Protecting life and reducing damage in earthquakes and terrorist attacks

Colin Gurley

Abstract

Images of structural collapse crowd our globalised television screens; from wars, from terrorist bombs and from earthquakes. The outcomes often look quite similar, but what are the engineering differences?

As engineers, we look for differences relating to collapse mechanisms:

- Earthquake engineering focuses on sway collapse mechanisms in which the building as a whole moves sideways and may collapse under its own weight.
- Explosions may remove one or several load-bearing columns or walls leading to a 'lost support' vertical collapse which may be:
    - localised to the lost support perhaps extending over just a few stories or
    - 'disproportionate' or 'progressive', extending over much of the floor area and, perhaps, the full height of the building.

Records of earthquake damage show that earthquakes can also remove supports, often corner columns, precipitating a vertical collapse which may, again, be 'progressive' and 'disproportionate'. It seems logical and appropriate, in terms of the expertise and technology involved, to regard 'robust' design for 'lost column' events as an extension of earthquake engineering and so to merge earthquake engineering with design against 'disproportionate' collapse.

A further motive for this merger arises when considering the quality and details of ductile detailing to be provided in regions of low earthquake risk such as Australia.

Introduction

Structural thinking has developed in response to the 1968 Ronan Point accidental gas explosion, the 1995 terrorist bomb attack on the Murrah Federal Building in Oklahoma City and the 2001 attack on the World Trade Centre. It now seems evident that there is potential for two areas of engineering to merge in the development of robust standards.

The drafts of AS 1170.4 Earthquake actions and AS 3600 Concrete Structures already cover many of the issues relating to 'lost column' events.

Some matters may require review. One such issue is that:

- these two draft codes permit three different standards of ductile detailing:
    - 'ordinary', 'intermediate' and 'special' and
- they do not discourage 'ordinary' detailing indeed
- they rather encourage 'ordinary' detailing in the sense that the only advantage of 'intermediate' and 'special' detailing apparent from these drafts is a lower design earthquake load. Even this 'advantage' will be ineffectual if the earthquake design load effect is, anyway, less significant than the wind design load effect as it often is, particularly for taller buildings in Australia.

This article takes the view that the minimum standards of ductile detailing should be related to the Importance Level of the building as defined in BCA. The rationale for this view relates both to earthquake performance and also to performance following removal of a support by any event.

The calculation of earthquake design load can then take advantage of the standard of ductility actually provided but a higher earthquake design load should not be used to justify 'ordinary' detailing for important buildings.

The concept of ductile detailing involves a whole basket of design rules but the most obvious, perhaps the most crucial, issue has to do with the continuity of bottom rebars through columns and other intermediate supports. Other important issues relate to secondary reinforcement for shear strength, for confinement and for buckling restraint of compression rebars.

Bottom rebar at columns and other intermediate supports

The Draft of AS 3600 c8.1.10.4 has not changed from earlier editions:

- At an intermediate support "… not less than 25% of the total positive reinforcement required at mid-span shall continue past the near face of the support." No distance is specified so it could be as little, say, as 50 mm.

The 25% required can have negligible anchorage at the column face so the bending strength for any positive moments there will that of an un-reinforced beam hence brittle under cyclic load.

Australian 'ordinary' detailing is somewhat inferior to American 'ordinary' detailing. In ACI 318-05:

- c12.11.1 requires the same 25% but is explicit that it extend 150 minimum into the support while:
- c12.11.2 further requires that:
    - "When a flexural member is part of a primary lateral load resisting system, positive moment reinforcement required to be extended into the support by 12.11.1 shall be anchored to develop f_y in tension at the face of the support."

The ACI Commentary explains that this "… anchorage is required to ensure ductility of response in the event of serious overstress such as from blast or earthquake." Indeed it is although it would be better to make bottom rebars continuous through columns.

Support bottom rebars with wind

The worst design positive (tension bottom) bending moment at a support will be the signed sum of:

- A negative (tension top) moment caused by the minimum long-term gravity loads offset by
- A positive (tension bottom) moment caused by the ultimate (say 500 or 1000 year) wind loads.

If the negative gravity load moment exceeds the ultimate wind moment, even slightly, then a nett positive moment never occurs and 'ordinary' detailing to AS 3600 does not require any effective bottom rebars at the column.

