S G Butcher,
[redacted]
26 March 2025

Sheryn Scanlan,
Planning Technical Support Officer,
planningadmin@mstn.govt.nz

Dear Sheryn,

I would like to raise an issue further to the error that is the subject of the Minute of the Commissioner dated 25 March.

Other errors exist, although not in the costings report to the Masterton Distict Council but in the structural reports that form the basis for deeming the Town Hall/Municipal Building (Town Hall) to be earthquake prone and, as such, pertinent to the process leading up to demolition.

I would be grateful if you are able to convey these errors to the Commissioner with the request from me if he would be pleased to include this information within the scope of his resource consent review.

The short of it is that having practiced in the field of architecture and with some experience in this type of building, I do not accept the determination of earthquake prone as anywhere near accurate; it is simply wrong.

I do not have the time presently to set out matters in great detail but hope the following may suffice to raise some awareness of inadequacies in the LGE engineering report of 2016. I apologise in advance for any typos, errors or omissions.

Very briefly:

(1) Soil Loadings:

These were changed from Class C to Class D, effectively moving the goalposts - Class D soils make it much more difficult to meet loading requirements under the Building Act.

The Town Hall has been viewed as receiving the full Class D loadings. This is not the correct way of using these loadings.

I refer to the review by Structural Concepts. On p.16, that report "notes" the Town Hall as being very close to the boundary between C and D soils. This is so understated as to be easily overlooked, no doubt because professional ethics prevent Structural Concepts from criticising a colleague, where professional loyality ranks somewhat higher than any duty to a customer or to the public.

That "note" refers to the fact that the LGE engineer, Michelle Grant, has not interpolated the soil ratings, and she should have. Interpolation should be done so that the rating is close to the actual ground conditions rather than at the extreme end of the D category range.

For example, the Wairarapa Hospital is a very short distance away and is classed as being on a class C soil, not class D. In fact the Town Hall should be on Class C plus a small margin, but not the full Class D loading.

As it is, failure to interpolate has literally condemned an otherwise sound building.

The Town Hall is not at risk of failure on a class D soil as it is not subject to the full Class D loadings.

(2) Loss of Gravity Support:

This has been called up on p15 of the engineer's report and refers in part to the area between the two buildings.

The engineer claims this risk without putting any numbers to it. The two buildings that, if not connected in any way (they are), would move independently to each other in an earthquake: sometimes toward each other and sometimes away.

In my experience buildings of this type are likely to move in the order of 30mm in total. Any stringer must be at least 50mm in thickness as no smaller size is available or allowable for this function. So there is no risk of loss of gravity support under design earthquake loads. In addition the buildings are tied and the amount they can move towards each other is limited, so the landing required on any stringer would be in the order or 15-20mm. In short, the landings as they are have a safety factor of more than 2.0, much more than a post earthquake function requirement for a safety factor of 1.3 for a public building.

(3) Brick and Seismic Load:

The system of reinforced concrete columns and beams with brick infill panels is a proven belt-and-braces construction system with ample redundancy and safety factor.

The engineer's report appears in parts to treat brick as a seismic load, for example as Unreinforced Masonry (URM) on p23. This assumes that brick is loading the concrete structure and ignores the contribution of the brick to bracing and redundancy.

As the portal system flexs and moves, the brick will take up bracing loads. At failure the brick ceases to carry the full load and fractures, leaving the portal system to act alone. In practice this means the brick aids the resilience of the portal system, preventing rapid collapse and making a building inherently safe under extreme failure conditions.

In summary:

1 The soil loadings are not as high as claimed due to a failure by the engineer to interpolate loadings,

2 The brick infill on the ground floor should not be treated as adding to loadings on the portal structure but adding redundancy to the bracing system in a way that provides a considerable safety margin under failure conditions of the reinforced concrete portal structure, and

3 There is no risk of loss of gravity support for the floor between buildings, as the differential movement between the two buildings would be much less than the movement range allowed for on the stringer landing provided.

The engineer's report and subsquent reviews have not adequately addressed these issues. I have brought much of this to the attention of the Council to no effect.

My professional opinion based on my years of experience in the building and design industry and the information available to me convince me that the structural review process is unnecessarily biased by cherry picking numbers or leaving then out altogether, and by a failure to appreciate and understand older but proven safe construction methods.

