Higher mode effects in multi-storey timber buildings with varying diaphragm flexibility

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ABSTRACT: With the increasing acceptance and popularity of multi-storey timber buildings up to 10 storeys and beyond, the influence of higher mode effects and diaphragm stiffness cannot be overlooked in design. Due to the lower stiffness of timber lateral load resisting systems compared with traditional construction materials, the effect of higher modes on the global dynamic behaviour can be more critical. The presence of flexible timber diaphragms creates additional vibration modes, which have the potential to interact with each other, increasing the seismic demand on the whole structure.

This paper uses a parametric non-linear time-history analysis on a series of timber frame and wall structures with varying diaphragm flexibility to study their dynamic behaviour and to determine diaphragm forces. The analyses results showed that although higher mode effects play a significant role in the structural dynamic response, this increased demand can be successfully predicted with methods available in literature.

The parametric analyses showed that the diaphragm flexibility did not significantly increase the shear and moment demand; however, stiff wall structures with flexible diaphragms experienced large inter-storey drifts measured at diaphragm midspan compared with the drift of the wall alone. As expected, the diaphragm forces observed from the time-history analyses were significantly higher than the forces derived from an equivalent static analysis, leading to a potentially unsafe design. The paper presents a simplified approach for evaluating these amplified peak inertial diaphragm forces.

1 INTRODUCTION

Tall timber structures are becoming increasingly popular, with multi-storey timber buildings soon allowed to up to 25 meters in Australia and developers investigating multi-storey residential buildings in light timber framing and massive timber in New Zealand. However, the design of such structures in seismic prone areas requires the understanding of their dynamic behaviour. Because of the lower stiffness of timber lateral load resisting systems, higher mode effects can have a larger impact when compared to traditional construction materials. Timber diaphragms are often more flexible than their concrete counterparts and this has the potential to further alter the dynamic behaviour of the whole building and increase the diaphragm force demand.

Existing literature (Fleischman et al. 2001; Lee et al. 2007; Sadashiva et al. 2012; Humar et al. 2013) on concrete and steel structures proves that the presence of flexible diaphragms can change the dynamic behaviour of the whole system. Such effects were shown to be more pronounced in structures with a low number of storeys, particularly in shear wall structures. In such cases, flexible diaphragms increased the structure's fundamental period, and resulted in higher displacement demands. Numerical research carried out by van Beerschoten et al. (2010) concluded that diaphragm flexibility does not significantly affect the dynamic behaviour of multi-storey timber frame structures; however, it was recommended that in-plane floor flexibilities be accounted for in wall structures.

In order to study the influence of diaphragm flexibility in multi-storey timber structures, a parametric analysis on a number of frame and wall structures was carried out and presented in this paper. Non Linear Time History (NLTH) analyses were carried out comparing the outcomes of the structures with a rigid diaphragm assumption with several diaphragms with varying flexibility.

The impact of higher modes, including the influence of diaphragm flexibility, is discussed in terms of storey shear and moment distribution as well as inter-storey drifts. Current methods for the evaluation of storey shears and moments for concrete structures were validated for the analysed timber structures. The diaphragm force demand from the analyses is compared with predictive models available in codes and literature and a new simplified approach for evaluating peak diaphragm inertial forces is proposed.

2 MODEL DESCRIPTION

A set of pre-stressed laminated (Pres-Lam) timber (STIC 2013) frame structures with 2, 4 and 6 storeys and pre-stressed timber wall structures with 3, 6 and 9 storeys, respectively, with and without additional dissipation devices, have been studied in a parametric analysis. The analyses used the floor layout shown in Figure 1; the loading was applied in the North-South direction. The floor diaphragms were spanning between the five frames for the seismic frame structures; for the wall structures the diaphragms were spanning between the outer gravity frames which were attached to the shearwalls.

