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Design Guidance of Specifically Designed Bracing Systems in Light Timber-framed Residential Buildings

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Note

This report is intended for structural engineers and building officials.

Design Guidance of Specific Seismic Bracing Systems in Light Timber-framed Residential Buildings

BRANZ Study Report SR337

Angela Liu

Abstract

In the Canterbury earthquakes, residential houses of mainly light timber-framed (LTF) construction achieved the objective of “safeguarding people from injury caused by structural failure”, as required by the New Zealand Building Code. However, the earthquake damage was often significant and beyond homeowners’ expectations. Of special importance was that the mixed use of typical sheathed LTF wall bracing elements with specifically designed bracing systems significantly exacerbated the seismic damage to the LTF residential houses. This was most likely caused by incompatibilities between mixed bracing elements in a mainly LTF residential house. The objective of this study was to develop design guidance to mitigate, through better designing and detailing, the earthquake damage to LTF residential buildings. The scope of the study was limited to the buildings, which include a mixture of specifically designed bracing elements and conventional NZS 3604 sheathed LTF wall bracing elements.

In this guidance, the construction practice and engineering characteristics of mainly LTF residential buildings were examined first. The examination highlighted that potential stiffness incompatibility between conventional LTF bracing walls and specifically designed bracing elements could significantly facilitate earthquake damage to LTF buildings. In order to achieve stiffness compatibility between specific bracing elements and LTF wall bracing elements, the expected seismic performance level of LTF residential buildings with minimum NZS 3604 seismic bracing was assessed. The assessment was conducted, based on a displacement-based approach and available P21 test results of sheathed LTF walls. The assessment revealed that the displacement performance achievement of the building system with minimum NZS 3604 seismic bracing is inappropriate for use as the performance criterion for specifically designed seismic bracing elements. The seismic performance requirement for specifically designed seismic bracing elements in mainly NZS 3604 buildings was established as 1%, in terms of storey drift at ultimate limit state. This was based on observed test results of conventional LTF bracing walls and the current seismic loading standard NZS 1170.5. A step-by-step seismic design procedure for specifically designed bracing elements, which was developed according to a displacement-based approach, is presented.

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1. INTRODUCTION

The majority of residential buildings in New Zealand are light timber-framed (LTF) buildings. By definition, LTF buildings refer to low-rise residential buildings that have suspended timber floor/roof construction and have sheathed light timber-framed walls as the gravity load and lateral load-resisting systems.

Earthquake damage observed in the Canterbury earthquakes demonstrated that residential houses of mainly LTF construction achieved the New Zealand Building Code (NZBC) objective of “safeguarding people from injury caused by structural failure”. However, the earthquake damage was often significant and unacceptable. Figure 1 and Figure 2 show observed earthquake damage to modern plasterboard sheathed LTF walls during the Canterbury earthquake sequence (Buchanan et al. 2011).

Figure 1 Severe failure of high-grade plasterboard internal linings (Buchanan et al. 2011)

Figure 2 Failure of generic plasterboard/metal brace combination (Buchanan et al. 2011)

Of importance was that the damage magnitudes to light timber-framed houses varied significantly in the Canterbury earthquake sequence. The mixed use of typical sheathed LTF wall bracing elements with specifically designed bracing systems significantly exacerbated the seismic damage to the LTF residential houses. This was most likely caused by incompatibilities between mixed bracing elements in a mainly LTF residential house.

While not an objective of the NZBC, it has been an objective of the Earthquake Commission to improve the knowledge of natural hazard risk and to inform building standards. By reducing the damage sustained by a dwelling in such an event, there is a greater likelihood that people can remain in their homes following an ultimate limit state design level earthquake.

The objective of this study was to develop design guidance for LTF residential buildings, which include a mixture of specifically designed bracing elements and conventional NZS 3604 sheathed LTF wall bracing elements. This guidance provides a technical basis and a design procedure.

In section 2, the construction practice and engineering characteristics of mainly LTF residential buildings are examined first. This highlights that potential stiffness incompatibility between conventional LTF bracing walls and specifically designed bracing elements could significantly facilitate earthquake damage to LTF buildings.

Section 3 provides the technical basis for establishing the performance requirements of specifically designed seismic bracing elements within mainly NZS 3604 residential buildings. In order to achieve stiffness compatibility between the specifically designed bracing elements and conventional NZS 3604 LTF wall bracing elements, the expected seismic performance level of LTF residential buildings was examined. The examination revealed that the minimum NZS 3604 bracing requirement could result in very soft houses. Namely the minimum NZS 3604 bracing requirement potentially could lead to displacements larger than the 2.5% drift limit as specified by the current seismic loading standard NZS 1170.5. Apparently, it would be inappropriate to establish the performance criterion of specifically designed bracing elements based on the minimum NZS 3604 bracing requirements. Consequently, the seismic performance requirement for specifically designed bracing elements in mainly NZS 3604 buildings was established as 1% in terms of storey drift at ultimate limit state (ULS). This was based on observed test results of conventional LTF bracing walls and the current seismic loading standard NZS 1170.5.

Section 4 presents the seismic design procedure, step by step, for specifically designed bracing elements, and Section 5 summarises the findings and conclusions of the study.

2. SEISMIC ENGINEERING CHARACTERISTICS OF LTF BUILDINGS

2.1 Construction of LTF residential buildings in New Zealand

Construction of residential light timber-framed buildings in New Zealand largely follows a prescriptive standard – NZS 3604 *Timber-framed buildings*. NZS 3604 has been developed for constructing simple small-scale LTF buildings. The application of NZS 3604 has limitations, such as, the bracing lines should be spaced not more than 6 metres unless floor or ceiling diaphragm action can be assured through suitable detailing.

In NZS 3604, the seismic demand is determined by reading off a predefined table, based on the soil classification, seismic hazard zone, house foundation type and building envelope weight. NZS 3604 also specifies that the P21 test and evaluation procedure developed and published by BRANZ be used to evaluate the seismic bracing capacity of proprietary LTF wall elements. Designers just need to match the bracing capacity provided to the bracing demand. Apart from matching the bracing capacity provision with the bracing demand, the bracing elements are required to be evenly distributed along notional “Bracing Lines” in each orthogonal direction. In each bracing line, the minimum bracing provision is 50% of the total bracing demand divided by the number of bracing lines. As a result, LTF residential buildings constructed to NZS 3604 have an ‘egg-crate’ structural form and are believed to be reasonably regular both in plan and vertically. However, there has never been any verification work done to justify the insignificance of the irregularity issue for NZS 3604 buildings.

Nowadays, more and more LTF buildings are not completely within the scope of NZS 3604. This can be due to the desire to make use of large picture windows, for example, to gain the best benefit from the view. When constructing LTF buildings in this category, the special bracing elements in the areas beyond the scope of NZS 3604 are required to be designed by a structural engineer. The rest of the building can still be constructed according to NZS 3604. The specifically designed bracing elements are often different bracing systems rather than light timber-framed sheathed walls. In designing the specific bracing elements, the structural engineer determines the bracing demands for the concerned area based on the engineering basis of NZS 3604, and the design allows for two criteria:

- The strength criterion as required by AS/NZS 1170 for all the loading combinations at ultimate limit state.
- The stiffness criterion, which is the suggested serviceability limit state (SLS) criteria in AS/NZS 1170.0.

Are the specifically designed bracing systems as in current practice compatible with the P21 rated bracing systems used in conjunction with NZS 3604? There has been very little research work on this topic.

Observed earthquake damage in Christchurch showed significant discrepancies between simple LTF houses and LTF houses with specific bracing elements. Therefore, the compatibility of the design principles underlying NZS 3604 and the design principles underlying the current practice for specific bracing elements in LTF buildings need to be examined. In doing so, a consistent seismic performance of these buildings will be achieved.

2.2 Seismic engineering characteristics of LTF residential buildings

Seismic responses of a building structure depend on the seismic mass and many other structural characteristics of the bracing systems, such as its stiffness properties and energy-dissipating capacities.

