Bulletin of the New Zealand Society for Earthquake Engineering, Vol. 56, No. 1, March 2023

SEISMIC EFFECTS OF BRACING IRREGULARITY OF LIGHT TIMBER-FRAMED BUILDINGS

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(Submitted *February 2022*; Reviewed *April 2022*; Accepted *July 2022*)

ABSTRACT

Most residential buildings in New Zealand are low-rise light timber-framed (LTF) buildings, constructed according to a prescriptive standard – NZS 3604:2011 Timber-framed buildings. NZS 3604:2011 tabulates the seismic demand and also specifies the test procedure for evaluating the seismic resistance of proprietary LTF walls, which are often plasterboard walls. Designers need to ensure the provided total seismic bracing capacity is at least equal to the total seismic bracing demand, provided that bracing arrangements satisfy the specified irregularity limits.

The irregularity limits of bracing arrangements in NZS3604:2011 were established based on engineering rules of thumb rather than rigorous scientific evidence. Earthquake damage observed in the 2010/11 Canterbury earthquake sequence demonstrated that simple regular LTF houses performed well while irregular houses often had significant damage that was uneconomical to repair. This suggested that the irregularity of LTF buildings was an important factor responsible for the exacerbated earthquake damage.

To quantify seismic effects of permissible irregularities in NZS 3604:2011 and provide scientific evidence for elaborating irregularity limits in NZS 3604:2011, three single storey LTF buildings with varying degrees of permissible plan irregularities were designed and their seismic performance was studied by conducting three-dimensional non-linear push-over analyses. For the non-linear push-over analyses, the in-plane behaviour of LTF walls and ceiling diaphragms were modelled using the models, developed based on NZ practice, as reported in previous research.

The study revealed that permissible irregular bracing arrangements in NZS 3604:2011 could amplify lateral deflections significantly, in comparison with the regular counterparts. As a result, irregular LTF buildings within the scope of NZS 3604:2011 could be susceptible to damage due to excessive deformations in earthquakes and this suggests that the irregularity limits in current NZS 3604:2011 be reviewed and tightened.

https://doi.org/10.5459/bnzsee.56.1.29-37

INTRODUCTION

Residential buildings in New Zealand (NZ) are often low-rise light timber-framed (LTF) buildings constructed according to the prescriptive standard – NZS 3604:2011 Timber-framed buildings [1]. The majority of these buildings have plasterboard sheathed LTF walls as the gravity load-carrying and lateral load-resisting systems.

Although developed as a descriptive standard, NZS 3604:2011 has an engineering basis [2]. The seismic engineering basis in NZS 3604:2011 used a force-based approach in deriving the seismic design actions (demand). The force-based approach was the equivalent static method with a fundamental period of T1 = 0.45 second and a ductility of μ = 3.5 as in NZS 1170.5:2004 Structural design actions – Part 5: Earthquake actions – NZ [3]. In using NZS 3604:2011, the seismic demand is determined by reading from a predefined table, based on the soil classification, seismic hazard zone, house foundation type (concrete slab on ground or suspended timber floor) and building envelope weight. NZS 3604:2011 also specifies that the P21 test and evaluation procedure [4], developed by BRANZ, be used to evaluate seismic bracing capacity of proprietary LTF wall elements. Designers need to ensure bracing capacity is at least equal to bracing demand, provided that certain irregularity limits of bracing distribution are met. These irregularity limits were established based on engineering rules of thumb, rather than on rigorous scientific basis.

Earthquake damage observed in the 2010/11 Canterbury earthquake sequence demonstrated that LTF residential buildings all achieved the NZ Building Code objective of “safeguarding people from injury caused by structural failure”. However, the observed earthquake damage in Christchurch showed significant discrepancies [5]. Simple regular LTF houses generally did not have apparent damage unless the site experienced significant liquefaction related land damage. In comparison, irregular houses often had significant damage that was uneconomical to repair. This suggested that the irregularity of LTF buildings was an important factor responsible for the exacerbated earthquake damage.

The objectives of this study were to quantify the seismic effects of plan / bracing irregularities on the seismic performance of LTF buildings as typical of NZ construction and to provide a scientific basis for adjusting the current irregularity limits in NZS 3604:2011 if necessary.

ALLOWANCE FOR SEISMIC EFFECTS OF STRUCTURAL IRREGULARITY

General

Building structures often have various irregularities, plan (horizontal) irregularity is one of the most common structural irregularities in building structures. Plan irregularity arises

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when the centre of resistance is not coincident with the centre of the applied storey shear force at any level of the building.

When a building structure has plan irregularities, the building will have not only translational but also torsional responses. The translational seismic responses will be coupled with the torsional responses, leading to much more sophisticated seismic behaviour.

In addressing adverse seismic effects of structural irregularities, the current seismic design standards around the world are based on the same principles. Typically, the standards specify certain irregularity limits. Such irregularity limits are intended to determine the complexity level of the seismic analysis methods. While the specified irregularity limits provide some insights on the likely undesirable effects due to the presence of irregularity, these limits were developed by consensus, rather than being based on quantitative data. Generally, these limits are used to classify a building structure as either regular or irregular.