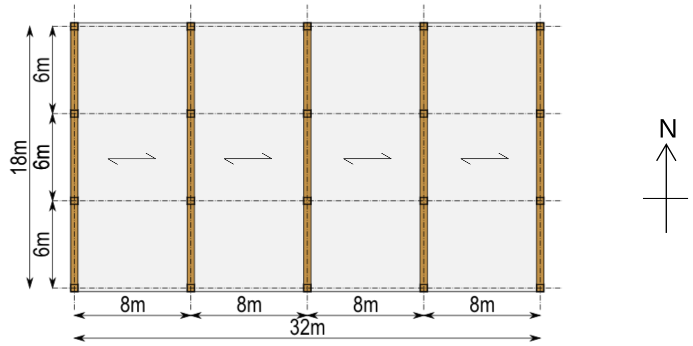


Figure 1: Sample building plan view

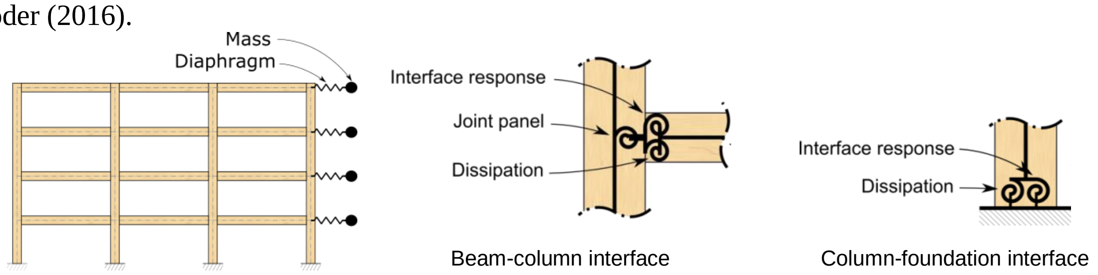
Even though the analyses were carried out on post-tensioned structures, similar trends can be expected for multi-storey buildings built with different timber lateral load resisting systems. This is because multi-storey timber structures are normally Serviceability Limit State (SLS) governed, leading to similar buildings stiffnesses.

The design of the frame and wall structures was carried out in accordance with the Displacement Based Design (DBD) approach (Priestley et al. 2007) and the STIC Design Guidelines for Post-Tensioned Timber Buildings (STIC 2013). A design drift of 1.8% and a ductility of 2 was targeted for the frame structures. For the wall structures the design drift was 1.2% and a ductility of 3 was used. For both the frame and wall structures two different re-centering ratios, β (defined as the ratio of the moment resistance provided by the post-tensioning to the total moment resistance, see STIC (2013)), of 1.0 (i.e. 100% re-centering contribution from the post-tensioning, 0% dissipative contribution) and 0.7 (i.e. 70% re-centering contribution, 30% dissipative contribution) were used; these are referred to as the damped and undamped structures, respectively.

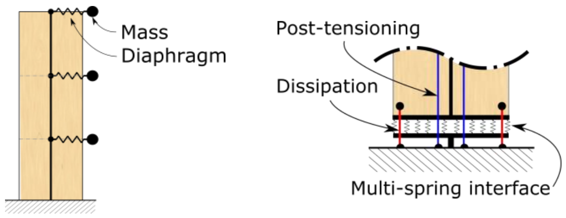
The design earthquake demand was assessed in accordance to NZS 1170.5 (Standards New Zealand 2004), assuming an importance level IL2, a 500 years return period, a hazard factor of Z = 0.4, a soil type C, a near-fault factor D = 1 and a structural performance factor S_p = 0.7 (applied to the base shear and not to the displacement spectrum).

The modelling program OpenSEES (McKenna et al. 2000) was used for the Non-Linear Time-History (NLTH) analyses applying a set of 10 earthquake records from the Pacific Earthquake Engineering Research Center (PEER) Next-Generation Attenuation (NGA) database (Chiou et al. 2008). The records were scaled to fit the design spectrum on a defined period interval around the structure's fundamental period in accordance to NZS 1170.5

The diaphragm flexibility was simulated through linear springs connecting the storey lumped seismic mass to the lateral load resisting system as shown in Figure 2. The rocking mechanism at the beam-column or wall-foundation interfaces was modelled with rotational spring elements with multi-linear elastic relationships for the interface response and with elasto-plastic relationships for the dissipation devices if present. For the beam-column joint an additional joint panel spring was added to account for the shear deformation of the timber column (Smith et al. 2014). For the wall-foundation interface a specially defined multi-spring interface has been used (Sarti 2015). More information regarding the structural dimensions, section geometries, material properties and input model can be found in Moroder (2016).



a) Frame model



b) Wall model

Figure 2: Models of a frame and wall structure for the analyses
(frame structure with four storeys and wall structure with three storeys)

2.1 Diaphragms stiffness

To study the influence of the diaphragm flexibility on the dynamic behaviour of the structures, the models were analysed with the rigid diaphragm assumption and with a series of flexible diaphragms. The assigned spring stiffnesses and respective diaphragm periods used to simulate the diaphragm flexibility are summarized in Table 1. In accordance to NZS 1170.5, the diaphragm response was considered to be linear elastic.