What if an abnormal wind load occurs? Calculation from AS1170.2 Table 3.1 shows that the ratio of wind pressures from 10,000 year return to 500 year return varies from 1.3 to about 1.5 in some cyclone areas. If one neglects global warming then there seems to be an upper limit on wind pressures such that, even at 10,000 years return the pressure, hence bending moment, from wind is, at most, 50% larger than the 500 year value.

This is not quite an acceptable answer. It does imply that:

- If the moment due to 500 year wind is exactly balanced by minimum gravity loads then
- Under the 10,000 year wind the beam will experience a nett positive (tension bottom) curvature equal to half of the gravity load value at a position with no bottom rebars.

This is not just a 50% overstress on steel rebar under extreme conditions, which would be acceptable. Rather it relies on the tensile strength of concrete for a load which is not cyclic but certainly erratic and pulsating. Nevertheless our code writers have accepted it and there may be reasons not least, in that, whatever happens to an individual beam is not likely to happen to the building as a whole so long as the 'overload' is just 50%.

Support bottom rebars with earthquake

The position with earthquake is a good deal worse than with wind for 2 reasons:

- Earthquake loads are not limited with increasing return period to anything like the extent that wind loads are so limited and
- this is compounded by the combined ductility factor of Draft AS1170.4 Table 6.2.

In terms of the ratios of 'Probability Factors':

- Draft AS 1170.4 Table 3.1 shows a ratio of earthquake loads 2500 years/500 years at 1.8 and
- NZS1170.5 shows the same value, 1.8 for 2500/500 years but
- NZS1170.5 Commentary Fig C3.3 shows ratios up to 2.9 for 10,000/500 years (at Dunedin)

In terms of ductility factor, the design earthquake load with just 'ordinary' detailing will have been reduced to just 38% (1/2.6) of the value for an elastically responding structure.

The effect of this is that:

- The 10,000 year response is not just 50% greater than design load response as with wind
- It may be greater by a factor 2.6 x 2.9 = 7.5 and
- The load will be cyclic

If the positive moment curvature at an intermediate column under the design earthquake load is just sufficient to counterbalance the opposite curvature due to minimum long-term gravity load then:

- AS3600 'ordinary' detailing permits zero effective bottom rebars at the column but
- The nett curvature at that un-reinforced section under the 10,000 year event may be 6.5 (=7.5 – 1) times the gravity load curvature.

One normally assumes that ductility provides the basis for reduced return periods and reduced design loads but this does seem to be increasingly questionable when the notion relies on beams with no effective bottom rebars at columns.

This does apply to important buildings including tall buildings and all facilities at BCA Importance Levels 3 and 4 including hospitals, police, ambulance and fire stations and chemical factories producing hazardous materials.

Support bottom rebars after loss of column

If a column in storey x is removed by any means (explosion or earthquake) then the effect is that:

- every floor structural element supported by that column over the full-height above storey x,
- will experience a sudden doubling of its span but,
- with 'ordinary' detailing,
- will have zero sagging strength at what will now become a critical mid-span region.

The total free-span moment strength at a typical floor will have been reduced by about one third in each new half-span. More importantly, a flexural failure at the new mid-span will be brittle so the span, as a whole, may be unable to develop its full plastic mechanism strength.

Floors cannot 'hang' from higher levels unless there is some non-typical stronger structure (or lower gravity loads) at those higher levels. If the total strength over the height of the building is inadequate for the total loads then a progressive collapse will follow.

Note that the better 'ordinary' detailing of ACI 318-05 c12.11.2 does not really cover the 'lost column' situation either. The bottom rebars do need to be continuous through the column.

Design gravity loads under these circumstances are discussed below in the paragraph on GSA 2003.

Britain & Ronan Point

In 1968, at Ronan Point in England, a 20-storey apartment building, of precast concrete construction, was destroyed by an accidental explosion in a domestic kitchen gas stove. After a judicial enquiry, the British Government adopted the 'Ronan Point Rules' for building design, of which the current version is Approved Document A3 2004 *Disproportionate Collapse* downloadable at www.odpm.gov.uk.

Precast concrete construction has moved on since to the extent that New Zealand and California now both have specialised codes for precast construction in regions of high earthquake risk.

Britain has been subject to terrorist bomb attacks for several decades and yet the 'Guidance' documents at the UK website make it clear that, while the writers were aware of terrorist threats, they were (2004) nevertheless primarily focused on accidents: gas explosions and vehicle impacts.