Stephen Butcher (B.Arch, Dip. BS)

Tonkin+Taylor

Job No: 1001891
29 March 2017

Masterton District Council
425 Queen Street
PO Box 444
Masterton 5840

Attention: Peter Whisker

Dear Peter,

Geotechnical Seismic Assessment - Site Investigation
Masterton Municipal Building & Town Hall

Introduction

Tonkin & Taylor Ltd (T+T) was engaged by Masterton District Council (MDC) to provide a geotechnical seismic assessment and investigation of the Masterton Municipal Building and Town, 64 Chapel Street, Masterton. T+T have carried out this work in accordance with our letter of engagement, dated 30 January 2017.

This letter report supplements our desktop geotechnical seismic assessment[1], completed in March 2017. The purpose of this letter is to:

- Present factual data collected during our site specific investigation
- Reduce the uncertainty of ground conditions assumed in our previous desktop assessment
- Update the register of geotechnical project risks presented in our desktop report
- Further assist Masterton District Council to select a redevelopment scheme, and develop design in the next project stages.

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[1] Tonkin & Taylor Ltd, 27 March 2017, Desktop Geotechnical Seismic Assessment, Masterton Municipal Building & Town Hall. Job ref: 1001891.

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A standpipe piezometer was installed in the borehole following completion of drilling. No water level readings were possible during our time on site (groundwater in the borehole had insufficient time to equalise with adjacent ground). As-built details are provided on the log in Appendix B.

Re-evaluation of geotechnical issues / risks

The desktop geotechnical seismic assessment presented a number of potential geotechnical risks associated with strengthening the existing buildings or redevelopment of the site. The table of geotechnical issues from our desktop report is included in Appendix C for reference.

Based on the data from BH-TT1, we do not propose any changes to this geotechnical issues register. Some additional comments are provided below.

Geotechnical risk	Comments based on BH-TT1 data
Liquefaction	Conclusions of desktop report remain the same. The soils identified in the borehole are unlikely to be susceptible to liquefaction. There remains a risk of localised pockets of liquefaction and the impact should be considered in future design.
Foundation capacity	Assessed bearing capacity of shallow foundations remains unchanged. Localised weak soils may be encountered, however this could be addressed by subgrade inspection and undercutting as required. Assessed vertical capacity of anchors remains unchanged. If anchors are required, further investigation and design will be required.

Summary and conclusions

The investigation data generally supports the assumptions we have made in our desktop assessment report. The conclusions stated in that report, and the suggestions for future stages of work remain unchanged. For clarity, the suggested future work is included again below:

- T+T to issue factual investigation data from site investigation (this letter)
- Project team to select preferred scheme for strengthening and / or rebuilding
- Structural and geotechnical engineer to inform the client of the required scope of works to develop the selected scheme. Depending on the chosen scheme, further geotechnical investigation and analysis may be required
- Structural and geotechnical engineer to proceed with the preliminary design stage.

GRIFFITHS DRILLING CO. LTD.

Telephone 269-943 Upper Hutt
P.O. Box 40422, Upper Hutt.

Client: Masterton Borough Council
Chapel Street, Masterton.

INVESTIGATION DRILLING RECORD

SITE: Borough Council Administration Building.

HOLE No: 2

DATE: 11-5-84

WATER LEVEL: 2.00m. approx.

Depth Metres	R.T.	Description
0.040mm		Bitumen.
0.300mm		Greyish brown moderatly firm sand and gravel fill.
1.00		Dark grey firm sandy silt.
1.20		Brownish grey moderatly dense sand and small gravels mixed.
.80		Grey dense sand and gravels mixed, high sand content.
3.45		Grey moderatly dense sand and gravels with some silt mixed, also some small sand seams.

RAYMOND TESTS

Depth (m) 150/100/100/100mm ie 450mm total.

1.00 6/14/20/20 A

2.00 17/$\underbrace{13/11}_{38}$/14

3.00 14/$\underbrace{10/12/27}_{49}$

phoned Griffiths 26/7/84

Depth Metres	R.T.	Description
64	CHAPEL ST MSTN	
		Bore 1 not drilled due to underground services in foot path.
		Received 16/7/84.

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P.O. Box 40422, Upper Hutt.

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Chapel Stre , Masterton.

INVESTIGATION DRILLING RECORD

SITE: Borough Council Administration Building.

HOLE No: 3

DATE: 11-5-84

WATER LEVEL: 2.00m approx.