Similarly, current NZ seismic loading standard, NZS 1170.5:2004, specifies irregularity limits and classifies a building structure as either regular or irregular. NZS 1170.5:2004 recommends different analysis methods, based on the irregularity classification of the buildings as well as the sizes of the buildings. For instance, NZS1170.5:2004 allows the simplest analysis method “the equivalent static method” to be used if the subject building meets at least one of the following criteria:

- The height between the base and the top of the structure is less than 10 m.
- The largest translational period calculated is less than 0.4 seconds.
- The structure is not classified as irregular and the largest translational period is less than 2.0 seconds.

For building structures classified as irregular, more sophisticated analysis methods would be required – for example, the modal response spectrum method or time history dynamic analysis.

Research Review of Seismic Effects of Structural Irregularities

There have been significant research efforts made in understanding the seismic effects of horizontal structural irregularity of building structures and most of these efforts are about the seismic effects of structural irregularities of concrete buildings [6-10]. These studies brought about the realisation that current methods used in addressing the irregularity-related seismic torsional responses were fundamentally different from the modern seismic design philosophy “capacity design”. General findings are:

- Three-dimensional analyses with adequate stiffness modelling of lateral seismic-resisting systems and floor diaphragms are essential in studying seismic performance of building structures with structural irregularities.
- Rigidity of floor diaphragms plays an important role when structural irregularity exists [11].

LTF residential buildings are tailored for individual’s needs and they often have much greater irregularities, in comparison with concrete or steel structures. Nevertheless, the seismic design requirements for LTF residential buildings are traditionally less rigorous than other structures. There have been only limited research activities undertaken on the seismic effects of structural irregularities in LTF buildings. These activities were based on overseas practice and most of the study was about the diaphragm stiffness modelling technique. Some examples of research reviewed are summarised here.

Shake table testing and numerical modelling on a three-storey timber-framed house constructed following the Japanese building practice showed the significant influence of irregular plan configurations on the building performance [12,13]. Bracing irregularity due to large garage door opening caused severe torsional responses and excessive damage in the bracing walls. The study also indicated that the diaphragm rigidity needs to be adequately considered in the models in order to capture the overall building seismic response.

Effect of plan irregularity due to irregular plan configurations on seismic performance of single-storey wood-frame dwellings was studied by employing many surrogate building models [14]. In this study, the modelling technique of the roof diaphragm was not explicitly explained, and it appeared that the roof diaphragm was considered as rigid. A focus of the study was the occurrences of maximum drifts exceeding the 3% collapse prevention limit. The study concluded that irregular configuration tends to induce eccentricity, causing one wall to exceed the allowable drift limit and fail earlier than others. Square-like buildings usually perform better than long, thin rectangular buildings.

Research effort also has been made to quantify the effects of diaphragm rigidity on seismic performance of irregular LTF buildings. For example, Kirkham et al. [15] studied the modelling techniques of roof diaphragm (as typical of USA practice) stiffness (rigid, semi-rigid or flexible horizontal diaphragm analyses) in relation to the plan configurations. The analysis results were compared with historical earthquake damage reports. The study concluded that semi-rigid modelling or an envelope method is prudent.

Chen et al. [16,17] developed two multiple spring models to estimate the load distribution in shear walls considering diaphragm rigidity, based on the deformation equations for shear walls and diaphragms in CSA O86. For the multiple-spring model, the translational springs are used to model the diaphragm stiffness and the stiffness of the different lateral load-resisting systems. The model was validated with test results and finite element analysis results of a specific benchmark building. The lateral load distribution between different lateral load-resisting systems with varying stiffness ratios of diaphragm to lateral load-resisting systems was also investigated. The results show that, contrary to common belief, the forces transferred by a semi-rigid diaphragm to supporting lateral load-resisting systems may be higher than those predicted by flexible or rigid diaphragm assumptions. Therefore, using the envelope force approach could lead to underestimation of the design forces in the shear walls.

LTF construction practice in NZ is significantly different from overseas practice. To advance our understanding of the seismic effects of irregular LTF houses in NZ, it is necessary to study the seismic performance of LTF buildings typical of NZ construction.

Methodology for Studying the Seismic Effects of Irregular LTF Houses

When a building structure has plan irregularities, it will not only translate but also rotate in earthquakes, and these two different responses are coupled with each other. To adequately capture the coupled seismic responses of an irregular house, it is essential to conduct 3D analyses and adequately model the in-plane behaviour of seismic bracing timber walls and the floor/roof diaphragms constructed according to NZ practice.

As such, several single-storey case study LTF buildings with varying degrees of structural plan irregularities, which are permitted by NZS3604:2011, are designed and studied to quantify the seismic effects of plan irregularities of LTF buildings within NZS3604:2011. All these buildings contain plasterboard walls and plasterboard ceiling diaphragms as typical of NZ house construction. In studying these case study buildings, the in-plane racking behaviour of plasterboard bracing walls and ceiling diaphragms are modelled, using the models developed based on NZ practice as described by Liu et al. [18,19]. The models simplify the plasterboard bracing walls and diaphragms into 2D plate elements with calibrated shear moduli at different deformation stages to match the test load-deformation curves. Non-linear static push-over analyses are conducted on the building models in order to gain insights of the seismic effects of structural plan irregularity on seismic behaviour of LTF residential buildings in NZ.

