16th World Conference on Earthquake Engineering, 16WCEE 2017

Santiago Chile, January 9th to 13th 2017

Paper N° 1279

Registration Code: S-Z1463221312

THE NEED FOR A SYSTEMATIC APPROACH IN DAMAGE CONTROL DESIGN FOR LIGHT TIMBER-FRAMED BUILDINGS IN EARTHQUAKES

A. Liu[1]

[1] *Senior Structural Engineer, BRANZ Ltd, angela.liu@branz.co.nz*

Abstract

Performance-based seismic design of building structures has advanced significantly over the last decade. The essence of performance-based seismic design is to promote multiple performance requirements including various damage control limits rather than one single requirement – life safety – as in the current New Zealand Building Code. The advancement of performance-based seismic design has been mainly limited to reinforced concrete structures, and there has been very little development in light timber-framed (LTF) buildings. In reality, damage control design is more necessary for LTF buildings than for other heavy structures.

Unlike heavy building structures, which could collapse in earthquakes due to large P-Δ induced instability of the gravity-resisting systems, LTF buildings have a low risk of collapse in earthquakes due to the light nature and being wall structures.

New Zealand Standard NZS 3604:2011 *Timber-framed buildings* is a prescriptive standard developed for LTF buildings in New Zealand. In deriving seismic demand, the engineering basis underlying NZS 3604:2011 is an elemental approach developed using a force-based design approach assuming a global ductility of 3.5. It has minimal allowance for deformation incompatibility between LTF walls. NZS 3604:2011 specifies a test procedure, P21, for evaluating the seismic resistance of bracing systems, which are commonly gypsum plasterboard walls in New Zealand. According to the P21 procedure, bracing walls are cyclically tested as cantilever walls, and the attained residual strength within a deflection range is the rated earthquake bracing capacity. Stiffness compatibility and composite action between different LTF walls are not considered, except for the requirement to rate the wall within a specified displacement range.

Examination of construction practices and engineering characteristics of LTF buildings reported here has reinforced the belief that the damage control limit state, rather than life safety, is a more appropriate performance requirement for LTF buildings.

A case study LTF residential building with minimum NZS 3604:2011 seismic bracing provided by plasterboard bracing walls was conducted using a displacement-based approach and available bracing test results of plasterboard walls. The study revealed potential shortcomings of current seismic design specifications, and the findings are summarised as follows:

(1) The expected seismic performance level of LTF buildings with minimum NZS 3604:2011 seismic bracing provided by plasterboard bracing walls would be expected to deflect well beyond the Code-specified deflection limit of 2.5% storey drift at the ultimate limit state. The current seismic bracing provision of NZS 3604:2011 potentially needs to increase by 40% in order that the displacement of LTF buildings can be adequately controlled.

(2) It is suggested that the effects of irregular bracing arrangements on the seismic performance of LTF buildings be studied by allowing for the semi-rigid nature of the floor/ceiling diaphragms.

(3) A displacement-based systematic approach is more appropriate to achieve consistent seismic damage control design for LTF buildings. The systematic approach should allow for the interactions between bracing systems and floor/ceiling diaphragms.

Keywords: seismic performance; diaphragm; light timber-framed buildings; storey drift; deformation incompatibility

1. Introduction

Performance-based seismic design of building structures has significantly advanced over the last decade. The costly earthquakes occurring in recent years around the world have provided renewed impetus to speed up the move from a single requirement "life safety" towards performance-based seismic design for building structures. The essence of performance-based seismic design of building structures is to design a building structure for multiple performance requirements (damage limit objectives) rather than one single requirement of life safety at ultimate limit state (ULS) as per the current seismic design standard.

New Zealand has a performance-based Code environment. However, the current seismic design standards are generally prescriptive and were developed to achieve life safety only at ULS and deflection control at the serviceability limit state (SLS). Extensive theoretical studies and earthquake damage observations showed that the building structures designed to modern codes, although likely to achieve life safety in an event equivalent to the ULS intensity, varied significantly in their earthquake damage performance. This is mainly because earthquake damage to building structures results from the excessive deformation experienced by the buildings. The force-based approach, which is the predominant methodology used in current seismic design standards, is inadequate in assessing seismic deflections [1, 2, 3]. Therefore, current seismic design standards are unable to provide buildings with uniform protection against earthquake damage [4]. As a consequence, it is often not clear what performance levels, except life safety, could be expected of the building structures designed to modern seismic codes. While there has been research on performance-based seismic designs of reinforced concrete building structures, there has been very limited research on performance-based seismic design of light timber-framed (LTF) buildings as are commonly built in New Zealand.