For large heavy buildings, such as reinforced concrete structures or steel structures with concrete floors, collapse in earthquakes can occur when the gravity load carrying systems become unstable. This is due to the combined effects of seismic actions and P-Δ actions when the buildings undergo significant earthquake-induced lateral deflections. In comparison, LTF buildings are less problematic in this regard. Typically, in an LTF residential building, both the gravity load carrying systems and the lateral seismic resisting systems are LTF sheathed walls. The lateral deflections experienced by these walls in design earthquakes would be small in comparison with the wall lengths. In addition, low-rise LTF residential buildings are generally light in nature, and subsequently, the P-Δ effects usually are not significant enough to cause instability problems. Therefore, the LTF residential buildings of mainly NZS 3604 construction could easily achieve life safety requirements in design earthquakes. The reported collapse limit state for low-rise LTF buildings could reach a storey drift of 6% (Paevere et al. 2003, Bahmani and Van de Lindt 2013).

However, LTF residential buildings could have significantly greater earthquake damage to the structural systems in comparison with large buildings of other structural forms such as reinforced concrete or steel. This is because LTF buildings potentially have greater incompatibility issues between the bracing systems, and the effects of stiffness and deformation incompatibility issues have not been vigorously quantified in the design. For LTF residential buildings, the typical lateral resisting systems are LTF walls, which are of very different lengths. The longer walls are stiffer while the shorter walls are more flexible. Therefore, even if all the lateral bracing systems of the LTF buildings are gypsum plasterboard sheathed walls, a deformation incompatibility of seismic bracing wall systems is still expected. When an LTF building contains a mixture of lateral load-resisting systems (LTF bracing walls and specifically designed bracing systems), deformation incompatibility between various bracing elements is very likely to be greater.

As a result of potential deformation incompatibility, the induced seismic actions in different bracing systems can significantly deviate from a force-based theoretical prediction as in NZS 3604 (Priestley et al. 2007). In these circumstances, the rigidity of the floor or ceiling diaphragm plays an important role in distributing the lateral seismic actions to the different bracing systems. In principle, an absolutely rigid diaphragm will force all bracing systems to be constrained to translate by the same amount for a regular resisting system. If there is an irregularity in the distribution of bracing resistance, the building will also rotate about its centre of rigidity. This would result in greater bracing wall displacement demands on the perimeter of the building. A completely flexible floor or ceiling diaphragm means that each bracing line has to resist the seismic actions associated with the seismic weight within its tributary area. For LTF buildings, the timber floor diaphragms are neither rigid diaphragms nor completely flexible diaphragms. It is prudent that the effects of diaphragm stiffness are studied using a semi-rigid method or an envelope method (Kirkham et al. 2015). NZS 3604 practice will not guarantee a stiff floor diaphragm. The rigidity of the floor/roof diaphragm can also vary a lot depending on the geometric shape of floors/roofs (Lucksiri 2012) and/or steps at floor or roof levels. This would further complicate the stiffness incompatibility issues. It is important that the floor diaphragm rigidity is properly allowed for in order to adequately quantify the effects of the potential deformation/stiffness incompatibility issue between the bracing systems.

In summary, LTF residential buildings can easily achieve the life safety criterion as specified by the current seismic design standard. However, LTF residential buildings potentially have significant stiffness/deformation incompatibility issues between the bracing systems. Consequently, some areas may deflect significantly more than the theoretical predictions. For LTF buildings, the effect of the stiffness incompatibility between the bracing systems is further complicated by the fact that the timber floor diaphragms of LTF buildings are not rigid. There has been very little research into the effect of stiffness/deformation compatibility of the bracing systems for LTF residential buildings. Therefore, the seismic actions induced in different bracing systems could significantly deviate from a force-based theoretical prediction as per NZS 3604.

3. PERFORMANCE CRITERIA ESTABLISHMENT OF DAMAGE CONTROL LIMIT

3.1 General

As described previously, residential houses of largely LTF construction could easily achieve the life safety performance criterion, as required by the current NZBC. Observations of seismic performance in Christchurch indicate that a meaningful ultimate limit state seismic performance criterion for LTF buildings should be damage control. Earthquake damage to LTF buildings is a result of either differential deformations between different levels or differential deformations between different parts within a plan. Differential deformations between two adjacent levels of a building depend on the stiffness of the bracing elements between the two adjacent levels and they may be quantified using storey drift. Differential deformations between different parts of the buildings are due to deformation incompatibility of the bracing elements over these building areas, especially when the floor or ceiling diaphragms are relatively flexible. This is especially the case when LTF buildings have mixed bracing elements, namely specifically designed bracing elements and NZS 3604 LTF wall bracing elements.

However, consideration of a damage control limit state when designing buildings is not required by NZS 1170.5 except at the serviceability limit state. Therefore, it is essential to establish the performance requirement for designing specific bracing systems within LTF buildings. In principle, the performance requirement for designing specific bracing systems shall match the expected performance level of the conventional LTF sheathed wall bracing elements as per NZS 3604. This is to ensure that the NZS 3604 bracing systems and the specifically designed bracing systems have compatible stiffness and deformation performance.

In this section, the engineering basis of NZS 3604 is examined first (section 3.2). Then, a simplified case with minimum NZS 3604 seismic bracing provision is studied. The expected performance level of the studied case was assessed based on typical P21 test results of conventional gypsum plasterboard sheathed LTF walls (section 3.3). The concluded finding of the case study is that minimum NZS 3604 bracing provision is not appropriate for being used in establishing the performance requirement of the specific bracing elements. Subsequently, the damage control performance requirement of specific bracing elements was established by using first engineering principles and typical P21 test results of plasterboard LTF walls (section 3.7).

3.2 Engineering basis of NZS 3604

NZS 3604, a design standard for light timber-framed buildings, is an Acceptable Solution for constructing LTF residential buildings within specified limits. In NZS 3604, the seismic bracing demand was derived using a force-based approach, namely, the equivalent static method as recommended by NZS 1170.5, assuming ductility of 3.5.

The governing equation for seismic base shear is as follows:

$$V = C_d(T_1) \times W_t$$

where:

V = horizontal seismic shear force at the base of the structure,

$C_d(T_1)$ = horizontal design action coefficient,

W_t = seismic weight.

$$C_d(T_1) = \frac{C(T_1)S_p}{k_\mu}$$

where:

S_p = 0.7, which is structural performance factor for a ductility of 3.5

k_μ = inelastic spectrum scaling factor
assuming T_1=0.4 s, k_μ is 2.4 for class A–D soils and 2.3 for class E soils

$C(T_1)$ = $C_h(T)$ Z R N(T,D)

$C_h(T)$ = spectral shape factor at T_1=0.4 s, and the value of $C_h(T)$ is:

- 1.89 for class A and B soils
- 2.36 for class C soils
- 3.0 for class D and E soils

Z = hazard factor and Z values are shown in Table 1 for the different seismic zones as per NZS 3604

R = return period factor at ULS – it is 1.0 for Importance Level 2 buildings covered by NZS 3604

N(T,D) = near-fault factor – 1.0 for building period ≤ 1.5 sec, regardless of the distance to the nearest major fault

Table 1: Hazard factor

Seismic zone in NZS 3604	Hazard factor Z
1	0.2
2	0.3
3	0.46
4	0.6

For example, for soil class D as per NZS 1170.5 and seismic zone 3 as per NZS 3604, $C_d(T_1)$ = 3.0 x 0.46 x 1.0 x 1.0 x 0.7 / 2.4 = 0.4 was used in calculating the seismic demand by NZS 3604.