DESIGNS OF CASE STUDY BUILDINGS

General

As described previously, seismic bracing designs of low-rise LTF buildings according to NZS3604:2011 are to ensure that the provided total bracing capacity is at least equal to the total bracing demand for a building as well as bracing system arrangements meets the specified irregularity limits. The current irregularity limits permitted by NZS 3604:2011 are:

1. Bracing lines in any storey shall be spaced at no more than 6 m in each direction.
2. Minimum bracing resistance for each bracing line shall be the greater of 100 BUs (note: BU represents bracing units, 1kN=20 BUs) or 50% of the total bracing demand divided by the number of bracing lines in the direction being considered.
3. Apart from the above two requirements, minimum bracing resistance for each external wall in any storey shall be no less than 15 BUs/m of external wall length.

In this study, three single-storey case study buildings were designed to have different permissible plan irregularities by NZS3604:2011, in order to quantify the seismic effects of structural irregularity associated with irregular arrangements of bracing systems as permitted in NZS3604:2011.

Design of Three Case Study Buildings

All three case study buildings were assumed to be constructed on a site that has a seismic hazard factor of Z = 0.46 and is classified as subsoil class D according to NZS 1170.5:2004. Furthermore, it was assumed that roofs are corrugated metal roofs with roof pitches less than 25°, the claddings are heavy claddings, the storey height is 2.4 meters and the ground floors are concrete rib-raft slabs. Accordingly, the seismic demand for the three single-storey building is 9 BUs/m2, according to NZS 3604:2011. This was derived based on a seismic coefficient of 0.4 where the lumped seismic mass at roof level equals 1.125 kPa, expressed in terms of floor area.

The three case study buildings have identical floor plans as shown in Figure 1, where the floor plan dimensions are 12 m x 15 m. The three buildings are actually identical except the arrangements of bracing systems in Y direction only. The total seismic demand for each building is 1,620 BUs for either X or Y direction. The irregularity limits for the case study buildings are established, according to NZS3604:2011 as follows:

The number of bracing lines in y direction is four and the building length in y direction is 12 m. Therefore, the irregularity limit specifications in NZS 3604:2011 are translated as follows:

- Minimum bracing provision for each internal bracing line is 202 BUs, which is the larger of IB1 and IB2 where IB1 = 100 BUs and IB2 = 203 BUs, which is calculated as 1,620 BUs x 0.5 / 4 as specified in clause 2.
- Minimum bracing provision for each external wall line is 202 BUs, which is the larger of EB1 and EB2, where EB1 = 203 BUs, calculated as 1,620 BUs x 0.5 / 4 as specified in clause 2 and EB2 =15 BUs/m x 12 m = 180 BUs.

The bracing arrangements of three buildings, named as RR, IR1 and IR2, are regular, extremely irregular, and moderately irregular. The bracing arrangements of the three buildings are explained below:

Building RR is perfectly regular in both directions. Building IR1 is identical to building RR except that the bracing arrangement in y direction is close to the irregularity limit allowed by NZS3604:2011, as described in section 3.1. Meanwhile building IR2 is identical to building IR1 except that the bracing arrangement in y direction is less irregular.

With regards to the seismic bracing provision, it is assumed that seismic bracing capacity of the external walls is 60 BUs/m, which is the middle range of seismic rating for singly lined plasterboard walls. Similarly, seismic bracing rating of the internal bracing walls was taken as 85 BUs/m, which is a close match to the seismic rating given to doubly lined proprietary plasterboard walls without hold-down devices. Figure 2 shows the bracing distributions of three case study buildings. Table 1 details the seismic bracing provision of three rectangular case study buildings.

As detailed in Table 1, the seismic bracing provisions of all three case study buildings were about equal to the seismic demands derived according to NZS3604:2011 and they all meet the requirements in the standard.

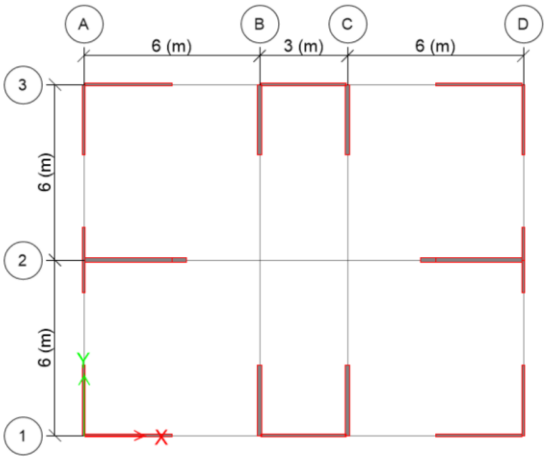
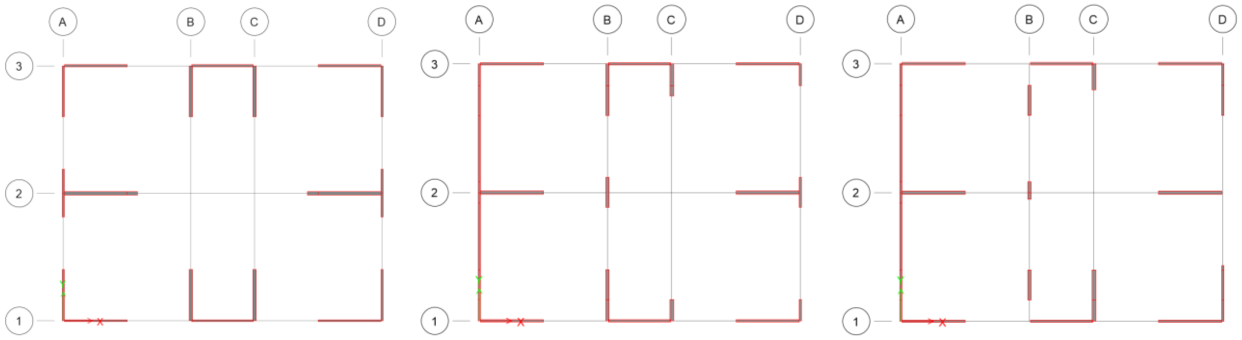