In New Zealand, the majority of residential buildings are low-rise LTF buildings, and both the gravity load-resisting systems and lateral seismic load-resisting systems are LTF plasterboard-lined walls. Seismic designs of LTF residential buildings in New Zealand have largely followed the prescriptive standard NZS 3604:2011 *Timber-framed buildings* [5], where the seismic design requirements were developed according to the force-based approach. Therefore, the expected seismic performance of LTF buildings, except for ensuring life safety, is likely to be inconsistent.

There is a need for advancing performance-based seismic design of LTF buildings. The objectives of this paper are: (1) to quantify the expected seismic damage performance of LTF buildings when designed to NZS 3604:2011 using the direct displacement-based approach; (2) to identify potential critical structural weakness of LTF buildings that could affect the building's seismic performance on a global scale; and subsequently (3) to identify the future research needs for advancing performance-based seismic design of LTF buildings in New Zealand.

2. Damage observation

Earthquake damage to LTF buildings is a result of deformations occurring within the building structure. In the Canterbury, New Zealand, earthquake sequence in 2010-2011, LTF buildings all achieved the Code-specified objective of life safety. However, the earthquake damage was often very significant. Of particular interest is that earthquake damage to LTF buildings varied significantly. Some LTF buildings had no damage at all, while other LTF buildings were badly damaged and had to be demolished. With regard to performance-based seismic engineering design, the observed seismic performance of some LTF buildings was not adequate.

Fig. 1 and Fig. 2 show the significant damage observed on some modern plasterboard-sheathed LTF walls in the Canterbury earthquake sequence [6]. While the inconsistency of observed earthquake damage to LTF structures is expected because NZS 3604:2011 uses a force-based approach in calculating seismic demand, a close examination of the assumptions made in the engineering basis of NZS 3604:2011 for seismic design is needed to get a better understanding of the causes of inconsistent seismic damage performance.

Fig. 1 – Failure of plasterboard/metal brace

Fig. 2 – Failure of plasterboard internal linings.

3. Seismic engineering characteristics of LTF residential buildings in New Zealand

3.1 General

NZS 3604:2011 has been developed for constructing simple small-scale LTF buildings. For typical LTF buildings, the lateral seismic-resisting systems are plasterboard-lined LTF walls, and therefore, LTF buildings perform in earthquakes in a similar way to other wall structures. The plasterboard-lined shear walls are also the gravity load-carrying elements of LTF buildings. 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, P-Δ effects usually are not significant enough to cause instability problems. Therefore, the LTF buildings of mainly NZS 3604:2011 construction could easily achieve life safety requirements in design earthquakes.

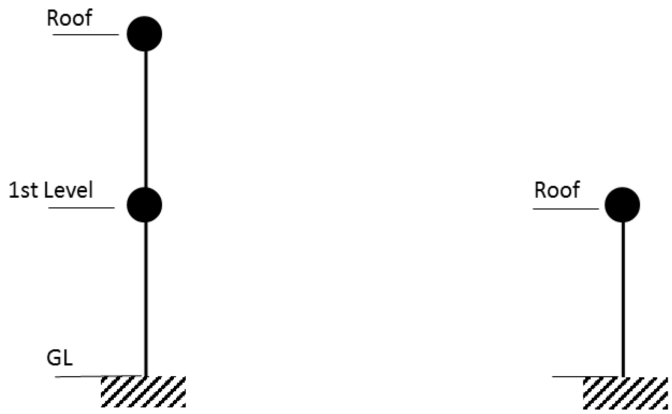
Other building structures often have skeleton members as gravity systems, such as reinforced concrete frames. Collapse associated with life safety could occur when the gravity load-carrying systems become unstable due to the combined effects of seismic actions and P-Δ actions, after the buildings have undergone significant earthquake-induced lateral deflections.

The reported collapse limit state for low-rise wooden buildings overseas could reach a storey drift of 7% or even more [7, 8]. The construction of LTF buildings in New Zealand is similar to the construction of wooden buildings in other countries except for the wall lining materials. Hence, the collapse limit state of LTF buildings in New Zealand should be somewhere around 7% as reported overseas. A storey drift of 7% is significantly higher than the ULS deflection limit of 2.5%, as specified in the New Zealand seismic loading standard NZS 1170.5:2004 *Structural design actions – Part 5: Earthquake actions – New Zealand* [9]. This means that the life safety requirement at ULS as per NZS 3604:2011 is not very meaningful.

While LTF buildings have a low probability of collapse in earthquakes, they are vulnerable to earthquake damage, and their seismic performance can vary from building to building. Apart from the application of a force-based approach to the seismic design, many other factors have also contributed to the inconsistent seismic performance of LTF buildings as observed in the Canterbury earthquake sequence. For instance, LTF buildings often have significant structural weaknesses, such as irregular arrangement of bracing elements across the floor plan, stiffness/deformation incompatibilities of different bracing systems and less-effective floor diaphragms for distributing the seismic actions to different bracing systems across the building. It is well known that, when significant structural weaknesses are present, the seismic actions induced in different bracing systems can significantly deviate from a force-based theoretical prediction according to NZS 3604:2011, causing greater variations of the expected seismic performance.