For the provision of bracing capacity, NZS 3604 adopted the P21 test procedure (Shelton 2010), developed by BRANZ, to evaluate seismic bracing capacity of proprietary LTF sheathed wall elements (Shelton, 2013). The P21 test is a slow cyclic racking test on a cantilever proprietary LTF wall element, applying a load at the top of the wall. The seismic rating of the wall element is determined from the fourth cycle force at a deflection between 15 mm and 36 mm, depending on when significant strength degradation occurs. P21 tests are often conducted on standard wall lengths, 0.4 m long, 0.6 m long and 1.2 m long. For longer walls up to 2.4 m length, the seismic rating per metre length is assumed to be the same as for 1.2 m long walls. That is, the determined rating may be applied to walls up to twice the length of the tested wall.

For the seismic bracing design, NZS 3604 designers just need to match the total capacity provided to the derived total bracing demand. The bracing systems are required to be on lines spaced at not more than 6 m, resulting in notional bracing lines.

3.3 Expected performance of minimum NZS 3604 seismic bracing provision

3.3.1 General

The expected performance level for NZS 3604 construction with minimum NZS 3604 bracing provision needs to be assessed. This hopefully will provide the basis for establishing the required performance requirements for specifically designed bracing elements in mainly LTF residential buildings.

Seismic resistance of any lateral load-resisting system depends on not only its strength but also its deformation capability and energy-dissipating capacity.

For LTF residential houses constructed to NZS 3604, the seismic bracing systems are proprietary sheathed LTF walls, which have seismic resistance rated from the P21 test. According to NZS 3604, the minimum required bracing capacity (to match the **demand**), in terms of the strength, is proportional to the seismic mass (weight), namely:

$$V_{cap} = C_d(T_1) \times W_t$$

where V_{cap} is the bracing capacity provided and W_t is the total seismic mass.

But what about the deformation capability of the LTF bracing elements?

In a typical LTF residential building constructed to NZS 3604, the lateral seismic resisting systems are LTF wall bracing elements. These elements will inevitably have different lengths and are therefore expected to have different stiffness performance. The longer the walls are, the less deformation capacities the walls have prior to the strength degradation (softening).

Figures 3–6 show several hysteresis loops of plasterboard sheathed LTF walls and plywood sheathed LTF walls measured during P21 testing. The wall height for all these reported tests was 2.4 metres.

In an NZS 3604 LTF building, the major contribution of the lateral seismic resistance is from LTF bracing walls longer than 1.2 m. Their bracing ratings are most likely to be their strengths at a deflection of 22 mm or 15 mm, as illustrated in Figures 3–6. An absolute differential deformation of 22 mm over a 2.4 m storey height equals a storey drift of about 1%. The 1% storey drift represents the deformation limit before an LTF sheathed wall of reasonable length would be expected to experience significant strength degradation and softening.