Depth Metres	R.T.	Description
0.040mm		Bitumen.
0.300mm		Dark brown sand and stone fill.
1.00		Dark grey firm sandy silt.
1.70		Grey very soft sandy silt.
2.50		Grey moderatly dense sand and gravels mixed very little silt present.
3.45		Grey very dense sand and gravels, very little silt present.
		RAYMOND TESTS
	1.00	2/1/1/2
	2.00	$14/\underbrace{15/12/12}_{39}$
	3.00	$16/\underbrace{17/24/22}_{63}$

Depth Metres	R.T.	Description

GRIFFITHS DRILLING CO. LTD.

Telephone 269-943 Upper Hutt
P.O. Box 40422, Upper Hutt.

Client: Masterton Borough Council,
Chapel Street, Masterton.

INVESTIGATION DRILLING RECORD

SITE: Borough Council Administration Building.

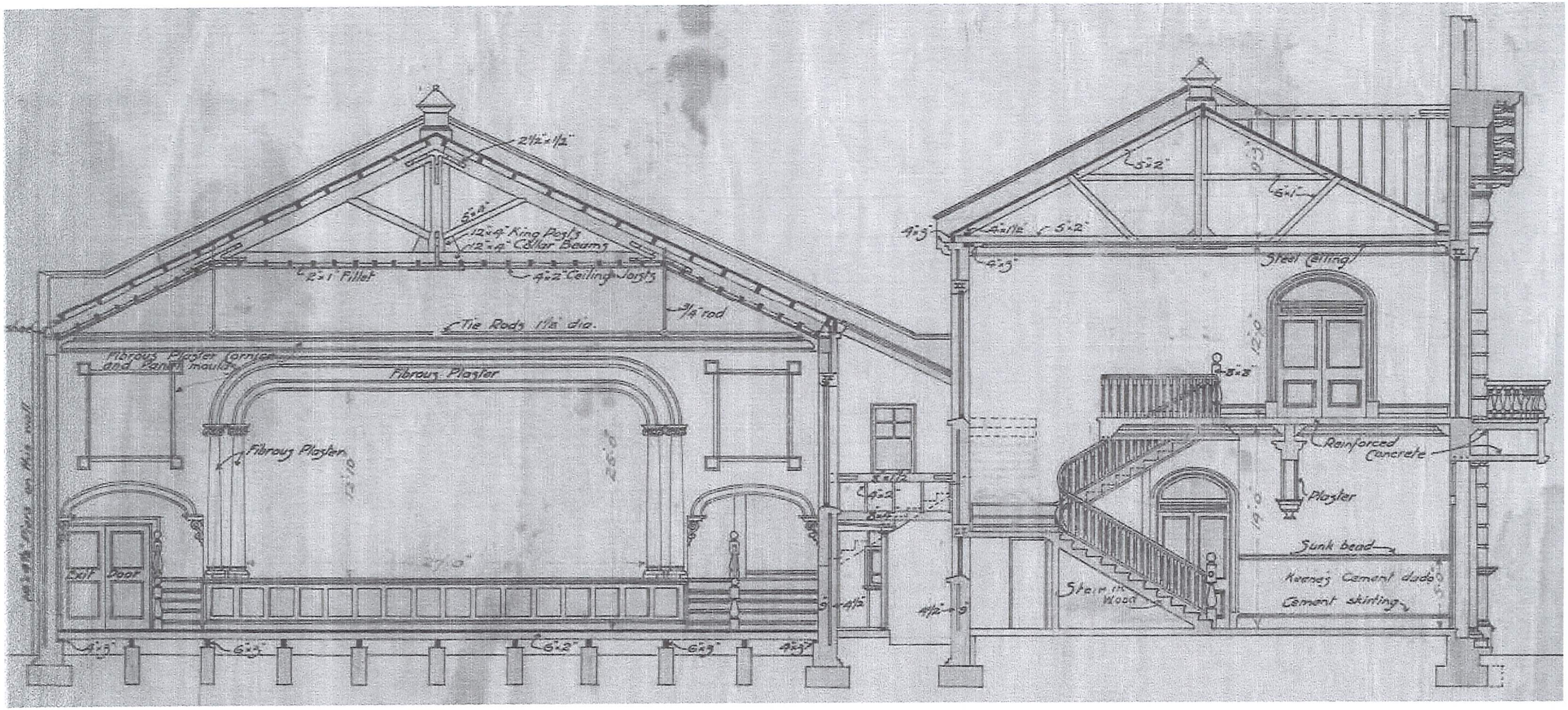
HOLE No: ④

DATE: 14-5-84

WATER LEVEL: 2.00m approx.