3.2 Seismic bracing provision of NZS 3604:2011

NZS 3604:2011 has clauses that specify the seismic design requirements for LTF buildings in New Zealand. The seismic bracing demand was derived using an equivalent static method as recommended by NZS 1170.5:2004, assuming a fundamental period of 0.4 seconds and a ductility of μ=3.5. The model used to derive the seismic demand in NZS 3604:2011 is an elemental lumped mass model as shown in Fig. 3 [10].



(a) Two-level building (b) Single-level building

Fig. 3 – Model used in deriving earthquake demand in NZS 3604:2011

The seismic base shear for the entire building is calculated according to the equivalent static method, according to Eq. (1) as follows:

$$V_b = C_d(T_1)\, W_t \qquad (1)$$

where:

V_b = horizontal seismic shear force at the base of the structure

$C_d(T_1)$ = horizontal design action coefficient

W_t = total seismic weight of the building.

$$C_d(T_1) = C(T_1)\, S_p / K_\mu \qquad (2)$$

where:

S_p = structural performance factor ($S_p = 0.7$ when $\mu > 2.0$).

$$C(T_1) = C_h(T)\, Z\, R\, N(T,D) \qquad (3)$$

where:

k_μ = inelastic spectrum scaling factor, calculated based on assumed T_1 and ductility μ

$C_h(T)$ = spectral shape factor at $T_1 = 0.4$ s, determined based on soil class.

Z = hazard factor

R = return period factor at ULS (= 1.0 for importance level 2 buildings covered by NZS 3604:2011)

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

The seismic action distributed to floor level i is calculated from Eq. (4) with an additional 8%V_b applied at the roof level:

$$F_i = 0.92 V_b \, m_i h_i / (\sum m_i h_i) \quad (4)$$

where:

m_i = seismic mass lumped at level i

h_i = height from the ground to floor level i or roof.

For example, for soil class D as per NZS 1170.5:2004 and Z = 0.46, $C_d(T_1)$ = 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 in NZS 3604:2011.

For the provision of bracing capacity, NZS 3604:2011 adopted the P21 test procedure [11], developed by BRANZ, to evaluate the seismic and wind bracing capacity of proprietary sheathed LTF wall elements. 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 of 15 mm, 22 mm, 29 mm or 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. The determined rating may be applied to walls up to twice the length of the tested wall.

As for the bracing distribution within a building plan, NZS 3604:2011 has limited provisions for quantifying the effects of structural irregularity. NZS 3604:2011 requires the bracing systems to be spaced at not more than 6 m, resulting in notional bracing lines. According to NZS 3604:2011, each bracing line needs to have a minimum bracing provision of 50% of the total bracing demand divided by the number of bracing lines. For external walls, the bracing capacity must also be greater than 15 bracing units per metre of external wall lengths. It is noted that 1 kN is 20 bracing units.

4. Expected earthquake damage performance of minimum NZS 3604:2011 seismic bracing provision

4.1 General

Observed earthquake damage to LTF buildings in Christchurch showed significant earthquake damage variation between LTF buildings. For the development of performance-based seismic design of LTF buildings, it is important to understand the causes of the damage variation in order to minimise the performance inconsistencies.

Studies and earthquake observations have frequently shown that earthquake damage to LTF buildings is caused by excessive inter-storey drift [12]. Therefore, inter-storey drift is a key indicator for measuring the earthquake performance of LTF buildings.

The seismic resistance of any lateral load-resisting system depends on not only its strength but also its deformation capability and energy-dissipating capacity. Direct displacement-based seismic design of building structures sets up the lateral deflection limits at the beginning of the design process. Therefore, it fits into the performance-based seismic design of building structures nicely. There has been active research applying direct displacement-based approaches to performance-based seismic design of building structures [13].

In the following section, the direct displacement-based approach is used to quantify the expected seismic performance of LTF buildings designed to NZS 3604:2011 in an earthquake equivalent to an event at ULS as specified by NZS 1170.5:2004.

4.2 Case study building

The case study LTF building is defined as follows:

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

(2) Structural bracing is provided by numerous gypsum plasterboard LTF walls. All the walls are 2.4 m high and 1.2 m long. The subsoil classification is D, and the earthquake hazard zone is Z = 0.46 as per NZS 1170.5:2004.

(3) The provided bracing capacity, V_{cap}, is exactly equal to the bracing demand calculated as per NZS 3604:2011, $V_{cap} = C_d(T_1) \times W_t = 0.4\, W_t$.

where:

W_t = the total seismic weight of the building.