Figure 1: Typical floor plan of three case study buildings.

SEISMIC ANALYSES OF CASE STUDY BUILDINGS

General

Earthquake performance of irregular building structures is significantly more complicated than that of regular counterparts. In the case of a perfectly regular building, a 2D seismic analysis could adequately capture the seismic performance characteristics of the building. In contrast, an irregular building is expected to have coupled responses of torsional and translational behaviour in earthquakes. In the latter case, seismic actions will be transmitted by the floor diaphragms into the lateral load-resisting systems in both directions across the entire building. The seismic performance of an irregular building, when subjected to a single directional earthquake, depends not only on the racking behaviour of primary lateral load-resisting systems in both directions but also the in-plane rigidities of floor/roof diaphragms. As such, three-dimensional global analyses with adequate modelling of the lateral seismic-resisting systems and the floor diaphragms are essential to capture the seismic performance of buildings with structural irregularities.



(a) Building RR *(b) Building IR1* *(c) Building IR2*

Figure 2: Bracing wall arrangements of three case study buildings.

Table 1: Summary of seismic bracing designs of rectangular case study buildings.

			x direction			y direction		
Bracing line #		**1**	**2**	**3**	**A**	**B**	**C**	**D**
Minimum bracing requirement in NZS 3604:2011 (BUs)		270	270	270	202	202	202	202
R	Bracing provision at each bracing line (BUs)	528	587	528	423	408	408	423
	Total bracing provision (BUs)		1642			1662		
IR1	Bracing provision at each bracing line (BUs)	531	587	531	720	527	212	204
	Total bracing provision (BUs)		1650			1665		
IR2	Bracing provision at each bracing line (BUs)	531	587	531	720	305	305	300
	Total bracing provision (BUs)		1650			1630		

Modelling Techniques of LTF Walls and Ceiling Diaphragms

For LTF residential buildings in NZ, the primary lateral seismic-resisting systems are typically plasterboard-lined LTF walls, and roof/floor diaphragms are also LTF construction. Figure 3 shows a typical configuration of 2.4m long single-lined plasterboard bracing wall [20]. Typical load-drift hysteretic curves of these bracing walls are shown in Figure 4. Figure 5 also shows an example of a typical ceiling diaphragm configuration used in the LTF construction in NZ. According to the experimental test results [19], the floor/roof diaphragms are neither rigid nor flexible. It is important to adequately model racking behaviour (stiffness, strength and energy-dissipating capacity) of LTF walls and model in-plane rigidity of floor/roof diaphragms.

In evaluating the seismic performance of the case study buildings, a 3D numerical model for each case study building was created using ETABS software [21]. For the 3D ETABS models, in-plane stiffness of LTF plasterboard walls and plasterboard ceiling diaphragms were modelled using the equivalent models developed by Liu [18, 19], where the racking model of the plasterboard bracing walls and in-plane rigidity model of plasterboard ceiling diaphragms were developed, based on backbone curves of the cyclic tests of these structural assemblies.

According to Liu et al [18], plasterboard walls are modelled as equivalent solid shell elements, the thickness of the shell elements was taken as 100 mm for singly lined LTF walls as typical of perimeter walls and 165 mm for doubly lined LTF walls as for internal walls. Similarly, plasterboard ceiling diaphragms are modelled as solid shell elements and the thickness of the shell elements was taken as 100 mm. For both plasterboard walls and ceiling diaphragms, the engineering properties representing the shell elements are modulus of elasticity (E) and shear modulus (G). E is a constant value and it is 2,500 MPa for both wall shell elements and ceiling diaphragm shell elements. G is a degrading parameter representing non-linear degradation of these elements, and it varies in relation to the imposed deformations. Values of G for shell elements representing plasterboard walls, as suggested by Liu et al. [18], are calibrated by the existing test data and reproduced in Table 2.