Depth Metres	R.T.	Description
0.040mm		Bitumen.
0.250mm		Dark brown sand and stone fill.
1.30		Brownish grey soft sandy silt.
3.45		Grey very dense sand and gravels, very little silt present.
		RAYMOND TESTS
	1.30	11/12/21/25
	2.00	23/22/20/22 64
	3.00	19/22/18/18 58

Depth Metres	R.T.	Description



Section NE-SW through Town Hall and Municipal Building

Tonkin+Taylor

BOREHOLE LOG

BOREHOLE No.: **BH-TT1**
SHEET: 1 OF 2

PROJECT: MDC Town Hall & Municipal Building	LOCATION: 64 Chapel Street, Masterton		JOB No.: 1001891.0000
CO-ORDINATES: (NZTM) 5463479 mN 1823862 mE	DRILL TYPE: Fraste XL Sonic	HOLE STARTED: 20/03/2017	
	DRILL METHOD: SNC	HOLE FINISHED: 21/03/2017	
R.L.: 112m		DRILLED BY: Griffiths Drilling	
DATUM: ELLIPSOID	DRILL FLUID: WATER	LOGGED BY: M Blakely	CHECKED: N Peters

GEOLOGICAL GEOLOGICAL UNIT, GENERIC NAME, ORIGIN, MATERIAL COMPOSITION.	FLUID LOSS (%) 25 50 75	WATER	CORE RECOVERY (%)	METHOD	CASING	TESTS	SAMPLES	RL (m)	DEPTH (m)	MOISTURE CONDITION / WEATHERING	STRENGTH/DENSITY CLASSIFICATION	SHEAR STRENGTH (kPa) 10 25 50 100	COMPRESSIVE STRENGTH (MPa) 1 5 20 50 100 250	DEFECT SPACING (cm) 20 60 200 600 2000	ENGINEERING DESCRIPTION Description and Additional Observations
Fill		NONE RECORDED	0	VACUUM EXCAVATION	NONE REQUIRED					N/A	N/A				Chip seal and base course.
										M	LP				Fine to coarse GRAVEL, some sand and minor cobbles; grey. Loosely packed, moist, well graded. Gravel is angular, slightly weathered sandstone (greywacke).
Alluvial Deposits										W	L				Silty SAND, trace organics (roots); grey. Loose, wet.
								111	1	M					Fine to coarse GRAVEL, some sand and minor cobbles; grey. Loosely packed, moist, clast supported. Gravel is sub-angular to sub-rounded, slightly weathered sandstone (greywacke).
			80	SPT		7/16 21/27 2 for 15mm N>=50 No liner					VD				1.3m: very dense
			100	HFS				110	2						
			46	SPT		3/9 12/11 13/11 N=47 No liner		109	3		D				3.0m: changes to dense
			42	HFS				108	4	W					Fine to medium GRAVEL, minor coarse gravel, cobbles, sand and silt; grey. Dense, wet, clast supported. Gravel is sub-rounded, slightly weathered, sandstone (greywacke). 3.9 to 4.5m: Core loss
			88	SPT		13/10 13/13 19 for 65mm N>=50 No liner				W	VD				Fine to coarse GRAVEL, some sand, minor cobbles and silt; grey. Very dense, wet, matrix supported. Gravel is sub-rounded, slightly weathered sandstone (greywacke).
			100	HFS				107	5	M					Silty fine to coarse GRAVEL, minor sand and cobbles, trace clay; brown. Very dense, moist, moderately graded, matrix supported. Gravel is sub-rounded (M-C) and SA (F), slightly weathered sandstone (greywacke).
			100	SPT		9/18 24/26 for 75mm N>=50 No liner		106	6						
			100	HFS				105	7						7.1m: changes to some silt (less silt observed)
			82	SPT		5/15 24/24 for 65mm N>=50 No liner									
			100	HFS				104	8						
			87	SPT		5/15 15/15 15/5 for 35mm N>=50 No liner		103	9						8.95m: changes to silty GRAVEL, as above (more silt observed)

COMMENTS: Automatic SPT hammer used with 87.9% energy efficiency. 'No liner' refers to SPT sampler; sampling spoon capable of including liner but none used.