4.3 Expected seismic performance of the case study building

The case study building is perfectly regular, and all the bracing elements are exactly the same. Therefore, a 2D elemental study is sufficient.

Step 1: Establishment of target displacement at ULS, namely the performance requirement

Fig. 4 shows a typical hysteresis loop of a 1.2 m long and 2.4 m high plasterboard-sheathed LTF wall generated during P21 testing. For the wall represented by the hysteresis loop in Fig. 4, the bracing rating was obtained at a deflection of 22 mm, and significant degradation was observed after 22 mm. An absolute differential deformation of 22 mm over a 2.4 m storey height equals a storey drift of about 1%. Therefore, it is rational to define 1% drift as the required deformation limit at ULS for LTF buildings. It is noted that the definition of 1% drift is also consistent with the defined limit for damage control by FEMA P-807 [12].

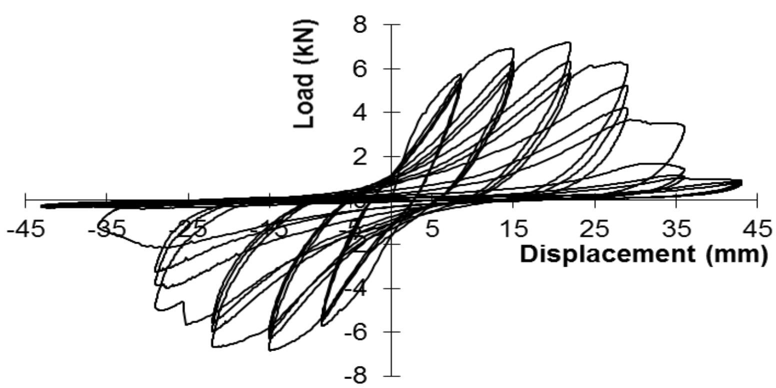


Fig. 4 – Typical hysteresis loop for a 1.2 m long LTF wall with plasterboard sheathing

Step 2: Determination of the equivalent viscous damping, ξ_{eq}, corresponding to the target displacement

Equivalent viscous damping is the parameter representing the energy-dissipating characteristics of a building's lateral seismic-resisting systems. For LTF buildings in New Zealand, the lateral load-resisting systems are commonly plasterboard LTF bracing walls, and equivalent viscous damping could be determined from the hysteresis characteristics of plasterboard LTF bracing walls.

For the case study building, the equivalent damping, ξ, of an LTF bracing wall element at a target lateral deflection of 22 mm is calculated 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 [1].

Step 3: Determination of the equivalent elastic period of the building, T_{eq}

The equivalent elastic period of the case study building, T_{eq} is obtained directly from the design displacement response spectrum generated for the building site and by considering the building as a single degree of freedom oscillator.

In constructing the displacement response spectrum based on the displacement-based method [4] and according to NZS 1170.5:2004, the seismic coefficient is calculated using Eq. (5) as follows:

$$C_d = S_p\, C_h(T_{eq})\, Z\, R_\xi \qquad (5)$$

where:

S_p = the structural performance factor, S_p = 0.7

$C_h(T_{eq})$ = the spectral shape factor for the site

Z = the hazard factor, Z = 0.46.

R_ξ is the reduction factor of seismic action due to the damping effect, and it is calculated using Eq. (6) [4]:

$$R_\xi = \mathrm{SQRT}(7/(2+\xi)) \qquad (6)$$

where:

ξ = the damping ratio, as a percentage.

As a result, the response displacement spectra is constructed according to Eq. (7):

$$\Delta = C_d\, (T_{eq}/(2\pi))^2\, g \qquad (7)$$

The design displacement response spectrum generated for the case study building is shown in Fig. 5. From Fig. 5, the equivalent elastic period of the building at the target deflection of 22 mm is determined as T_{eq}= 0.4 s.

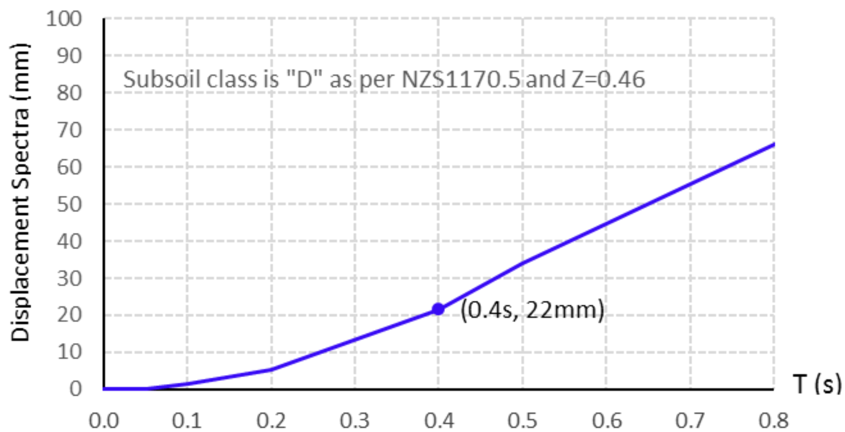


Fig. 5 – Displacement response spectrum

Step 4: Determination of the required equivalent lateral stiffness, K_{eq}

Representing the building as an equivalent linear single degree of freedom system, the required equivalent lateral stiffness is derived from Eq. (8):

$$K_{eq} = 4\pi^2\, W_t/(g\, T_{eq}^2) = 4\pi^2\, M/\, T_{eq}^2 \qquad (8)$$

where:

M = the seismic mass of the building in kg and it is calculated as W_t/g

W_t = the building's seismic weight in N

g = the acceleration of gravity.

The unit of K_{eq} is in N/m. Therefore, for this example, K_{eq} = 247M N/m.

Step 5: Lateral stiffness verification

The actual equivalent lateral stiffness, $K_{a,eq}$, is calculated, based on the provided bracing capacity V_{cap} and the target displacement Δ_t where $V_{cap} = 0.4\ W_t$ and $\Delta_t = 22$ mm.

$K_{a,eq} = V_{cap}/\ \Delta_t = 0.4W_t = 0.4Mg/0.022 = 178M$ N/m, which is about 72% the required lateral stiffness.

Therefore, the provided bracing capacity from NZS 3604:2011 is about 40% less than the required bracing capacity if the lateral storey drift is to be limited to 1% at ULS.

4.4 What to expect in a 500-year event for the case study building?

As demonstrated above, the case study building with the minimum bracing provision as per NZS 3604:2011 would need to be able to deflect significantly beyond 22 mm in order to satisfy the current seismic design standard NZS 1170.5:2004. More deflection means less stiffness and a longer period.

Fig. 6 shows the constructed response spectra, expressed as response acceleration (seismic coefficient, S_a) versus response displacement (S_d) for the site of the case study building. For the case study building, which has a bracing strength (capacity) equivalent to $S_a = 0.4$, the bracing walls potentially need to deflect to 70 mm (3.0% storey drift) in a 500-year event even if the LTF bracing wall systems could maintain the bracing strength and the damping level of 20% beyond 22 mm. This has significantly exceeded the NZS 1170.5:2004 specified deflection limit of 2.5% at ULS.

Furthermore, for plasterboard-lined 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.

It is of interest that other researchers [3] reached similar conclusions about the minimum seismic bracing strength requirement as per NZS 3604:2011. This was based on studies using non-linear time history analyses with multiple earthquakes.

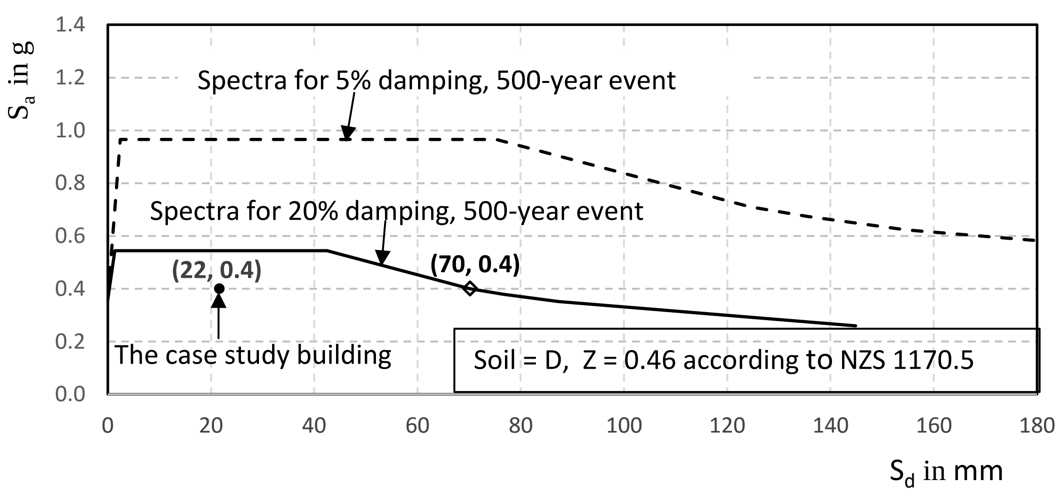


Fig. 6 – Constructed spectral acceleration (S_a) versus spectral displacement (S_d)

5. Discussion

5.1 Potentially excessive lateral deflections of NZS 3604:2011 construction

LTF buildings have a low risk to collapse in earthquakes, even after the buildings have undergone significant lateral deflections, which are well beyond the code specified deflection limits of 2.5% storey drift. However, excessive lateral deflections could cause significant damage and the buildings might have to be demolished after the earthquakes. Damage control is a more appropriate performance requirement for LTF buildings in earthquakes.