Australian states adopted some of the 'Ronan Point Rules' a few years after the Ronan Point event and there are still vestiges in BCA and in AS1170.0 s6 *Structural Robustness*. These guides are general and not at all specific.

The basic Requirement of British Document A3 is that:

- *"The building shall be so constructed so that in the event of any accident the building will not suffer collapse to an extent disproportionate to the cause.*
- *Subject to the "Limits of Application*
- *"Requirement A3 applies only to a building of 5 or more stories"* counting basements but not roof space if pitch < 70 degrees.

There are still 3 distinct groups of provisions in the British provisions:

- Design of structural elements for blast pressures
- Tie every element to every adjacent element
- Alternative load-path analysis

This list, identified almost 4 decades ago, still covers the important issues arising in 'lost support' events.

Ronan Point 'ties'

The British emphasis is on 'ties' rather than 'ductile detailing'. From an earthquake engineering view:

- Ductile detailing is always important while
- Ties are vital but not often important because they are anyway, in many/most cases, provided by an in-situ floor-slab as diaphragm, and by continuous vertical rebars or by splices of steel columns.

In in-situ concrete-framed buildings, floor rebars will be passed through, or close to, the rebar cages of concrete columns. In steel-framed buildings, rebars will flank 'I' and 'H' columns close on all 4 sides.

One situation that does require care is that of a façade steel column placed close to or outside the edge of the floor-diaphragm. This is already covered by AS4100 c6.6 and c5.4.3;

- a horizontal buckling restraint force of 2.5% of column axial load
- or 2.5% of flange forces if there are also substantial column moments and
- the crucial 'ties' must prevent detachment of the column from the floor-slab as diaphragm.

If the buckling restraint force cannot be supplied by rebar then it must be supplied by steel-to-steel connections (bolts or welds). Construction sequence also needs to be considered; what loads are on the column before the floor-slab is 7 days old?

Ronan Point design of structural elements for blast pressures

is one of the 3 areas covered by the Ronan Point Rules. My guess is that this is/will be a specialist activity for those who design embassies, defense establishments and similar where car/truck bombs can be kept at a distance.

The British document A3 mentions a design blast pressure of 34 kPa (710 psf). FEMA 227/ASCE 1996: The Oklahoma City Bombing calculates a blast pressure of 10,000 psi (69 MPa = 69,000 kPa) at mid-height (smaller pressure above, bigger below) of the crucial 2-storey transfer column located 5 metres from the centre of the bomb.

It is difficult to believe that blast-pressure design can save such a column.

Ronan Point alternative load-path (removed support) analysis

is another of the 3 areas of the Ronan Point Rules and an area that AEES can and should address.

Load-bearing elements are:

- Individual columns or
- Any length of load-bearing wall of length between piers but ≤ 2.25 H (H = clear storey height) or
- Transfer structures supporting 2 or more columns

Any such element not designed for a blast pressure of 34 kPa must be removable without causing 'disproportionate' collapse exceeding:

- 15% of floor area or 70 sq metres (700 sq feet) whichever is less
- On each of, at most, 2 adjacent floors

GSA 2003 (see below) now also covers this area and has requirements for new (US) Federal Buildings involving the notional removal of edge, corner or interior columns or load-bearing walls in the bottom storey only.

The Murrah Federal Building in Oklahoma City

did have concrete transfer beams on the long north facade at 2 stories above ground doubling the façade spans from 6 to 12 metres.

From detailed analysis FEMA 227/ASCE 1996 concludes that:

- Blast pressure directly failed less than 10% of the total floor area. Floor-slabs are normally detailed only for downward loads and one would expect that these floor-slabs were not reinforced for upward loads; hence no mid-span top rebars and no continuous bottom rebars through supports. The failed areas were presumably close enough to the bomb that upward blast pressure exceeded slab self-weight and tensile strength.