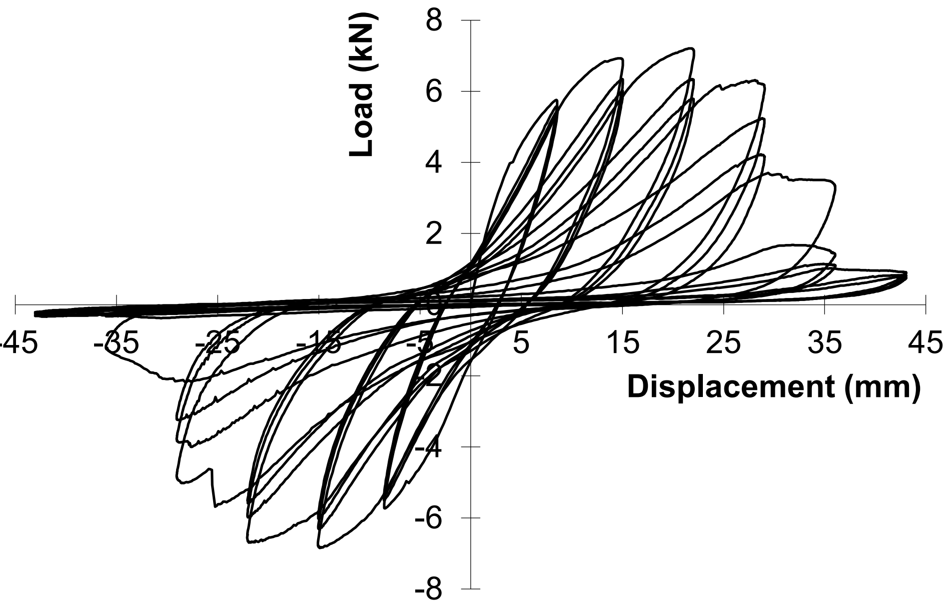

Figure 3 One 1.2 m long LTF wall with plasterboard sheathing

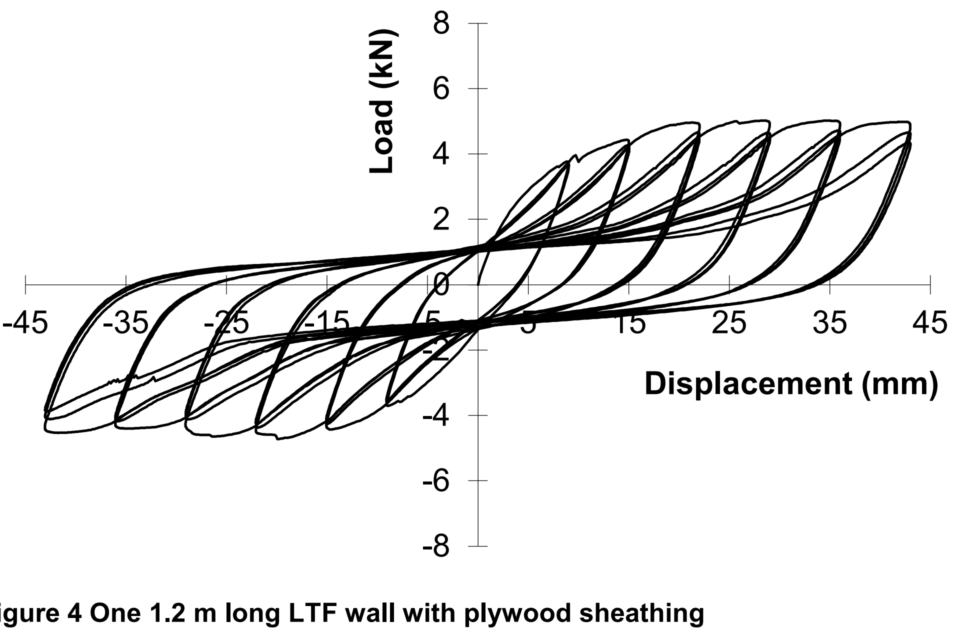

Figure 4 One 1.2 m long LTF wall with plywood sheathing

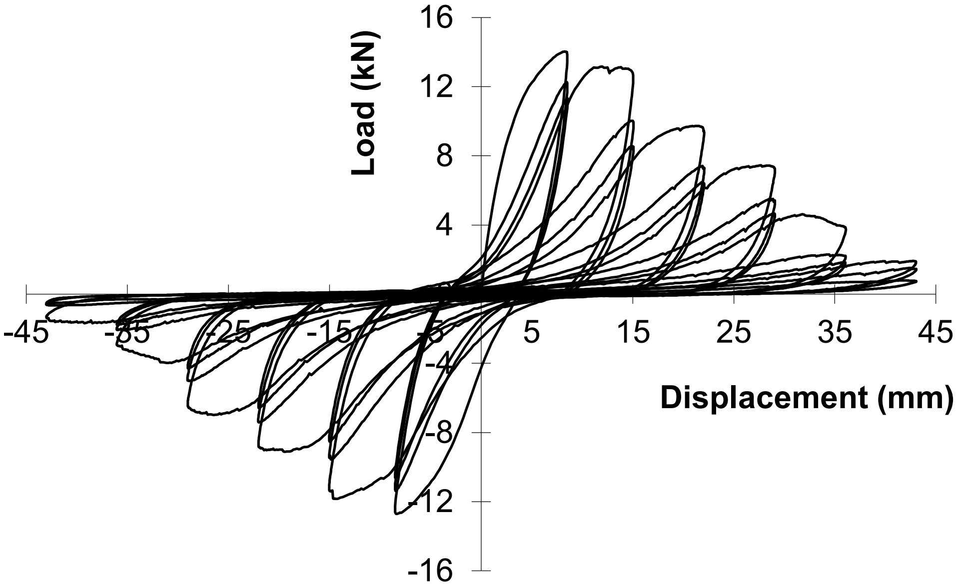

Figure 5 One 2.4 m long LTF wall with plasterboard sheathing

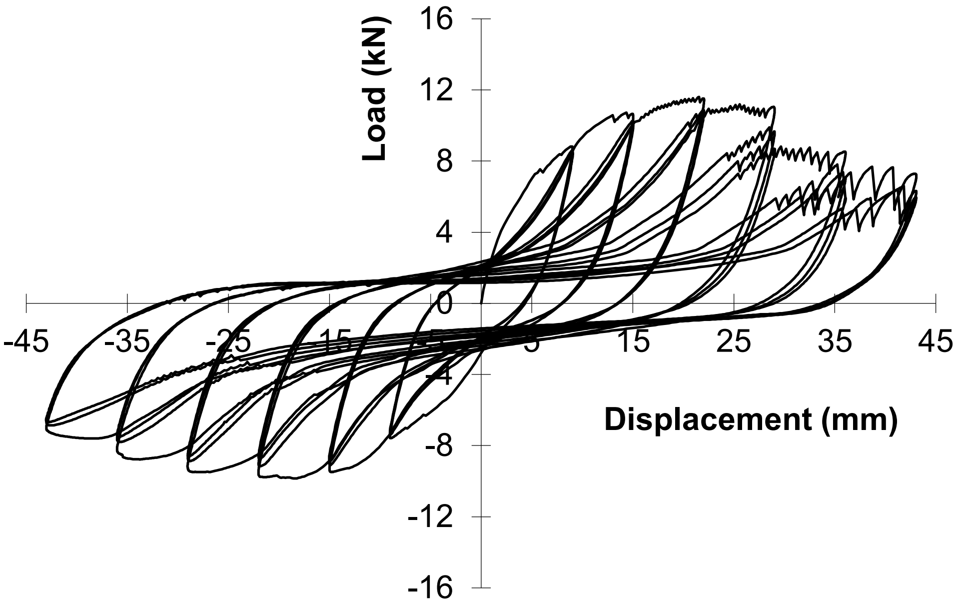

Figure 6 One 2.4 m long LTF wall with plywood sheathing

3.3.2 Case study LTF building

To verify the expected performance of minimum NZS 3604 seismic bracing provision, a case study LTF building is defined as follows:

a) It is a perfectly regular single-level LTF building with a storey height of 2.4 metres.

b) Structural bracing is provided by gypsum plasterboard LTF walls, and all the walls are of equal length.

c) The subsoil classification is D as per NZS 1170.5, and the earthquake zone is 3 as per NZS 3604.

d) The provided bracing capacity, V_{cap}, is exactly equal to the bracing demand as per NZS 3604, $V_{cap} = C_d(T_1) \times W_t = 0.4\ W_t$.

where:

W_t is the total seismic weight of the building

$C_d(T_1)$ is 0.4 and it is from Table 2 as used in NZS 3604.

Table 2: Seismic coefficient $C_d(T_1)$

	Zone as per NZS 3604			
Soil class as per NZS 1170.5	**1**	**2**	**3**	**4**
A or B	0.11	0.17	0.25	0.33
C	0.14	0.21	0.32	0.41
D	0.18	0.26	0.40	0.52
E	0.18	0.27	0.42	0.55

e) The bracing wall elements will achieve the rated bracing capacity, V_R, when the lateral deflection of the bracing wall elements is 22 mm. Significant strength degradation is expected when the bracing wall elements deform beyond 22 mm, as illustrated in Figure 7. It is assumed that the bracing strength (capacity) of the bracing wall elements will be $0.7V_R$ and $0.3V_R$ at a lateral deflection of 29 mm and 36 mm respectively.

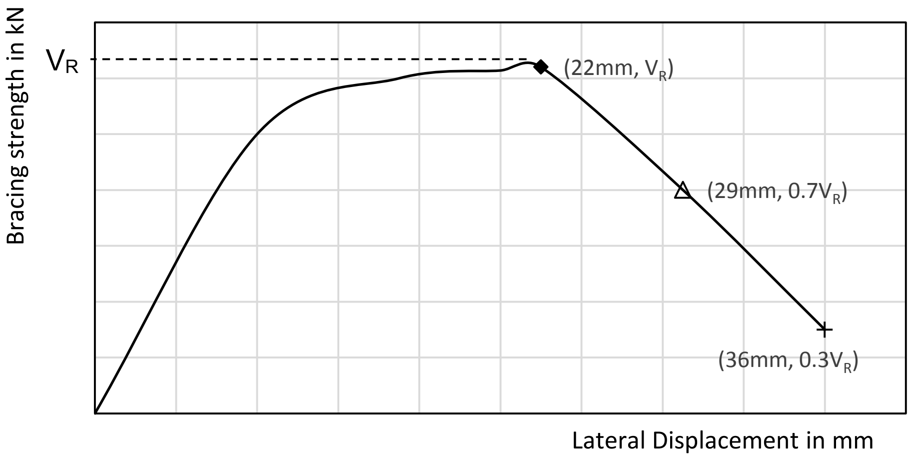

Figure 7 Assumed bracing strength – lateral deformation skeleton curve of LTF walls

f) The available damping, ξ, of LTF bracing wall elements is taken as ξ = 20%. This was the value calibrated from several P21 test results on gypsum plasterboard LTF walls using area based equivalent viscous hysteretic damping. Use of ξ = 20% is compatible with the damping level suggested by Newcombe and Batchelar (Newcombe and Batchelar 2012).

ξ = 20% is likely to be the upper limit for LTF bracing walls of reasonable lengths. As the cyclic loading progresses, the hysteretic loop pinching is more significant so damping level tends to degrade.

3.3.3 Expected performance of the case study LTF building system (Priestley et al. 2007)

To assess the performance level that the above defined study case can achieve, the following steps are undertaken:

Step 1: Effective stiffness of the bracing systems when the rated capacity is achieved:

$$K_{eff} = \frac{V_{cap}}{\Delta} = \frac{0.4W}{\Delta} = \frac{0.4Mg}{\Delta}$$

where: M is the mass of the building

Δ is the displacement of interest, which is the deformation level when the rated bracing capacity is attained and it is taken as 22 mm

g is the gravitational acceleration.

Step 2: Effective period of the building, T_{eff} – the dynamic property of the studied building system when the rated bracing capacity is achieved.

$$T_{eff} = 2\pi\sqrt{\frac{M}{K_{eff}}} = 2\pi\sqrt{\frac{\Delta M}{0.4Mg}} = 2\pi\sqrt{\frac{\Delta}{0.4g}} = 2\pi\sqrt{\frac{22\ (mm)}{0.4\times 9810\ (\frac{mm}{s^2})}} = 0.47\ (s)$$

Step 3: Displacement demand according to NZS 1170.5

Displacement demand of the case study building, which has the effective period of 0.47s, can be determined using displacement spectra at ULS developed for this site and the corresponding damping levels. This is shown in Figure 8.

For a damping of ξ = 20%, $C_d = S_p \times C_h(T)\ Z\sqrt{\frac{7}{2+\xi}} = 0.7 \times 3.0 \times 0.46 \times \sqrt{\frac{7}{2+20}} = 0.55$

Required displacement capability, Δ_{demand}, of the LTF wall bracing systems, associated with K_{eff} (or T_{eff}) and ξ, is

$$\Delta_{demand} = C_d\ (\frac{T_{eff}}{2\pi})^2 g = 0.55 \times (\frac{0.47}{2\pi})^2 \times 9810 = 30\text{ mm}$$

This is the displacement demand at ULS, derived for a system with T_{eff} = 0.47 s (This is the period of the building with minimum NZS 3604 bracing provision). In comparison with the assumed deformation capacity (as shown in Figure 7), 22 mm, of the LTF bracing walls, the displacement demand is much higher.