A damage controlled deflection limit was suggested as 1% inter-storey drift at ULS for LTF buildings, based on the test evidence of plasterboard bracing walls typical of LTF building construction in New Zealand. Numerous tests show that plasterboard bracing walls would likely have significant strength/stiffness degradation after 1% inter-storey drift.

The theoretical examination of the expected seismic performance of LTF buildings constructed to NZS 3604:2011, as described in section 4, was conducted on a case study perfectly regular LTF building designed to NZS 3604:2011. The theoretical study demonstrated that the seismic bracing provisions of NZS 3604:2011 potentially could result in excessive lateral deflections (significantly beyond 2.5% storey drifts), and this would be the case even without allowing for any detrimental effects of irregular arrangements of bracing elements. To achieve a damage controlled deflection limit of 1% inter-storey drift at ULS, the provided seismic bracing capacity of LTF buildings according to NZS3604 potentially needs to be increased by 40%.

This finding is no surprise because NZS 3604:2011 has used the equivalent static method, a force-based approach, in developing the seismic design clauses. Two assumptions in the force-based equivalent static method are usually responsible for its inadequacy in predicting the seismic displacement 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 buildings, the major reason why the current minimum NZS 3604:2011 seismic bracing provision is theoretically inadequate is because NZS 3604:2011 has overestimated the energy-dissipating capacity of typical LTF bracing walls. The force-based equivalent static method assumes a displacement ductility of $\mu = 3.5$ in simulating the energy-dissipating capacity of an LTF building. Consequently, the inelastic spectrum scaling factor, namely the reduction factor of design seismic action, $k\mu$, is 2.43. This has reduced the design seismic action to about 41% of the elastic seismic design action. However, the energy-dissipating capacity of typical gypsum plasterboard bracing walls, which are the common structural bracing for NZS 3604:2011 construction, is equivalent to a damping level of about 20% at the most when it is calibrated using P21 test results. For a damping level of 20%, the design seismic action is 57% of the elastic design action. As a consequence, the design seismic action from NZS 3604:2011 is about 40% less than the seismic bracing demand corresponding to the target displacement of 22 mm. The seismic bracing design principles underlying NZS 3604:2011 have significantly underestimated the seismic bracing demand. The consequence is that NZS 3604:2011 construction could potentially deflect excessively and have significant damage in design level earthquakes.

5.2 Deformation/stiffness incompatibility between plasterboard-lined LTF bracing walls

The LTF bracing walls in a structure often have varying lengths. The shortest LTF bracing walls may be only 400 mm in length, but the longest LTF bracing walls could be many metres long. LTF bracing walls of different lengths attain their peak bracing strengths at different deflection levels. Fig. 7 shows the comparison of the P21 test results on two plasterboard LTF walls where one wall is 1.2 m long and the other wall is 2.4 m long. Clearly, the two attained their peak bracing strengths at different deflection levels. Hence, significant deformation incompatibility between plasterboard bracing walls of different lengths is expected.

As a result, the induced seismic actions in bracing systems can significantly deviate from a force-based theoretical prediction as in NZS 3604:2011 [4]. In these circumstances, the rigidity of the floor or ceiling plays an important role in distributing the lateral seismic actions to the different bracing systems. For instance, an absolutely rigid floor or ceiling will force all bracing systems to be constrained to translate by the same amount, assuming no building rotation. For LTF buildings, the timber floors are neither rigid nor completely flexible. It is important that the rigidity is properly allowed for in order to adequately quantify the effects of the potential deformation/stiffness incompatibility between LTF bracing walls of significantly varying lengths on the overall seismic performance of the building.