Table 1. Diaphragm stiffnesses and periods (in parenthesis)
(values in bold are considered to represent real diaphragms)

Structure	Diaphragm stiffness in kN/m (and diaphragm period in seconds)							
frame	∞	160,000 (0.11)	80,000 (0.15)	**40,000** **(0.22)**	20,000 (0.30)	10,000 (0.43)	5000 (0.61)	2500 (0.86)
wall	∞	96,000 (0.28)	80,000 (0.31)	64,000 (0.35)	**48,000** **(0.40)**	32,000 (0.49)	16,000 (0.7)	8000 (0.98)
	rigid	very stiff						very flexible

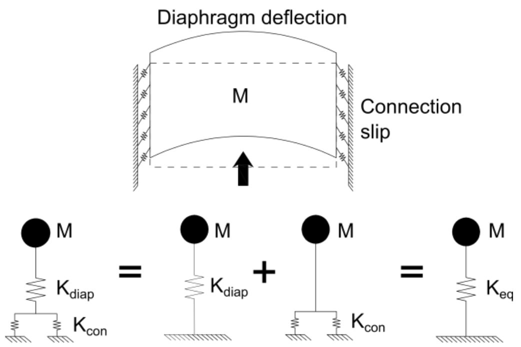


Figure 3: Schematic contributions of diaphragm and connection stiffnesses to the overall stiffness
(modified from Brignola et al. (2008))

The diaphragm stiffnesses of 40,000 kN/m and 48,000 kN/m for frame and wall structures respectively were assumed to be representative of realistic floor layouts built of 100 mm thick Cross Laminated

Timber (CLT) panels with dimensions of 1.2 m x 8 m connected with metallic fasteners with a slip modulus of 3000 kN/m. For the wall structures the diaphragm panels were fixed to collector beams which were fixed to the walls. Since the horizontal forces are transferred from the collector beam to the walls, an additional source of flexibility was added to the total diaphragm stiffness. The stiffness of the diaphragms (K_{diap}), combined with the stiffness of the connection to the lateral load resisting system (K_{con}), was idealized as a single degree of freedom oscillator with an equivalent stiffness (K_{eq}), as shown in Figure 3.

3 RESULTS AND DISCUSSION

For the sake of brevity, the key parameters of fundamental period, storey shear forces and moments, as well as the inter-storey drifts are shown and discussed mainly for the damped 4 storey frame and 6 storey wall models. These are considered to be the most relevant building examples in terms of height and additional dissipation. The trends of these key parameters are representative of all other structures analysed for this paper. If not elsewhere stated, all values are taken as the average of the maximum values from each of the ten earthquake records.

3.1 Fundamental period

In general, the analyses results showed that increased diaphragm flexibility elongated the fundamental period of stiff structures, whereas the increase of period is negligible for flexible structures.

As shown in Table 2 the difference in the fundamental period with rigid and real diaphragms were 1% and 5% for the damped 4 storey frame and 6 storey wall structures, respectively. For the most flexible diaphragms, the period increased by 22% and 27%, respectively. It is worth noting that the structures' higher mode periods were tending to the period of the diaphragms.

For both the damped and undamped 3 storey wall structures, which were relatively stiff, the model with the real diaphragm stiffness increased the fundamental period by 39%.

Table 2. Period of vibration of the damped 4 storey frame and damped 6 storey wall structures with varying diaphragm stiffnesses (values in bold are considered to represent real diaphragms)

Mode	Period in seconds					
	4 storey frame			6 storey wall		
	rigid	real	very flexible	rigid	real	very flexible
1	1.22	**1.24**	1.50	1.27	**1.33**	1.60
2	0.36	**0.42**	0.93	0.24	**0.47**	1.01
3	0.17	**0.27**	0.88	0.11	**0.42**	0.99
4	0.11	**0.24**	0.87	0.08	**0.41**	0.99
5				0.06	**0.41**	0.99
6				0.06	**0.41**	0.98