- Blast pressure ‘brisance’ removed north façade column G20 supporting the transfer beam and resulting, momentarily, in a 24 metre span from G16 to G24.
- Columns G16 and G24 then failed in shear under the blast pressure,
- resulting in a 48 metre total span with no effective mid-span bottom rebars thereby progressively bringing down 50% of the occupiable floor area over the full height of the building.
- *“… up to 90% of the (168) fatalities were the result of crushing caused by falling debris”* as distinct from the bomb explosion and the blast pressure.
- The collapse started 2 seconds after the explosion and took a further 3 seconds.
- *“Column G20 would be likely to have been destroyed by brisance even if detailed as a Special Moment Frame.”*
- *“If the more recently developed detailing for Special Moment Frames had been present at the time of the blast, Columns G16 and G24 would have had enough shear resistance to develop a mechanism without failure.”*

It is clear from FEMA 227/ASCE 1996 (and not surprising) that the 1974 design did not comply with the 2005 edition of ACI 318-05 c12.11.2 on anchorage of bottom bars from a column face. Perhaps there was no corresponding clause in ACI 318-71 and/or perhaps the north façade was not considered to be *part of a primary lateral load resisting system* given that there was a shear core near the centre of the south façade and 4 large vertical (apparently structural concrete?) air-ducts at the 4 corners of the building.

The overview findings from FEMA 227/ASCE 1996 were that:

“It is noted that the loss of 3 columns and portions of some floors by direct effects of the blast accounted for only a small proportion of the damage. Most of the damage was caused by progressive collapse following loss of the columns.

“There is no evidence to suggest any significant lateral or torsional displacements of a global nature..

“The type of damage that occurred and the resulting collapse of nearly half the building is what would have been expected for an Ordinary Moment Frame Building of the type and detailing available in the mid-1970s when subject to the blast from the large bomb that was detonated.

“Special Moment Frame design would provide (presumably bottom) reinforcement in the transfer girder at the third floor that would greatly increase the possibility that the slabs above would not collapse” and “… it is estimated that losses would be reduced by as much as 80%.” Note that American third floor = Australian second floor.

“The engineering analysis performed on the Murrah Building suggests that the higher lateral force levels required for seismic design in areas of high seismic risk are not required for blast mitigation in regions of low seismic activity; only the detailing requirements need be followed” and

“Investigations to determine the cost of using Special Moment Frames rather than Ordinary Moment Frames were conducted by the Building Seismic Safety Council. These investigations along with more recent changes in designs, suggest that increase in cost is in the range 1% to 2% of the total construction cost of the building.”

The Murrah Federal Building was a 9 storey (35 metres) high building with an area of 1400 sq metres per floor total 12,600 sq metres in a region with no earthquake design requirements at the time of construction. An identical building in Australia would barely qualify as BCA Importance Level 3.

America now

In America there is ongoing debate in the light of the attack on the World Trade Centre 2001 and the 1995 attack in Oklahoma City. There are initiatives on the US East Coast to amplify and adopt the Ronan Point Rules in more voluminous detail.

The British writers of Document A3 have written what they have in the context of a country that has a lower level of earthquake risk (than Australia, New Zealand and America) and a lower level of awareness in the general population and in the home building industry. There are British firms of consulting engineers practicing internationally who are expert at earthquake engineering but they are not the typical home readership of Document A3. Australian consulting engineers often have experience in neighbouring Asian countries where earthquake risk is high.

There is a danger in writing separate codes for 'lost support' terrorist events and for 'progressive collapse' that ignore but largely overlap earthquake codes in earthquake-aware countries thereby adding to the un-necessary length and complexity of codes and to the risk of mis-interpretation.

The American debate is also concerned to protect emergency responders and there are suggestions that some buildings be designed to survive 'burn-out' of contents after active fire suppression systems fail. 'Intermediate' or 'special' ductile detailing might also contribute to this worthwhile objective.

GSA 2003 Progressive Collapse Analysis and Design Guidelines for New Federal Office Buildings

This US Federal document provides extensive direction on the details of lost support /double-span mechanisms.

It recognizes blast-pressure design as a specialist activity and seems to accept that it may only be useful when unauthorized vehicles can be kept outside a 'defended stand-off distance'.

It has some helpful illustrations of double-span mechanisms in concrete and in structural steel.

