-length of, say, 10-20 mm at the support. The bottom rebar at supports is thus effectively zero. The question is whether such floors suffered disproportionate damage when compared to an insitu floor?

(2) Precast fire-stairs were common in Christchurch and collapsed in several buildings because of excessive movement at non-integral supports where the flights were supported on projecting steel plates. This left the occupants to be evacuated by ropes, cranes and helicopters. One is not sure how these precast flights sitting on steel plates satisfied fire-rating requirements.

A more important example is next.

Shear-walls and shear-cores

The term 'shear-wall' is common in New Zealand but one rarely hears of 'shear-cores' as one does, universally, in Australia. The difference is most easily explained in terms of Vlasoz 1961 *Thin-walled elastic beams*: an 'open' cross-section is one in which there are no closed cells. A 'closed' cross-section is one in which there is one or more closed cell(s).

Closed sections have much larger torsional strength and stiffness than open sections because the latter rely on warping, for example, of the opposing flanges of an I-beam whereas the former carry torsion effects by shear-forces that wrap around the closed cells.

Vertical shafts in buildings (fire-stairs, lifts and air ducts) have to be surrounded with fire-rated walls so as to prevent the rapid spread of fire vertically up a building. In New Zealand these fire-rated walls are mostly non-structural gypsum board on steel-studs as, indeed, they were also in the World Trade Center 2001. Structural concrete walls are a minority, if any, of these fire-rated walls. In Australia these firewalls are usually 100% structural concrete.

NZ shear-walls are usually ‘open’ in the sense that they are just elongated columns or, perhaps, T- or L-sections. It seems obvious that the purpose of this is to minimise formwork costs. Australian shear-cores are almost always closed. They vary from:
(1) Several distinct single-cell rectangular sections around individual fire-stairs and lift-wells in, say, an apartment building up to, say 20 storeys up to
(2) Large multi-cell sections surrounding fire-stairs, lifts, air-suppply and air-return ducts in office-buildings plus perhaps rooms (kitchens and toilets) requiring access to plumbing. Fire rules prohibit the spread of fire from one cell to another and so all of these cells must be separated from each other by minimum 200 mm walls with N12 minimum rebars each way each face.

Australian shear-cores are often 5 – 10 or 20 metres in size in one or both plan directions.

One remembers the 1960s when this seemed the right way to go perhaps because of Australian expertise developed in the 1930s for the construction of large grain-silos at most rural rail-heads and, in Sydney, next to the (later) Anzac Bridge.

Most cells in shear-cores have openings for doors and horizontal air-ducts. Nevertheless there will be coupling-beams across these openings being the residual piece of wall above door-head up to the next floor-level. These need to be carefully designed for shear and that might determine the wall thickness. There will be some reduction in torsional strength and stiffness but usually much less than that to reduce them to ‘open’ sections. Park & Paulay 1976 Chapter 12; Gurley 2011b.

This also relates to the precast stair issue. Australian fire-stairs are surrounded on 4 sides by a closed in-situ core wall not less than 200 thick. The stairs are poured inside the core after the slip-form for the core has gone past. This is not quite the same as poured integrally and one does need to be careful about starter-bars from the core to the landings and from the core to the surrounding floor slab but it is probably better than the New Zealand articulated precast connections not least because the core stiffness will limit sway deflections to lower, perhaps much lower values.

In the late 1980s, there was a supplier in Adelaide who constructed precision permanent left-in-place formwork for stairs using cold formed steel about 1 mm thick. It seemed like a good idea then and probably it still is in terms of formwork costs.

For these reasons, past New Zealand discussion of ductile shear cores has been limited and the following list may therefore be somewhat more contestable.

AUSTRALIAN CHECK LIST ‘DSC’ FOR DUCTILE SHEAR-CORES

See Park & Paulay 1976 Chapter 12. This list seems fairly correct to this writer but, in New Zealand it may have nothing like the wider acceptance of the ‘DF’ list. On the other hand it does use shear-cores which is consistent with existing Australian practice.

DSC1: The building as a whole should (1) not contain any brittle structural elements and (2) all of the fire-rated walls should be structural concrete. Good robust buildings should be a combination of ductile frames and ductile cores all of which are both ductile and robust. Earthquakes and other extreme events are unpredictable things and any notion of some brittle structural elements within an otherwise robust building seems quite wrong. Shear-cores provide stiffer buildings and therefore tend to minimise non-structural damage. This seemed obvious after the San Fernando 1971 earthquake and may have occurred again in Christchurch 2011.

DSC2: For taller buildings, a ductile cantilever bending failure of the core as a cantilever with again $k_u < 0.20$. This may require thickening of perimeter walls and/or some largish columns at the corners of each core. Think about diagonal attack!

Shear failure may be inevitable in some low-rise buildings but they will naturally be much stronger simply because they are low-rise.

DSC3: First yield will often occur in core coupling-beams and spread, at any one location, up and down from there. The shear reinforcement should be sufficient to carry all of the ‘capacity design’ shear-force across a diagonal bending yield-line steeper than the clear-span diagonal. Park & Paulay 1976, Gurley 2011b. The DF rules for ductile beams apply.

DSC4: The pieces of wall between core-coupling beams will then need to be checked/designed for the corresponding axial forces and moments. This does include any attached flanges and webs. It also includes confinement as in DF4.

DSC5: Those same pieces of wall then need to be checked for shear strength along the lines suggested by the DF5.

DSC6: Shear-core buildings will often be much stronger and stiffer than ductile frame buildings so they are more likely to involve rocking foundations. A raft foundation, say a metre thick or more might might be appropriate for tall buildings on soft sites.

Analytical software

The author will leave description of analytical software to others. He is intrigued that there is still no software that will predict simple rigid-plastic collapse mechanisms in 2 or 3 dimensions unless those mechanisms are known to the programmer in advance. The author has no doubt that this conclusion follows from a huge but unsuccessful research effort but such efforts tend not to be well reported. The author is perversely pleased that the human mind retains some superiority over automated software but, alas the result of that is that analytical software has had to go to algorithms that are much more complicated.

Nigel Priestley’s suggestion of a forced displacement analysis seems to be a reasonable step in a logical direction but alas it still contains some arbitrary rules as to the displaced shape up the height of the building.
Perhaps experts in this area should be asking, say, …

- “What is the displaced shape that, for a given displacement at roof level, will maximise the beam hinge rotations at, say Level 30” or alternatively …
- “What is the distribution of inertial forces up the height of the building that, for a given displacement at roof level will maximise beam hinge rotations at, say Level 30”.

The author last studied mathematics at university level more than 50 years ago but his memory is that such questions are addressed by the subject: 'Calculus of Variations'. Maybe we need some more advanced mathematical thinking in this area now rather than just guessing! Over the same period, computers have improved so that a laptop is now more powerful than the size-of-several-houses UTECOM was at UNSW 50 years ago. The use of advanced mathematics as an engineering tool is now more appropriate and more accessible.

If this approach is fruitful then the next question will be: "Are these distributions the same for Level 31, for Levels 20 – 40 and when and why do they differ". Answers to these questions are really the only way of expanding our knowledge of structures and how and why they respond to earthquakes and other extreme events.

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