Hole Depth
12.45m

Scale 1:50

Rev.: A

BoreLog - 27/03/2017 10:12:21 a.m. - Produced with Core-GS by GeRoc

17th U.S.-Japan-New Zealand
Workshop on the Improvement of
Structural Engineering and Resilience

NZ LOADINGS STANDARD (NZS1170.5) 2016 AND 2018 MODIFICATIONS TO STRUCTURAL CLAUSES FOR INCREASED SEISMIC RESILIENCE

Rob Jury[1], Des Bull[2], Gregory MacRae[3]
Beca[1], HCG[2], University of Canterbury[3]
New Zealand

Abstract

The New Zealand Standard for Structural Design Actions, NZS1170.5 has required amendment after recent earthquakes in New Zealand.

The September 2016 amendment (Amendment 1) involved making modifications for a number of issues including the spectral shape factor, allowances for subsoil amplification, spectra for vertical loading, overlap distances for ramps/stairs between building floors or adjacent buildings, floor diaphragm design, non-structural elements, and ratchetting of structures. The definition of Ultimate Limit State was also amended. Many of the revisions were in response to the recommendations of the Royal Commission on the Canterbury Earthquakes.

After the Kaikoura earthquake in November 2016, the New Zealand Government requested that any issues with the existing standard be considered and addressed in a further amendment likely to be available in 2019. This addresses ratchetting provisions, which have been both simplified and generalised, as well as building inelastic torsion effects. The amendment committee was also requested to consider changes as a result of the partial collapse that occurred to the Statistics Building and in particular changes in relation to the effects of the Wellington Basin.

Also, it may be of interest to overseas engineers that the current NZ standard does not yet explicitly address the MCE level of earthquake shaking.

This paper describes the provisions incorporated, or likely to be incorporated into the 2016 and 2018 amendments, the reasons for them, and some of the latest thinking on these topics. In addition, the method by which the standard considers levels of shaking greater than the design level, and reasons both for and against this approach, are described.

Introduction

Sizable earthquakes in New Zealand over the last eight years have provided the impetus for inclusion of several new, and the review of a number of existing, requirements for seismic design of new buildings in New Zealand.

The Canterbury Earthquakes Royal Commission deliberated following the Canterbury Earthquake Sequence (which included the damaging 22 February 2011 Christchurch Earthquake) and made over thirty recommendations for action that could be considered relevant to definition of design earthquake loadings in New Zealand. These covered issues such as; specification of vertical earthquake effects, allowances for drift, definition of ultimate limit state for earthquake, allowances for regularity (torsional and unbalanced strength), diaphragms and provisions for support of stairs and ramps.

development of inelasticity in parallel, otherwise identical, lateral load resisting elements. With increasing earthquake shaking, damage resulted in an ever increasing level of plan irregularity and greater levels of plan rotation than would have previously been expected or allowed for in the design for buildings of this type.

The effect was very apparent in a rectangular multi-storey building with near identical ductile frames on the perimeter of opposing sides of the building. On the basis of an elastic code based analysis the torsional resistance was predominantly provided by the frames which were spaced the greatest distance apart. When the earthquake loading was in the direction of these frames a small irregularity was surmised to cause one of the frames to enter the inelastic range before the other. The loss in stiffness in the inelastic frame caused the centre of rotation to move towards the undamaged frame increasing the torsion on the building which could not be resisted by the couples involving the yielding frame and the perpendicular frames. As a result the building had become torsionally irregular and the ductility demands on the yielding frame and the demands on the perpendicular frames would have been much greater than would have been calculated using the then code provisions.

Amendment 1 of NZS 1170.5 requires increased demands due to inelastic torsion to be considered in the design of buildings where the design ductility is greater than 1.25 and there are fewer than three major lines of resistance in the direction being considered. It is felt that three lines as a minimum provides some assurance that torsional resistance can be maintained when inelastic behavior is experienced. Structures not meeting these requirements need to be subjected to further consideration of inelastic torsional effects in the design although it is recognized a full appreciation of these effects needs further research.

Further provisions are under discussion at the time of writing.

Ratcheting

Structures that are required to exhibit ductile behavior in an earthquake and with a significantly greater strength in one direction from the other have the potential to be subjected to higher lateral inelastic displacements in the weaker direction. These inelastic deformations can increase progressively over the duration of strong earthquake shaking and can lead to accumulated deformations in the weaker direction that are significantly greater than might be predicted by elastic based methods, typically employed in design. These additional deformations can have ramifications for seating of secondary structural and non-structural elements and for the stability of affected structures.

The phenomenon is referred to as ratcheting and made its first appearance in NZS 1170.5 in Amendment 1 following evidence that the behaviour had affected several buildings during the 2011 Christchurch Earthquake.