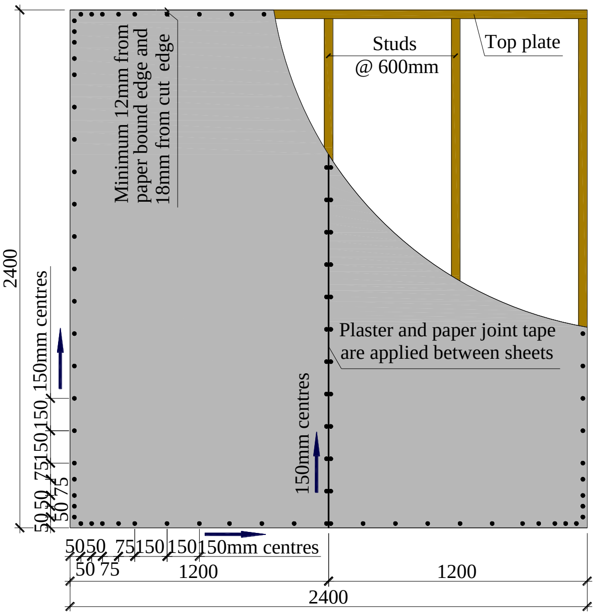


Figure 3: A 2.4 m bracing wall without hold-downs [20].

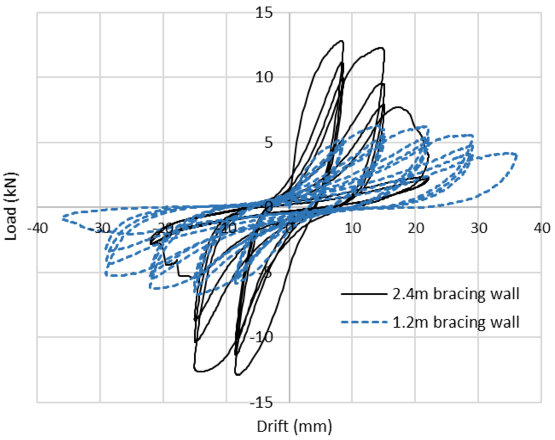


Figure 4: Load-drift hysteresis of 1.2m and 2.4m bracing walls without hold-downs.

For plasterboard ceiling diaphragms, values of G vary significantly, as a result of different construction details in the different application situations. As reported by Liu [19], the value of G corresponding to the stiffest plasterboard ceiling diaphragms could be 17 times that of the most flexible plasterboard ceiling diaphragms, where the stiffest case represents the diaphragms where the fasteners to attach plasterboards to timber frames are closely spaced with tapes and stops between ceiling and walls. In contrast, the most flexible plasterboard ceiling diaphragms are the diaphragms where the fasteners from plasterboard to frames are sparsely spaced with neither tapes nor stops between ceiling and walls. The values of G used for plasterboard ceiling diaphragms in this study is shown in Table 3 and these values represent the common practice where plasterboards are attached to the roof framing using 6 g screws spaced at 150 mm around the perimeters and 300 mm along the battens.

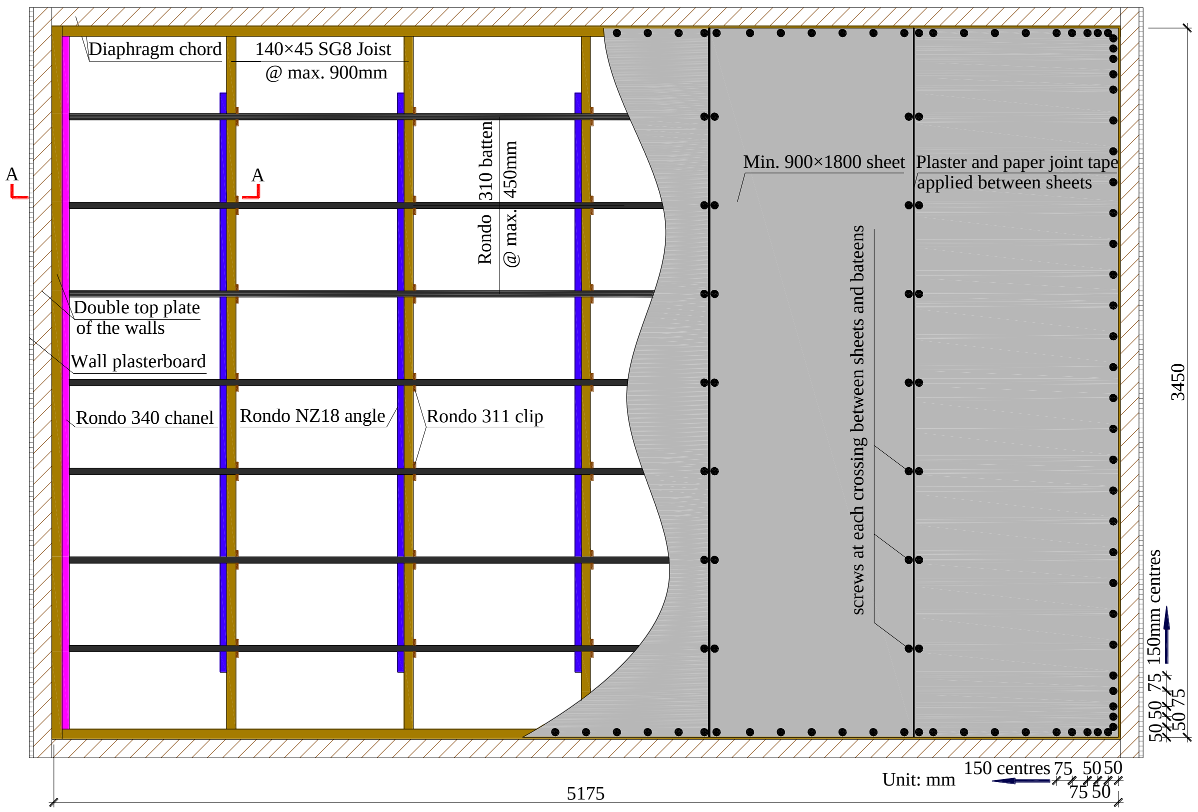


Figure 5: An example of a ceiling diaphragm with plasterboard linings [20].