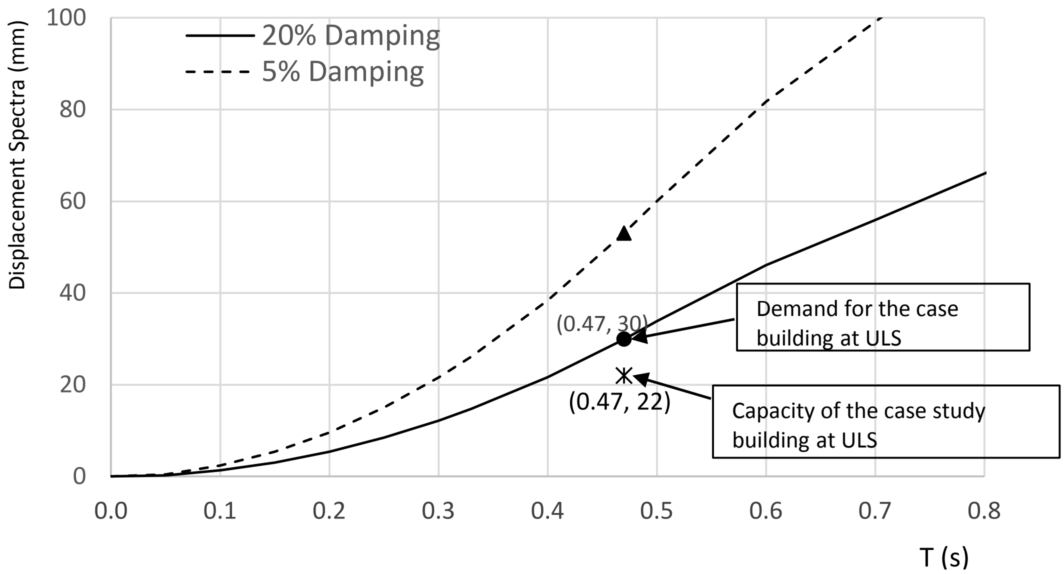

Figure 8 Displacement spectra for soil D to NZS 1170.5 and zone 3 as per NZS 3604

Step 4: What to expect in a 500-year event for the case study building?

As demonstrated above, the building system with the minimum bracing provision as per NZS 3604 would need to deflect significantly beyond 22 mm to satisfy the current seismic design standard NZS 1170.5. More deflection means less stiffness and longer period. The required deflection capacity under a 500-year event would be much greater than 30 mm if the bracing strength provision does not increase. Therefore, the building will be much softer and the fundamental period will be much longer. This is further explained as follows:

Figure 9 shows the constructed response spectra, expressed as response acceleration (seismic coefficient) versus response displacement for the site of the case study. For the case study building, the bracing strength (capacity) is equivalent to S_a = 0.4. In this case, the bracing walls need to deflect laterally for 70 mm (3.0% storey drift) at ULS even if the LTF bracing wall systems could maintain the damping level of 20%. This has significantly exceeded the specified deflection limit at ULS of 2.5% by NZS 1170.5. Furthermore, for gypsum plasterboard LTF bracing walls, significant strength degradation would be expected when the bracing walls deflect beyond 22 mm, leading to reduced bracing strength and greater displacement requirement.

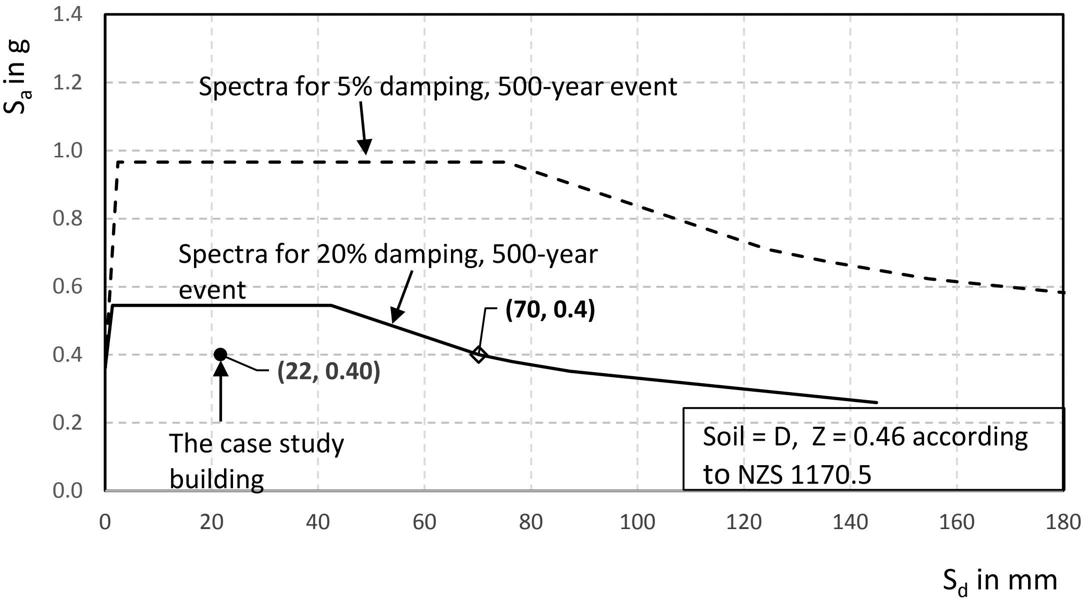

Figure 9 Constructed spectra acceleration (S_a) versus spectral displacement (S_d)

It is of interest that other researchers (Newcombe and Batchelar 2012) reached similar conclusions about the minimum seismic bracing strength requirement as per NZS 3604. This was based on their studies using displacement-based design principles and time history analyses with multiple earthquakes.

3.3.4 Discussion

The expected seismic performance of an artificial perfectly regular LTF house with minimum seismic bracing provision as per current NZS 3604 is examined above. This demonstrates that the seismic bracing provision according to NZS 3604 potentially does not meet the NZBC-specified stiffness performance criteria, even if the effects of irregular arrangement of bracing elements are ignored. In detail, should only minimum seismic requirements of NZS 3604 be satisfied, the LTF bracing walls potentially have to deflect well beyond the Code-specified deflection limit of 2.5% storey drift at ULS. The consequence is that significant to irreparable earthquake damage to LTF houses would be expected in a 500-year event.

The finding is no surprise because NZS 3604 has used the equivalent static method, a force-based approach, in developing the seismic design clauses. A force-based approach is not adequate in predicting inelastic seismic performance of building structures, which contain lateral load-resisting systems with significantly different stiffness characteristics (Paulay and Restrepo 1998).

Two assumptions in the force-based equivalent static method approach are usually responsible for its inadequacy in predicting the seismic performance of building structures. They are the fundamental period and the assumed displacement ductility, μ, where μ is the indicator of the energy-dissipating capacity of the lateral seismic resisting systems. For LTF houses, the effective period at the storey lateral deflection of 22 mm is 0.47 s, and it is comparable with the assumed T_1 = 0.4 s as per NZS 3604. The major reason why the current minimum NZS 3604 seismic bracing provision is inadequate is that NZS 3604 has overestimated the energy-dissipating capacity of typical LTF bracing walls by about 50%. In detail, the force-based equivalent static method as for NZS 3604 uses the displacement ductility as an index for energy-dissipating capacity of lateral seismic load-resisting systems. The assumed displacement ductility in NZS3604 is μ = 3.5. In this case, the inelastic spectrum scaling factor, k_μ, is in the range of 2.3 to 2.43 for different soil classes, assuming a fundamental period of 0.4 seconds as for LTF houses. This has reduced the design seismic action to about 41% of elastic seismic design action.