The potential influence of ratcheting is assessed by calculating a ratcheting index which is basically the ratio of the provided lateral strength in the strong direction to the strength in the weak (reverse) direction. If the ratcheting index is above a certain level, dependent on the expected hysteresis shape for the structure, then the deflection profile for the structure in the weak direction is determined by multiplying the deflection profile in the strong direction by a ratcheting magnification factor.

The effects of ratcheting can be mitigated by a designer by taking steps to ensure the strength provided is similar in each (forward and reverse) direction of loading irrespective of the determined demand.

The degree of out of balance that requires action and the extent of the amplification required is currently under review for the latest amendment of NZS1170.5.

- Introduction of category for parts required to maintain operational/functional continuity in all buildings, not just those classified as post disaster structures. This is in recognition that there are some elements that are necessary for commercial and residential buildings to remain occupied and that the occupiable state is necessary at levels of shaking greater than that currently defined for the onset of damage. Currently this requirement only applies for parts and components but it is likely that it will constitute a new limit state for the structure of these buildings (between current serviceability limit state, SLS1 and the ultimate limit state, ULS) once a more detailed review of the Standard has been completed.

Parts Supported on Ledges

Complete failure of a precast concrete scissor stair in an 18 storey building during the 22 February 2011 Christchurch Earthquake (CERC Volume 2) indicated both the importance of maintaining stair access in high rise buildings to enable egress after an earthquake and the particular vulnerability of stair cases reliant on ledges for support. In this particular case the stair support ledge was arguably sufficient to sustain the building lateral deformations predicted during the earthquake but it has been surmised that permanent deformation in the flight when forced into compression as clearance gaps were taken up, shortened the flight so that on the reverse cycle the provided ledge length was insufficient and the flight(s) fell taking out the complete stair.

This failure indicated the need for sizing ledges conservatively for parts of buildings where these are the sole means of vertical support and where exceeding the ledge length results in collapse. It was also recognized that in order to meet safety objectives vertical support provided by such ledges must be capable of sustaining typical design deformations by a considerable margin and that these should not be reduced for ductility in the structure via the structural performance factor, S_p.

The provisions now require ledges to be sized to cope with ULS drifts multiplied by $2/S_p$ after allowing for all other factors that could lead to a reduction in support length such as construction tolerances, creep and shrinkage, foundation deformations, spalling and permanent inelastic deformations in the part between the points of support.

Ultimate Limit State Definition

In the 1992 Earthquake Loadings Standard (NZS 4203) it was recognized that Ultimate Limit State (ULS) for earthquake represented a different limit state (greater levels of strain) than applied for other load cases, e.g. gravity and wind.

This distinction was lost in NZS 1170.5 in the attempt to reach consensus with the Australians regarding a joint standard for general loadings, including earthquake. A definition referring to a state of instability, losing equilibrium and a small residual capacity to prevent collapse resulted. This caused confusion over what the ULS for earthquake represented and how much margin was expected against collapse once the ULS had been reached.

The definition has now been amended to relate to strength, strain, ductility and deformation limits specified for the ULS in the Standard and a for reserve capacity (deliberately undefined) to avoid structural collapse, even though the structure may have sustained significant structural damage.

There is no right or wrong way to address these issues but it is important that the objectives are clearly understood by designers and any additional design effort can be justified in the context of providing resilience within the risk-based code framework. Consideration of these aspects will be a necessary part of any future general review of earthquake design provisions and of the New Zealand Earthquake Loadings Standard in particular.

Acknowledgements

The authors are members of the NZS 1170.5 Standards Committees that deliberated on Amendment 1 and are currently considering Amendment 2. They would like to acknowledge the contribution to the various debates that their fellow committee members have made but also make it clear that the opinions given in this paper are their own and are not intended to support any particular view or position.

References

CERC:2012, Final Report of the Canterbury Earthquakes Royal Commission, Volumes 1 to 7, 2012

NZS 1170.5:2004 *Structural Design Actions – Part 5: Earthquake Actions – New Zealand,* New Zealand Standard, (incorporating Amendment 1 (following Canterbury Earthquake Sequence)).

NZS 4203:1992, *Code of Practice for General Structural Design and Design Loadings for Buildings*, New Zealand Standard, 1992, (now superseded by NZS 1170)

New Zealand Building Code *Clause B1 – Structure,* First Schedule of the Building Regulations 1992, New Zealand Legislation