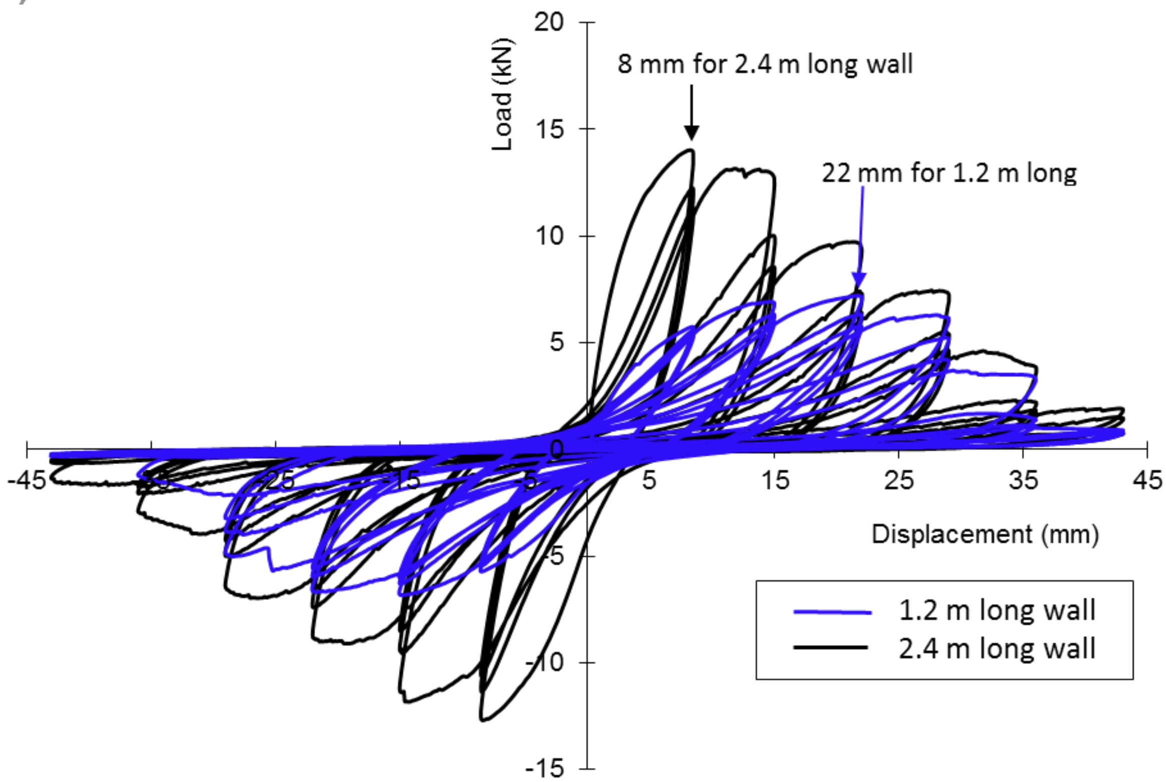


Fig. 7 – Hysteresis loops for plasterboard LTF walls of different lengths

5.3 Structural irregularity issue

In comparison with other building structures, LTF buildings potentially have significantly greater structural irregularities. For instance, the bracing walls in an LTF building are often arranged in an irregular manner across the floor plan, called plan irregularity. This often occurs due to the desire to make the best use of sunlight, outdoor living or to gain the best benefit from the view. As a consequence of plan irregularity, the building would tend to rotate about its centre of rigidity, resulting in greater bracing wall displacement demands on the perimeter of the building. In this case, how the seismic actions will be distributed to different bracing systems depends on the stiffness of the floor or ceiling. For example, a flexible floor or ceiling means that each bracing line is likely to be required to resist the seismic actions associated with the seismic weight within its tributary area. For LTF buildings, the timber floors are semi-rigid, and currently, there are no adequate methods for quantifying the effect of structural irregularities on the seismic performance of LTF buildings.

5.4 Potential seismic bracing redundancies

Contrary to the detrimental issues possibly present in an LTF building as described above, LTF buildings designed to NZS 3604:2011 often have significant redundancies (the ‘system effect’), which have beneficial effects on the seismic performance of the building.

One common redundancy is in the coupling elements between bracing wall elements. The bracing wall systems in LTF construction are usually not cantilever walls as 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 Fig. 8. As a consequence, the LTF bracing walls will be stiffer than the sum of the individual cantilever walls. However, there is a need to develop an adequate method for quantifying various stiffening effects of penetrated plasterboard walls in order to adequately predict the seismic performance of an LTF building in earthquakes.

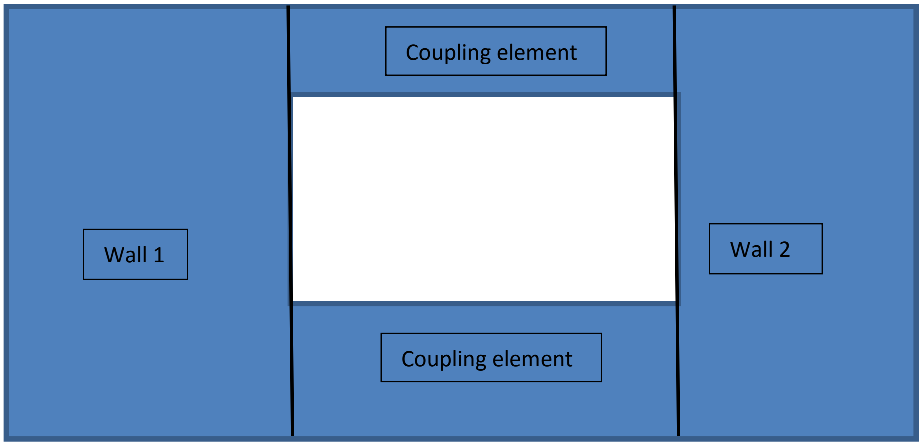


Fig. 8 – Coupling elements between cantilever LTF bracing walls

6. Conclusions

The engineering characteristics of LTF residential buildings have been theoretically examined. The examination has concluded that the damage control or deflection criterion at ULS, rather than the life safety (collapse prevention) criterion as in the current New Zealand Code, is a more appropriate seismic performance criterion for LTF residential buildings.