However, the energy-dissipating capacity of commonly used gypsum plasterboard bracing walls for NZS 3604 construction is equivalent to a damping level of less than 20%, as calibrated using P21 test results. For a damping level of 20%, the design seismic action reduction equals 57% of the elastic design action.

In summary, the seismic bracing design principles underlying NZS 3604 have underestimated the seismic bracing demand.

3.4 What if minimum NZS 3604 seismic bracing provision increases by 50%?

What is the expected performance level of an LTF house, in terms of deformation, should current NZS 3604 seismic demand be increased by 50%? It is to be noted that the same damping level of ξ = 20% will be appropriate because the bracing systems are still LTF bracing walls.

In this case, the provided bracing capacity is V_{cap} = 1.5 x 0.4W = 0.6W = 0.6Mg. The deformation at attaining the rated bracing capacity remains at 22 mm if the structural bracing still uses gypsum plasterboard LTF walls.

When the minimum NZS 3604 seismic bracing provision increases by 50%, the expected seismic performance level of an LTF house is estimated by repeating steps 1 to 4 as in section 3.3.3.

The process is summarised as follows:

$$K_{eff} = \frac{0.6Mg}{\Delta}$$

$$T_{eff} = 2\pi\sqrt{\frac{M}{K_{eff}}} = 0.38\ (s)$$

The reduction of the seismic action due to the damping is calculated according to:

$$R_\xi = \sqrt{\frac{7}{2+\xi}}$$

Assuming ξ = 20%, $R_\xi = \sqrt{\frac{7}{2+20}}$ = 0.56

Subsequently, the design seismic coefficient is:

$$C_d = S_p \times C_h(T)\ Z\ R_\xi = 0.7 \times 3 \times 0.46 \times 0.56 = 0.54$$

The required displacement capacity of the provided bracing systems as above is derived as follows:

$$\Delta_{required} = C_d \left(\frac{T_{eff}}{2\pi}\right)^2 g = 20 \text{ mm},$$

This is a close match to the deformation capacity, 22 mm, at rating the LTF wall bracing systems using P21.

In summary, the expected seismic performance of a regular LTF house will likely satisfy the deflection criterion of the current seismic loading standard NZS 1170.5 if:

- the seismic bracing actions as per current NZS 3604 increase by 50%
- the bracing systems are LTF bracing walls.

If the bracing systems have different damping level from ξ = 20%, adjustment needs to be made to allow for the effect of damping on the seismic demand.

3.5 Potential stiffening of LTF residential buildings designed to NZS 3604

Above discussion in section 3.3 is limited to LTF houses with minimum seismic bracing provision as per current NZS 3604. Quite often, LTF buildings constructed to NZS 3604 have significant reserve bracing capacities.

In LTF residential house construction, the LTF bracing wall systems are usually not cantilever walls as assumed in P21 tests. Rather, they have coupling actions initiated by infill panels between the bracing wall elements at the wall top and/or at the wall base, as shown in Figure 10. As a consequence, the LTF bracing walls will be stiffer than the sum of the individual cantilever walls.

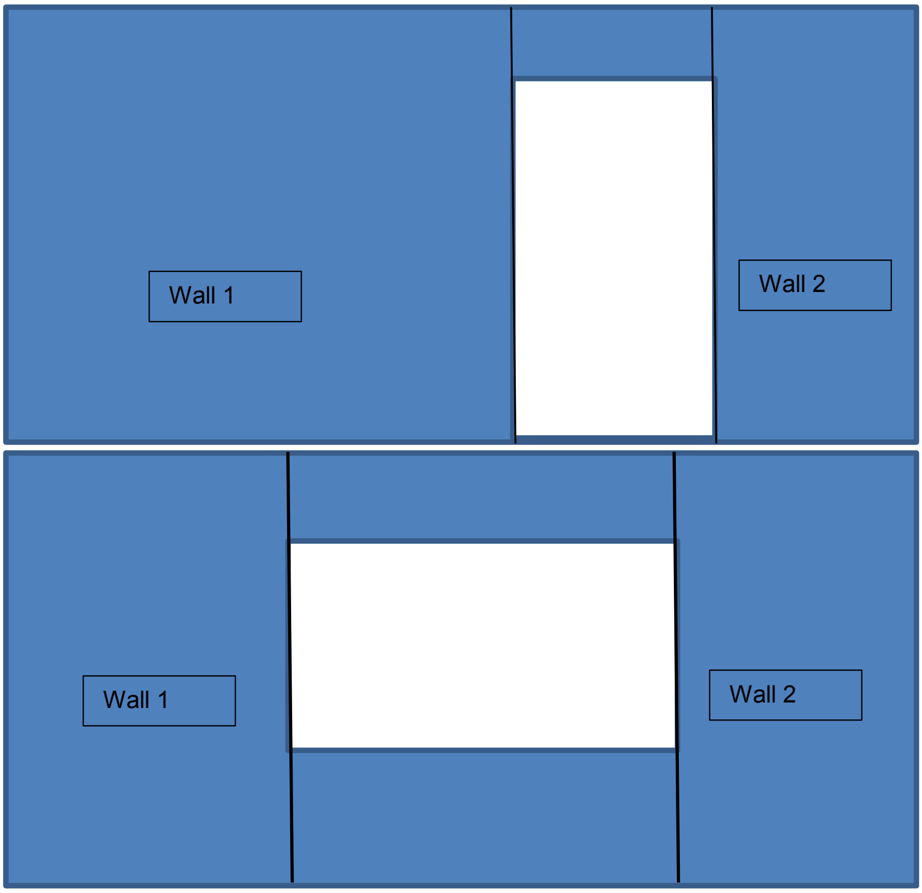

Figure 10 Potential coupling actions between cantilever walls

Li (2005) conducted a study on seismic performance of LTF buildings. A few single-storey wall configurations were studied using non-linear time history analyses where non-linear behaviour of fasteners from sheathing to timber framing and hold-down details were calibrated against test results. The study concluded that the strength (therefore stiffness) enhancement by the coupling actions was very significant in comparison with the sum of individual cantilever walls.

A 2D static analysis on a penetrated LTF wall has also been carried out in this study using the stiffness derived from typical P21 test results on plasterboard sheathed walls (Figure 11). The study revealed that stiffness performance of the LTF bracing wall systems, due to the presence of 400 mm deep lintel, increased to twice the total stiffness of the two cantilever elements.

The shorter the lintel beam spans are, the more significant the stiffness increase is due to the coupling action. The deeper the lintel beams are, the more significant the stiffness increase is due to the coupling action.

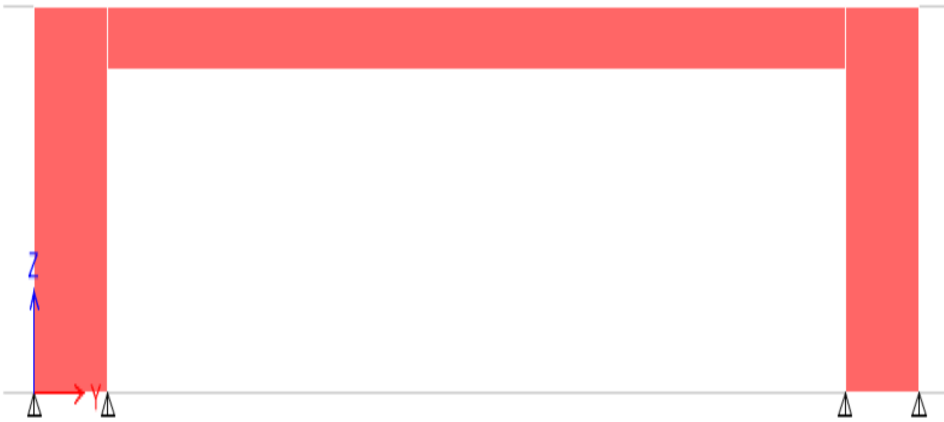

Figure 11 Coupling between two walls

It was this type of reserve bracing capacity in LTF bracing walls that prevented many regular LTF buildings from being subjected to significant damage during the Canterbury earthquakes.