Table 2: Shear modulus, G, of virtual solid LTF walls.

Δ (mm)	8	15	22	29	36	43
G (MPa)	9.00	5.00	3.50	2.75	2.00	1.50

Note: Δ = Magnitude of lateral storey deflection

Table 3: Shear modulus, G, of common plasterboard ceiling diaphragms.

Δ (mm)	0.5	1.0	1.6	2.75	4.5	5.0	6.0
G (MPa)	40	35	28	21	16	14	10

Note: Δ = Magnitude of lateral storey deflection

Non-linear Static Pushover Analyses

General

In conducting non-linear static push-over analyses for three case study buildings, only seismic responses in y direction were studied because these three buildings are identical except the bracing arrangements in y direction.

The seismic demand for each building in y direction is 1620 BUs, or 81 kN, and this is the seismic action at ultimate limit state (ULS), calculated according to NZS 3604:2011. The roof area is 180 m^2, same as the floor area. As such, the seismic action applied for each case in y direction is equal to a uniformly distributed in-plane action at roof level of 0.45 kPa.

It needs to be appreciated that the applied load, 0.45 kPa, remains unchanged throughout the analyses. In fact, seismic actions expected in the modelled LTF structures could be significantly different from the applied seismic actions derived from NZS 3604:2011. This is because NZS3604:2011 in some ways could underestimate the seismic action as revealed in previous studies [22] and meanwhile the seismic action would reduce gradually as the push-over analyses progress due to the structure softening. However, this study is limited to quantifying the effect of permissible irregularity on seismic performance of NZS 3604:2011 constructions and the determination of the appropriate seismic actions for NZS 3604:2011 construction is beyond the scope of this study.

Calibrations of Shear Modulus of Walls and Diaphragms

For the analyses, elastic modulus E of the shell elements, which is associated with the flexural and axial properties, is a constant value. However shear modulus G for each shell element representing either an LTF wall or a plasterboard ceiling diaphragm was adjusted iteratively to ensure that the assigned stiffness parameters were compatible with the deformation levels experienced.

Table 4 summarises the calibrated engineering parameters of the shell elements representing LTF walls in the case study buildings, when the buildings were subjected to lateral seismic actions at ULS. Table 5 summarises the calibrated engineering parameters of the shell elements representing ceiling diaphragms in the case study buildings, when the buildings were subjected to lateral seismic actions at ULS.

Analysis Results

House damage in earthquakes is frequently observed to be closely related to the inter-storey drift levels. As such, the discussion here focuses on the induced lateral deflections obtained from the non-linear static push-over analyses.

Figures 6 to 8 graphically show the obtained lateral deflections at the roof eave level for three case study buildings when subjected to y directional seismic actions derived from NZS 3604:2011. Table 6 details the obtained Y directional deflection results of the case study buildings at the roof level.

Discussions

For typical LTF buildings in NZ, plasterboard LTF walls are the lateral seismic-resisting systems and also the gravity load-carrying systems. The in-plane lateral deflections experienced by the walls in design earthquakes would be much smaller than the lengths of the walls. In addition, low-rise LTF residential buildings are light in nature, and subsequently, P-Δ effects usually are not significant enough to cause instability problems after the buildings have undergone significant earthquake-induced lateral deflections. Therefore, LTF buildings of mainly NZS 3604:2011 construction could easily achieve life safety requirements in design earthquakes, as reported in past studies [22,23]. As such, the life safety requirement at ULS as per NZS 3604:2011 is not meaningful for LTF buildings. Instead, damage control should be a more appropriate ULS seismic performance criterion for LTF buildings.

Studies of P21 tests at BRANZ [18] show that an LTF plasterboard wall of reasonable length is expected to experience significant strength degradation and softening when the wall drift exceeds 22 mm over a 2.4 m storey height, i.e. approximately a drift ratio of 1%. Thus the drift ratio of 1% is established as the damage control criterion in evaluating the seismic performance of our case study buildings in an ULS earthquake event.