It provides the first American version of the British rules for *alternative load-path analysis*:

- Supports may be lost only in the lowest storey at about ground level
- Any single perimeter column may be lost
- Any single 9 metre (30 feet) length of perimeter load-bearing wall may be lost
- Any single interior column or any single 9 metre length of interior load-bearing wall may be lost if there is uncontrolled parking beneath the building
- Any corner column may be lost
- Any double-legged (equal angle shaped) portion of load-bearing wall at a corner may be lost for 4.5 metre (15 feet) along each façade or at any interior support if uncontrolled parking
- Under these circumstances collapse should be limited to the immediately adjacent bays of the floor immediately above the removed support but not more than 1800 sq feet (180 sq metres) at a perimeter support or 3600 sq feet (360 sq metres) at an interior support.

This is quite a useful document but it does seem somewhat focused on car/truck bombs. GSA Figure 4.7 shows strengthening at floors 2 and 3 which would be effective if a bottom storey column was lost but not if some column above level 3 was lost. (American floor 3 = Australian floor 2)

Higher level columns might be lost by earthquake or to a terrorist attack using weapons other than car/truck bombs. To accommodate all such unforeseeable possibilities it would seem best to place any special strengthening elements as high in the building as reasonably possible.

GSA 2003 requires double design gravity load but permits doubled strength:

- For static analysis: Gravity Load = 2 * (DL + 0.25 * LL) ; GSA Equation (4.1) and
- Doubled strength: DCR = Demand / Capacity Ratio ≤ 2 for "typical structural configurations"; GSA Equation (4.2)

These appear to be offsetting factors but it is not quite as simple as that:

- In GSA Equation (4.1):
    - The factor 2 is, presumably, an impact factor to account for the sudden removal of a lost support. It seems about right.
    - 0.25 seems to be a reasonable estimate for the long-term (earthquake) component of live-load. It is slightly lower than the figure of 0.30 in the Draft AS1170.4 Earthquake actions. The figure in AS1170.0:2002 *General principles* is still 0.40.
- The DCR = Design to Capacity ratio is also set at 2 for "typical structural configurations". In this regard, GSA 2003 refers to FEMA 356 *Prestandard and Commentary for the Seismic Rehabilitation of* (Existing) *Buildings*

The earthquake literature and FEMA 356 contains extensive material on the over-strength and strain-hardening of steels and this accounts for much of the DCR up to, say, DCR = 1.5 which is the GSA value for "atypical structural configurations".

The value DCR = 2 from FEMA 356 assumes elastic (not plastic) analysis and so also includes an allowance for plastic redistribution. To use this value with capacity calculated from a plastic analysis may involve some double counting.

In any case, one can understand that code-writers, such as the writers of FEMA 356, might reasonably be somewhat optimistic about the strength of existing buildings particularly those that are also heritage buildings. This would be within the public policy domain of code-writers because a value that was not somewhat optimistic would tend to pick up numerous existing buildings that are only marginally under-strength. Should one be just as optimistic in the design of new buildings for abnormal load?

Australia

Records of earthquake damage often give the impression that the quality of ductile detailing is more important to performance in actual earthquakes than the value of earthquake design load.

This does seem to be true for regions designed for high earthquake loads such as New Zealand and California. Perhaps it is even more true for regions designed for low earthquake loads such as Oklahoma City and most of Australia. Earthquake events may far exceed code predictions, especially in areas of low seismic risk as at Newcastle (NSW) 1989, but a ductile building will always fare much better than an otherwise-identical brittle building.

And ductile detailing for bomb attacks is pretty much the same as it is for earthquakes if not always for the same reasons. This is an area that AEES can and should progress.

And we have the opinion from (US) Building Seismic Safety Council that the extra cost of high-quality ductile detailing (as distinct from higher lateral design loads) is so small as to be barely quantifiable. This does make for a strong cost/benefit argument!

There seems to be nothing in the drafts of AS1170.4 or AS 3600 which relate the minimum quality of ductile detailing to the scale and importance of the building. The distinction between 'ordinary', 'intermediate' and 'special' merely results in a lower ductility factor hence more design earthquake load on 'ordinary' frames. If the earthquake load anyway happens to be a low, non-critical load condition compared to wind then there is nothing to encourage the use of 'intermediate' or 'special' ductile detailing. This could apply to tall buildings and to any and all facilities at BCA Importance Levels 3 and 4 such as hospitals, police, ambulance and fire stations and chemical factories producing hazardous materials.