It has to be realised that, where the specifically designed bracing systems are required, there is often no this type of robustness. As a consequence, there are often significant stiffness incompatibilities between specific bracing systems and conventional LTF bracing walls. This was perhaps the cause why LTF building with mixed bracing systems often had more significant damages as observed in Canterbury earthquakes.

3.6 Concluded seismic performance of minimum bracing provision

In conclusion, there is a mismatch between the seismic bracing provision and the seismic demand calculation in NZS 3604. The minimum earthquake bracing design of NZS 3604 could potentially be inadequate in terms of stiffness requirement, according to NZS 1170.5. To satisfy the current Code-specified performance criteria seismically and also control the earthquake damage to LTF residential buildings, seismic bracing demands determined according to NZS 3604 need to increase by 50%.

However, there are potential redundancies (the ‘system effect’) in LTF construction, such as the potential coupling actions due to the presence of wall beams at wall bases or wall tops. This sort of robustness in LTF building construction has never been quantified but potentially exists. Carefully consideration needs to be given to where, when and how much this type of robustness can be relied on. Unless the standard has clear specifications to make sure that this type of robustness exists, there is no certainty whether or not these reserve capacities could be relied on.

Clearly the assessed achievement level for an LTF building with minimum seismic provisions based on NZS 3604 is inappropriate for being used to establish the performance requirement in specifically designed bracing systems.

3.7 Performance requirements for specifically designed bracing systems

In areas where specific bracing elements are required, there is usually no reliable robustness to be counted on. Therefore, the performance requirements for specifically designed seismic bracing elements have to be established on the basis of assuming that there is no such robustness available.

What are the appropriate performance criteria for light timber-framed residential buildings, with specifically designed bracing elements included, to achieve damage control?

In this guidance, the damage control limit state is established as a storey drift of 1.0%. This is because the rated bracing capacity of plasterboard sheathed LTF walls of reasonable lengths is likely to be the peak bracing strength at a lateral deflection of 22 mm. A lateral deflection of 22 mm is approximately equal to a storey drift of 1%. The seismic hazard level used for verifying the damage control limit state for LTF residential buildings is taken as the design event of a 500-year return period. This is because the rated bracing capacity by P21 test is used for seismic bracing design at ULS. The definition of 1% drift as the damage control limit is also consistent with the defined limit for damage control by FEMA P-807 (ATC 2012).

For one or two-storey LTF buildings within the scope of NZS 3604, the engineers / designers only need to ensure that all storeys of the buildings meet the 1% storey drift limit.

A step-by-step design procedure for specifically designed seismic bracing elements is described in section 4.

4. SEISMIC DESIGN OF SPECIFIC BRACING SYSTEMS IN LTF BUILDINGS

4.1 General

The design procedures described here are to be used for specifically designed seismic bracing systems within a mainly NZS 3604 construction where the rest of the bracing designs follow NZS 3604.

The engineering basis underlying the design procedure of specifically designed seismic bracing systems is a displacement-based approach and the target deflection limit at ULS is 1% storey drift (see section 3.7).

Different bracing systems have different damping levels at a deflection equivalent to 1% inter-storey drift. Most specifically designed bracing systems other than LTF walls will have less damping than 20%. For example, steel portals are likely to be still in elastic range under seismic actions at ULS and the damping level is about $\xi = 5\%$. The overall damping level of the seismic bracing systems is the weighted damping of all the lateral load-resisting systems within the concerned area.

The procedure for specifically designed seismic bracing systems, although developed based on a displacement-based approach, is presented as following in a way most structural engineers working in residential buildings are familiar with.

4.2 Design procedures

Step 1: Base EQ design action allocated to specifically designed bracing system is

$$V_{b,\,sp} = C_d(T_1)W_{sp} \qquad \text{4-1}$$

as per NZS 1170.5: 2004 by assuming $T_1 = 0.4$ (s) and $\mu = 3.5$

where:

W_{sp} is the seismic weight associated with the tributary area allocated to the specific bracing system and it is determined based on the flexible diaphragm method. This assumption is believed to be prudent given the flexibility of ceiling and floor elements (Cobeen et al. 2004).

$C_d(T_1)$ = horizontal seismic design action coefficient as per NZS 1170.5

Step 2: Design EQ action for specifically designed bracing system in the principal axis of the specific bracing element is

$$V_{d,\,sp} = 1.5 \times \beta \times \alpha\, V_{b,sp} \qquad \text{4-2}$$

where:

1.5 is a constant coefficient, derived from section 3.4, based on the assumption that the damping level of the bracing systems is 20%.

β is the factor to adjust the seismic actions when the overall damping level of the bracing systems is different from 20% and is calculated as in Step 3.

α is a coefficient to allow for plan irregularity as determined in step 4.

Step 3: Derivation of β

β is calculated as follows:

$$\beta = \frac{R_\xi}{R_{\xi=20\%}} \qquad \text{4-3}$$

where:

$R_\xi = \sqrt{\frac{7}{2+\xi}}$ and $R_{\xi=20\%}$ is 0.56

ξ is the overall damping level expressed as percentage, which is the weighted damping of all bracing systems.

For example, for steel portal frames, the portals are likely to be still in elastic range under seismic actions at ULS and its damping level is about ξ = 5%. In this case, there are two different bracing systems for the area under consideration – LTF bracing walls with a damping of 20% and steel portals with a damping of 5%.

Assuming the tributary area to LTF bracing walls equals the tributary area to steel portals for the concerned area, overall damping of these two different types of bracing systems is taken as:

$$\xi = \frac{A_{LTF} \times 20\% + A_{SP} \times 5\%}{A_{LTY} + A_{SP}} = \frac{0.5\times 20\% + 0.5\times 5\%}{1} = 12\%$$

where:

A_{LTF} is the tributary area to LTF bracing walls and A_{SP} is the tributary area to specifically designed steel portal frames for the concerned area.

In this case, $R_\xi = 0.71$, $R_{\xi=20\%} = 0.56$, β = 0.71/0.56=1.3, then

$V_{d,sp}$= $1.5 \times \beta \times \alpha\, V_{b,sp}$ = 1.5x1.3 $\alpha V_{b,sp}$ = 2.0 $\alpha\, V_{b,sp}$

Therefore, the seismic design actions at ULS for specifically designed bracing systems, which remain in elastic range at ULS earthquake events, should be taken as twice that as derived from NZS 3604.

Step 4: Effect of plan irregularity on design action for specific bracing systems

Plan irregularity of bracing arrangement will have an effect on design action for specific bracing systems because the floor diaphragm will not be absolutely flexible.

Plan irregularity is quantified based on eccentricity between the centre of bracing resistance (CR) for the storey under consideration and the centre of actions from the storey/roof above (Figure 12) (ATC 2012).

The centre of bracing resistance (CR) of all the bracing elements for the storey under consideration is defined as follows:

$$CR_x = \frac{\sum_{i=1}^{N} f_{i,x} x_i}{\sum_{i=1}^{N} f_{i,x}} \qquad 4\text{-}4$$

$$CR_y = \frac{\sum_{j=1}^{M} f_{j,y} y_j}{\sum_{j=1}^{M} f_{j,y}} \qquad 4\text{-}5$$

where:

CR_x is x coordinate of the building's centre of resistance

CR_y is y coordinate of the building's centre of resistance

$f_{i,x}$ = rated seismic bracing capacity of wall bracing element, i, in x direction

$f_{j,y}$ = rated seismic bracing capacity of wall bracing element, j, in y direction

x_i = x coordinate of the centre of the wall bracing element, i

y_j = y coordinate of the centre of the wall bracing element, j

i and j = the number of bracing elements respectively in x direction and y direction.