Seismic Effects of Permissible Plan Irregularities

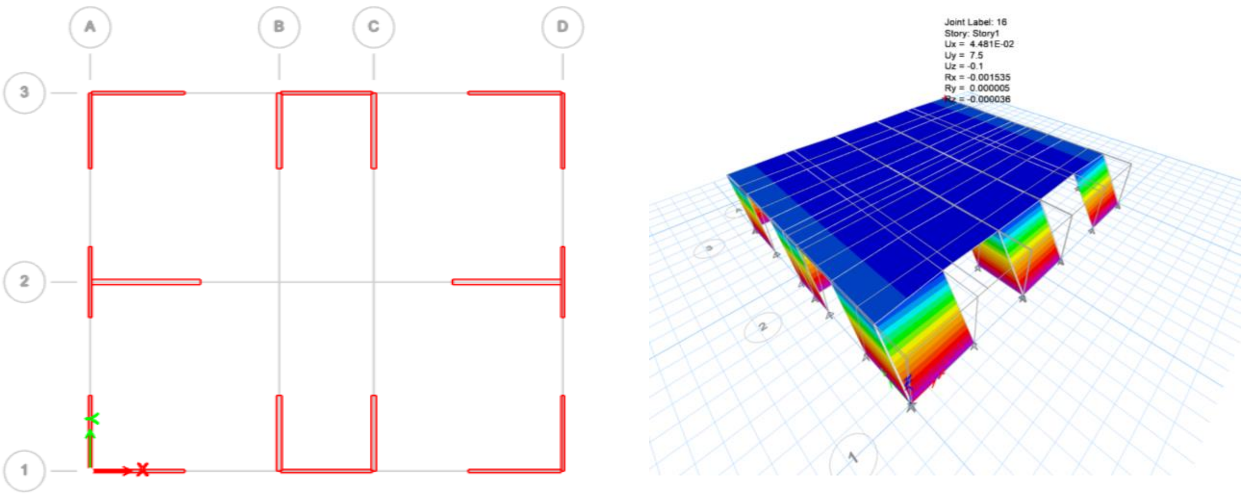
As described previously, the three case study buildings have varying degrees of plan irregularities, which are permissible by NZS3604:2011. Comparison of the induced lateral deflections of building RR with building IR1, as the values in table 6, shows that extreme irregular bracing arrangements as allowed by NZS 3604:2011 could result in significant amplification of lateral deflections, resulting in significant earthquake damage to the buildings in design earthquake events. In detail, the largest induced lateral deflection under an ULS seismic action as specified by NZS 3604:2011 was respectively 7.5 mm for the perfectly regular building RR and 37.8 mm for the extremely irregular LTF building IR1. A lateral deflection of 7.5 mm over a storey height of 2.4 mm of an LTF building is well within the established damage control limit, 1% drift ratio. In contrast, a lateral deflection of 37.8 mm over a storey height of 2.4 m of an LTF building (about 1.6% drift ratio for the case study building IR1) means that the building will experience significant damage and might end up with demolition due to high repair cost. The analysis indicates that the current irregularity limits in NZS 3604:2011 are not robust enough to avoid excessive damage and achieve reasonably consistent seismic performance of LTF buildings.

Table 5: Shear modulus (G) for ceiling diaphragms of rectangular case study buildings (MPa).

Case study building	G (MPa)
RR	36
IR1	21
IR2	28

Table 4: Shear modulus (G) of wall elements of rectangular buildings (MPa).

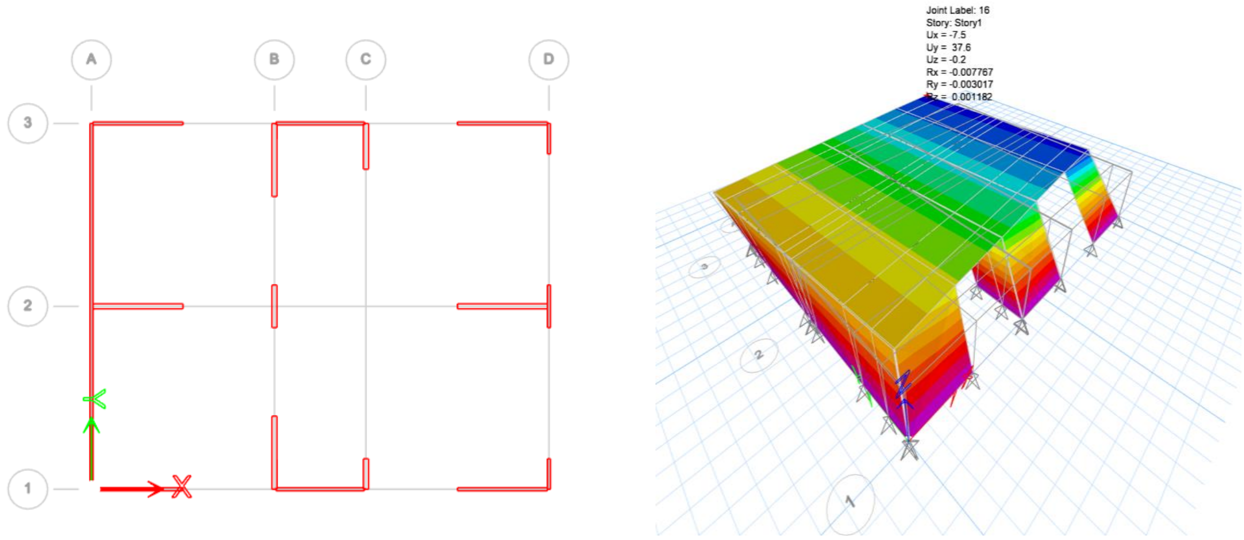
Case study buildings	x direction grid #			y direction grid #			
	1	2	3	A	B	C	D
RR	9.00	9.00	9.00	9.00	9.00	9.00	9.00
IR1	9.00	9.00	9.00	5.00	2.75	2.00	1.50
IR2	9.00	9.00	9.00	5.00	5.00	5.00	3.50