In New Zealand, LTF residential buildings are designed to a prescriptive design standard, NZS 3604:2011. The underlying seismic design philosophy and method of NZS 3604:2011 was examined, and the seismic bracing provision of NZS 3604:2011 is not expected to provide uniform protection against earthquake damage to LTF buildings because it is a force-based elemental approach. Using the displacement-based seismic design method, the requirements in NZS 3604:2011 have been shown to be inadequate for damage control.

Application of performance-based seismic design through the direct displacement-based approach to a case study regular LTF building with minimum NZS 3604:2011 bracing was conducted. Based on available P21 test results for plasterboard-sheathed LTF walls, the deflection requirement at ULS was determined to be 22 mm, which equals to a storey drift of 1%, and the 20% equivalent damping of LTF wall bracing elements at the deflection level of 22 mm was derived. The expected seismic performance of the LTF case study building was predicted.

The findings were as follows:

(1) The minimum seismic bracing provision according to NZS 3604:2011 will potentially result in a building, that will deflect well beyond the Code-specified deflection limit of 2.5% storey drift in a ULS design earthquake event. Therefore, significant damage to LTF houses is expected in ULS design level earthquakes. The minimum seismic bracing provision as per NZS 3604:2011 would likely have to increase by 40% in order for LTF buildings to not deflect excessively at ULS load levels.

(2) It is suggested that the effects of allowable irregular bracing arrangements within the scope of NZS 3604:2011 on the seismic bracing requirement be studied. This is because the torsional effect caused by the irregular bracing arrangement is likely to lead to significant amplification of lateral deflections in some parts of the building.

(3) LTF buildings often have load-resisting redundancies that 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 stiffness. It is suggested that these reserve capacities be quantified and the effect of the floor/ceiling flexibility on the utilisation of these reserve capacities be studied.

7. Acknowledgements

This research project was funded by the New Zealand Earthquake Commission and the Building Research Levy.

8. References

[1] Liu A (2015): *Design Guidance of Specifically Designed Bracing Systems in Light Timber-framed Residential Buildings*. EQC Report. http://www.eqc.govt.nz/research/research-papers/390-design-guidance-bracing-systems-light-timber-framed-res-bldgs

[2] Paulay T, Restrepo, JI (1998): A displacement and ductility compatibility in buildings with mixed structural systems. *Journal of the New Zealand Structural Engineering Society (SESOC),* **11** (1), 7–12.

[3] Newcombe MP, Batchelar ML (2012): Seismic design of plasterboard wall bracing systems. *Structural Engineering Society (SESOC) Journal*, **25** (2), 42–51.

[4] Priestley MJN, Calvi GM, Kowalsky MJ (2007): *Displacement-Based Seismic Design of Structures*. IUSS Press.

[5] Standards New Zealand (2011): NZS 3604:2011 *Timber-framed buildings*.

[6] Buchanan A, Carradine D, Beattie G, Morris H (2011): Performance of houses during the Christchurch earthquake of 22 February 2011. *Bulletin of the New Zealand Society of Earthquake Engineering*, **44** (4), 342–357,

[7] Paevere P, Foliente GC, Kasal B (2003): Load-sharing and redistribution in a one story woodframe building. *Journal of Structural Engineering*, **129** (9), 1275–1284.

[8] Bahmani P, van de Lindt JW (2013): Direct displacement design of vertically and horizontally irregular woodframe buildings. *Proceedings of Structures Congress*, Pittsburgh, USA, 1217–1228.

[9] Standards New Zealand (2004): NZS 1170.5:2004 *Structural design actions – Part 5: Earthquake actions – New Zealand*.

[10] Shelton R (2013): *Engineering Basis of NZS 3604*. BRANZ Ltd, Judgeford, New Zealand. http://www.branz.co.nz/Engineering_Basis_of_NZS_3604

[11] Shelton R (2010): *A Wall Bracing Test and Evaluation Procedure*. BRANZ Technical Paper P21. BRANZ Ltd, Judgeford, New Zealand. http://www.branz.co.nz/P21

[12] ATC (2012): *Seismic Evaluation and Retrofit of Multi-unit Wood-frame Buildings with Weak First Stories*. FEMA P-807. Federal Emergency Management Agency, Washington, DC.

[13] Filiatrault A, Folz B (2002): Performance-based seismic design of wood framed buildings. *Journal of Structural Engineering*, **128** (1), 39–47.