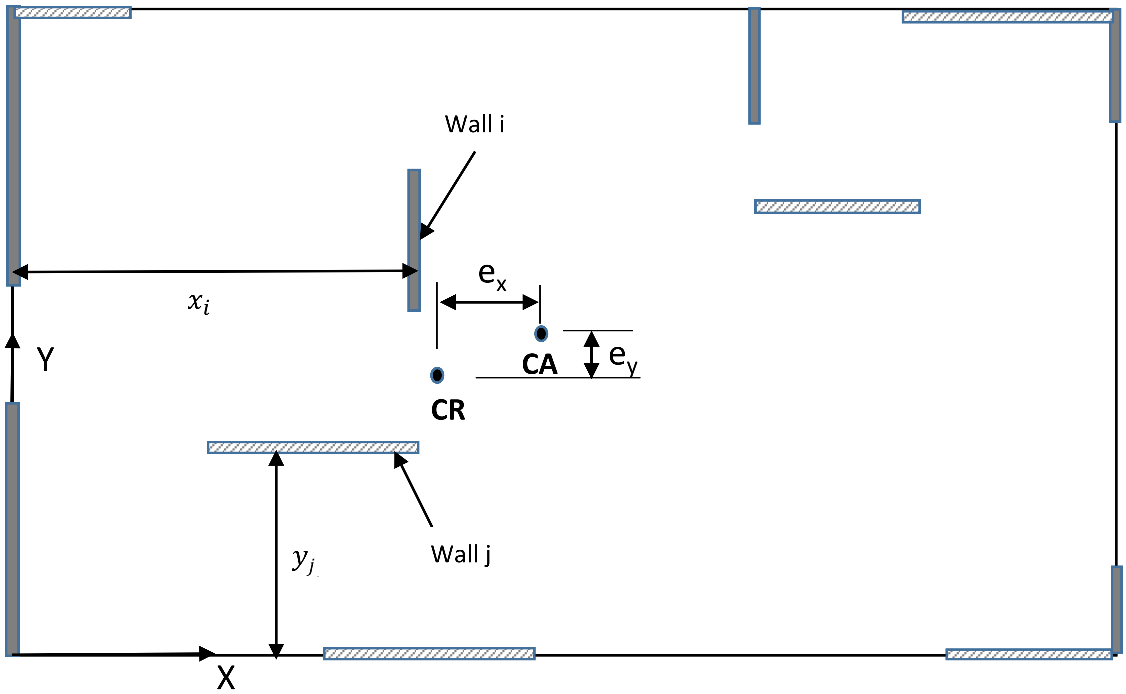

Figure 12 Floor diagram showing eccentricities in both directions

Centre of the seismic actions (CA) of the storey above is assumed to be the same as the geometrical centroid of the floor/roof diaphragm above. Coordinates of CA are CA_x and CA_y respectively in the x and y directions. This is reasonable because mass irregularity has insignificant effect on structural seismic response parameters (Sadashiva 2010).

Eccentricities are the absolute values of the differences in the coordinates of the CR and CA, and they are determined as:

$$e_x = |CR_x - CA_x| \qquad 4\text{-}6$$

$$e_y = |CR_y - CA_y| \qquad 4\text{-}7$$

where:

e_x is eccentricity in x direction and e_y is eccentricity in y direction

α is a parameter to allow for the torsional effect due to plan irregularity, and it is simplified as (ATC 2012):

$$\alpha = 4 \times \frac{e_x + e_y}{L_x + L_y} + 1.0 \qquad 4\text{-}8$$

where:

L_x = length of the building in x direction

L_y = length of the building in y direction.

Step 5: Deflection performance requirement

The deflection performance requirement for the specifically designed bracing elements at ULS shall be

$$\Delta_{\text{storey drift}} < 1.0\% \qquad 4\text{-}9$$

where:

$\Delta_{\text{storey drift}}$ is the deflection of the specifically designed bracing systems under the seismic action $V_{d,\,sp}$, as derived in Step 2.

This is generally sufficient to achieve compatible deformation performance with conventional LTF walls. If the specific bracing element needs to support brittle non-structural components, a more strict deflection criterion may be required.

Step 6: Strength requirement

Finally, a strength check needs to be completed for specific bracing systems.

The seismic action for strength design is determined using equation 4-2.

Step 7: Floor/ceiling diaphragm design

The floor/ceiling diaphragm that links the specifically designed bracing systems to the NZS 3604 designed part of the building needs to meet the following criteria:

1. The dimensions of any penetration of the floor/ceiling should not be greater than 30% of the total building dimensions in either direction.
2. The floor/ceiling diaphragm chords perpendicular to the axis of the specifically designed bracing system must be continuous over the area spanning between the specifically designed and conventional LTF wall bracing systems. They must also continue over the rest of the floor diaphragm to where effective bracing systems in the perpendicular direction is available. By definition, chords being continuous means that, should joints in the chords be present, an axial tensile and compression capacity over each joint is at least 6 kN.

5. DISCUSSIONS AND CONCLUSIONS

LTF residential buildings, which are mainly constructed to NZS 3604, often contain specifically designed seismic bracing systems. In the Canterbury earthquake sequence, LTF residential buildings including specifically designed bracing systems often sustained more significant earthquake damages due to stiffness incompatibility between specifically designed and conventional LTF wall bracing systems. The objective of this study was to develop a rational procedure so that specifically designed bracing systems are properly designed and have compatible stiffness/deformation performance with the conventional LTF bracing walls.

The engineering characteristics of LTF residential buildings were theoretically examined. The examination has concluded that the deflection criterion at ULS, rather than the life safety criterion as required by the current code, is a more appropriate performance criterion for LTF residential buildings. The design basis of NZS 3604 was examined to establish the appropriate deflection performance criterion and achieve stiffness/deformation compatibility design for specifically designed seismic bracing systems. The expected deflection performance of an artificial regular LTF building with minimum NZS 3604 bracing provision was studied. The study of the artificial LTF building was according to a displacement-based approach and based on available P21 test results of plasterboard sheathed LTF walls. Finally, the performance criteria for specifically designed bracing systems were established, based on P21 test results on plasterboard sheathed LTF walls.

The conclusions obtained from these studies are summarised below.

Findings on the seismic design clauses of NZS 3604

1. Minimum seismic bracing provision according to NZS 3604 will potentially result in a building, which has to deflect well beyond the code-specified deflection limit of 2.5% storey drift at ULS. Minimum seismic bracing provision as per NZS 3604 would appear to need to increase by 50% in order that the deflection requirement of the current seismic loading standard NZS 1170.5 is satisfied.

2. LTF bracing systems often have redundancies, which will significantly enhance the stiffness/strength performance of the structure. However, the enhancement effect on the building's seismic performance varies, depending on the locations of the redundancies and the floor/roof diaphragm stiffness. It is suggested that these reserve capacities be quantified and the effect of the diaphragm flexibility on the utilisation of these reserve capacities be studied.
3. It is suggested that the effects of allowable irregular bracing arrangements within the scope of NZS 3604 on the seismic bracing requirement be studied. This is because the torsional effect is likely to lead to further increase in seismic bracing demand.

Conclusions on specifically designed bracings systems

1. Performance criterion at ULS for specifically designed bracing systems was established to be 1% in terms of storey drift, based on observed P21 test results on common plasterboard sheathed LTF walls. This is in order that specifically designed seismic bracing systems are compatible with conventional plasterboard sheathed LTF bracing walls.
2. Seismic design actions used for verifying deformation performance of specifically designed seismic bracing systems should be determined based on tributary area theory and expected overall damping level of various bracing systems. Furthermore, the effect of irregular bracing arrangements on the seismic demand of specifically designed bracing systems should be properly allowed for.

Finally, a step-by-step seismic design procedure, developed using a displacement-based approach, was presented for specifically designed bracing systems within an LTF residential building of mainly NZS 3604 construction.

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