What then about AS 4100 *Steel Structures*? One normally assumes that steel structures are inherently ductile. But it is common practice to design many steel beams for gravity loads with 'pin' supports. If 'pin' connections really do behave as such when a column is lost then those steel beams may be in a worse predicament than concrete beams with zero bottom rebars. Some of the American research following the Northridge Earthquake of 1994 suggests that some, not all, 'pin' connections are better than that and can develop significant moments so long as both flanges have, at least, cleats to the supporting columns or girders.

Conclusion / Recommendation

Earthquake engineering, as a field of professional expertise should be extended to include 'lost column' events whether caused by earthquakes, accidental explosions or terrorist bombs.

Relevant Australian Standards should be reviewed so as to:

- consider ductility under 'lost support' events whether by explosions or earthquakes
- relate the minimum standards of ductile detailing to the Importance Levels of BCA

In particular, the draft of AS3600 Clause A12 *Moment Resisting Frame Systems* should be urgently revised to require 'intermediate' and/or 'special' detailing as minimum standards for buildings at BCA Importance Levels 3 and 4 particularly for tall buildings and for emergency response facilities such as hospitals, ambulance, fire and police stations and utility services.

One has to ask whether Australians would find it acceptable, in the light of what we now know and in the current geopolitical climate, for a newly built building in Australia:

- comparable in size and importance to the Murrah Federal Building in Oklahoma City
- to suffer the same degree of loss of life and collapse
- as the result of a truck bomb placed by a group of 3 terrorists and
- as a now foreseeable consequence of 'ordinary' detailing?

Particularly when the marginal cost of a much improved 'intermediate' or 'special' performance is so low as to be barely quantifiable (1% to 2% of total cost).

These issues also need to be addressed in AS 4100 *Steel Structures*. Many steel buildings are designed with 'simple pin' connections for beams. These may also be brittle under 'lost support' events.

In the longer term, we should, with our colleagues around the world, start to develop a systematic understanding of the collapse mechanisms associated with the loss of supports (columns, load-bearing walls and transfer systems) comparable to our understanding of the sway mechanisms more usually associated with earthquake.

A collaborative approach is likely to achieve more robust solutions and more cohesive structural codes, reducing the complexity of regulatory documentation and improving outcomes for engineers and for the community.

References

ACI 318-71 Building Code Requirements for Reinforced Concrete 1971, American Concrete Institute, Detroit, Michigan, USA

ACI 318-05 Building Code Requirements for Structural Concrete and Commentary, American Concrete Institute, Farmington Hills, MI 483312, USA

Approved Document A3 2004 Disproportionate Collapse; The Building Regulations 2000, Office of the Deputy Prime Minister, London downloadable from www.odpm.gov.uk

AS1170.2: 2002 Structural design actions Part 2: wind actions, Standards Australia, Homebush NSW 2140

AS1170.0: 2002 Structural design actions Part 0: General Principles, Standards Australia, Homebush NSW 2140

AS 4100: 1998 Steel Structures, Standards Australia, Homebush NSW 2140

BCA Building Code of Australia, Australian Building Codes Board, Canberra

Draft for Public Comment DR 04303 2004: Revision of AS1170.4: 1993 Structural Design Actions Part4: Earthquake Actions in Australia; Standards Australia, Homebush NSW 2140

Draft for Public Comment: Revision of AS 3600: 2001 Concrete Structures, Standards Australia, Homebush NSW 2140

FEMA 222A / Building Seismic Safety Council 1994/5 NEHRP Recommended Provisions for Seismic Regulations for New Buildings, Washington, DC

FEMA 227/ASCE August 1996: The Oklahoma City Bombing: Improving Building Performance Through Multi-Hazard Mitigation. Federal Emergency Management Agency, Washington

FEMA 356 Prestandard and Commentary for the Seismic Rehabilitation of Buildings, Federal Emergency Management Agency, Washington

FEMA 403/ASCE May 2002: World Trade Center Building Performance Study. Federal Emergency Management Agency, Washington

GSA June 2003 Progressive Collapse Analysis and Design Guidelines for New Federal Office Buildings, General Services Administration, Office of the Chief Architect, Washington

NZS 1170.5: 2004 Structural Design Actions Part 5: Earthquake Actions – New Zealand; Standards New Zealand, Private Bag 2439, Wellington 6020.

NZS 1170.5 Supp1: 2004 Structural Design Actions Part 5: Earthquake Actions – New Zealand - Commentary; Standards New Zealand, Private Bag 2439, Wellington 6020