(i) *Undeformed plan view*

(ii) 3D deformed view

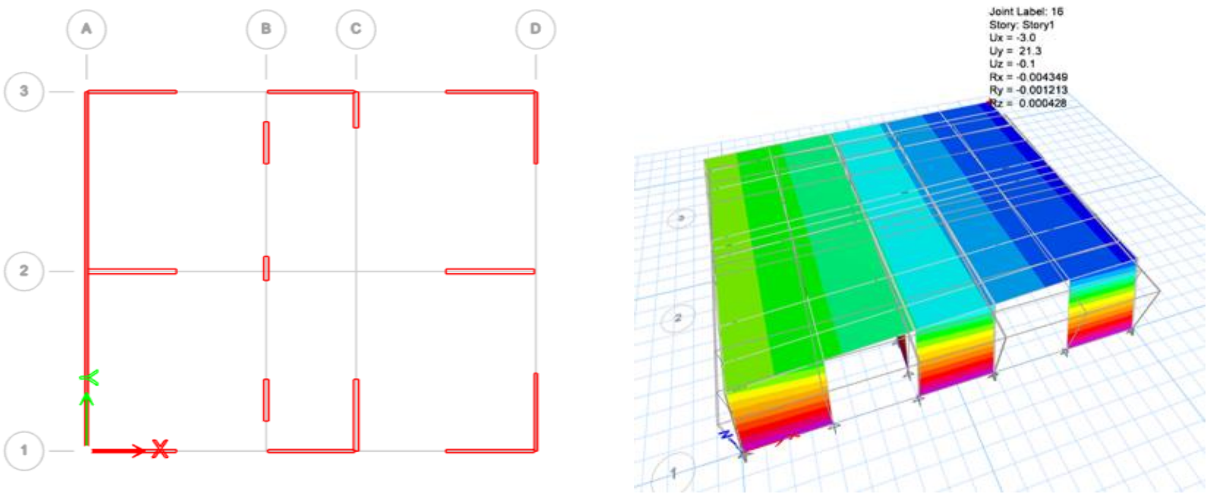
Figure 6: Deflections obtained for case study building RR.



(i) Undeformed plan view

(ii) 3D deformed view

Figure 7: Deflections obtained for case study building IR1.



(i) Undeformed plan view

(ii) 3D deformed view

Figure 8: Deflections obtained for case study building IR2.

Table 6: Y directional deflections at roof level along grid lines.

Case study building	Grid line ID			
	A	B	C	D
RR	7.5 mm	7.5 mm	7.5 mm	7.5 mm
IR1	16.0 mm	25.0 mm	29.7 mm	37.6 mm
IR2	12.5 mm	16.6 mm	18.5 mm	21.3 mm

Effect of Tightened Irregularity Limits

Case study building IR2 was less irregular than case study building IR1, although both buildings have exactly the same total bracing capacity. For IR2, the minimum bracing capacity along the perimeters was calculated using 75% rather than 50% of total seismic bracing demand divided by bracing line numbers.

As shown in Table 6, this adjustment resulted in a significantly reduced lateral deflection, and the maximum lateral deflection for building IR2 was smaller than 22 mm, which is the established damage control limit 1% drift ratio. This implies that the current irregularity limits in NZS 3604:2011 should be tightened and the minimum bracing provisions along the bracing lines, especially the perimeter lines, should be revised in order to reduce the building torsional effect and enhance the seismic resilience of the LTF houses.

CONCLUSIONS AND RECOMMENDATIONS

Residential buildings in NZ are often LTF buildings constructed according to NZS 3604:2011. With regards to the seismic bracing arrangement, NZS3604:2011 specifies irregular arrangement limits but these limits were not based on rigorous scientific evidence. In the Canterbury earthquake sequence, one apparent evidence was that presence of irregular bracing arrangements in LTF buildings exacerbated the earthquake damage significantly. This study was to quantify the seismic effects of allowable irregularity limits in NZS 3604:2011 and ultimately to help inform the new irregularity rules to control earthquake damage to LTF buildings.

In this study, three single-level LTF case study buildings were designed to NZS 3604:2011 and they have varying degrees of allowable bracing irregularities. Three-dimensional static non-linear push-over seismic analyses were conducted, using the ETABS program, for each case study building. The walls and the ceiling diaphragms were modelled using equivalent shell elements and their properties were calibrated by the test results using NZ practice. The main findings are summarised below:

- Current permissible irregular seismic bracing arrangements in NZS 3604:2011 could potentially amplify the lateral deflections significantly in comparison with the regular buildings. LTF buildings with minimum bracing provision along building perimeters could sustain earthquake damage well beyond repair.
- In-plane rigidity of plasterboard ceiling diaphragms are generally more rigid in comparison with cantilever plasterboard bracing walls. The higher the in-plane rigidity, the better the overall performance of an irregular building.
- Tightening the minimum bracing provisions in NZS 3604:2011 by 50% along the perimeter bracing lines could reduce the induced lateral deflection by 43% and keep the deflection within a tolerable damage control limit.

ACKNOWLEDGEMENTS

This research project was funded by the Building Research Levy.

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