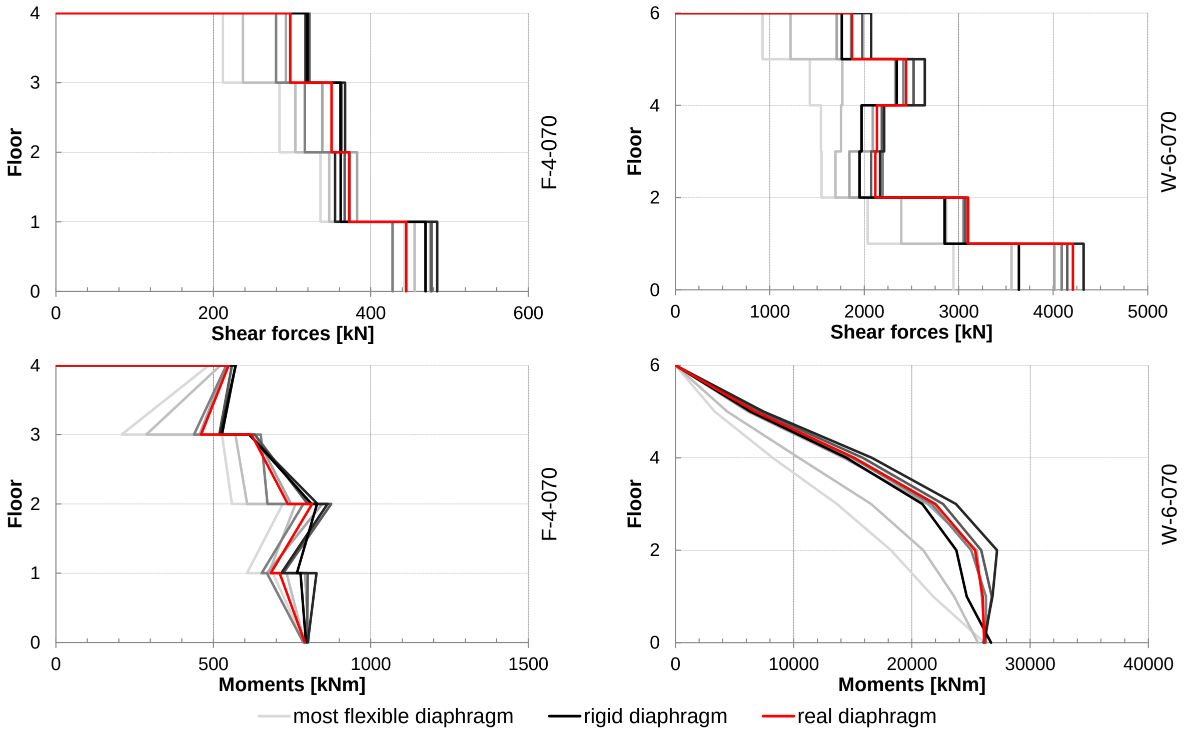
3.2 Storey shears and moments

For both frame and wall structures higher mode effects influenced the storey shears and moments. This effect was more pronounced for tall frame and wall structures and almost negligible for the short structures. Wall structures tended to be affected more by higher modes as shown in Figure 4.

Diaphragm flexibility had a relatively small effect on the shear and moment distribution for both the frame and wall structures. Only the results for the very flexible diaphragms differed notably from the rigid diaphragm structures.

Although the observed period elongation due to diaphragm flexibility normally led to a decrease in seismic demand, the magnitude of maximum storey shears and bending moments was almost unaltered between the flexible and rigid diaphragms as shown in Figure 4.

In real applications, the effect of higher modes of the lateral load resisting system needs to be considered for timber structures taller than two to three storeys. Diaphragm flexibility on the other hand can normally be neglected for the determination of the shear and moment demand.



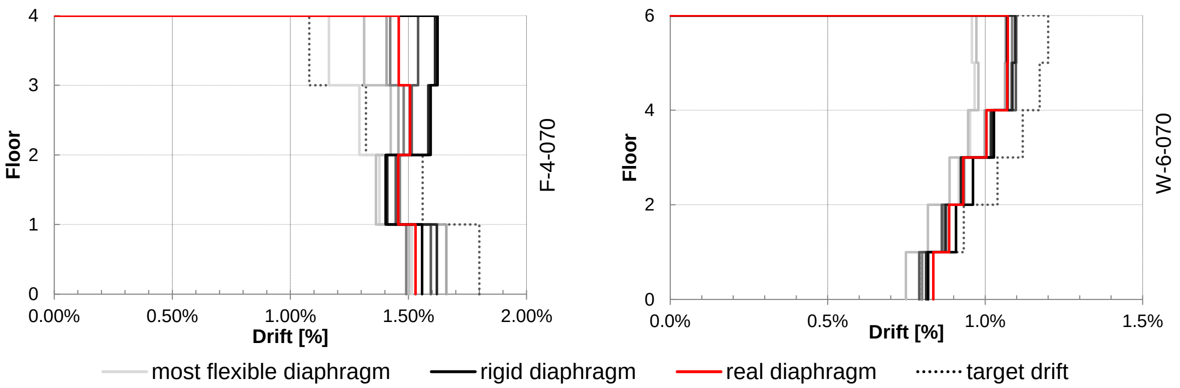
a) 4 storey damped frame structure

b) 6 storey damped wall structure

Figure 4: Storey shear and moment distribution for the damped 4 storey frame and damped 6 storey wall structures with varying diaphragm stiffnesses

3.3 Inter-storey drifts

The frame structures' inter-storey drifts were higher than predicted due to the influence of higher modes in the upper storeys as shown in Figure 5a. In the lower storeys a pull-back effect was decreasing the drift values. It was observed that diaphragm flexibility had a beneficial effect on the structures' inter-storey drift. The inter-storey drift measured at the mid-span of the diaphragms (diaphragm inter-storey drift), tended to be slightly larger than the inter-storey drift of the lateral load resisting system, adding about 0.2% additional drift to structural and non-structural elements connected to the diaphragms when compared to the corresponding frame drift.



a) 4 storey damped frame structure

b) 6 storey damped wall structure

Figure 5: Inter-storey drift of the damped 4 storey frame and damped 6 storey wall structures with varying diaphragm stiffnesses

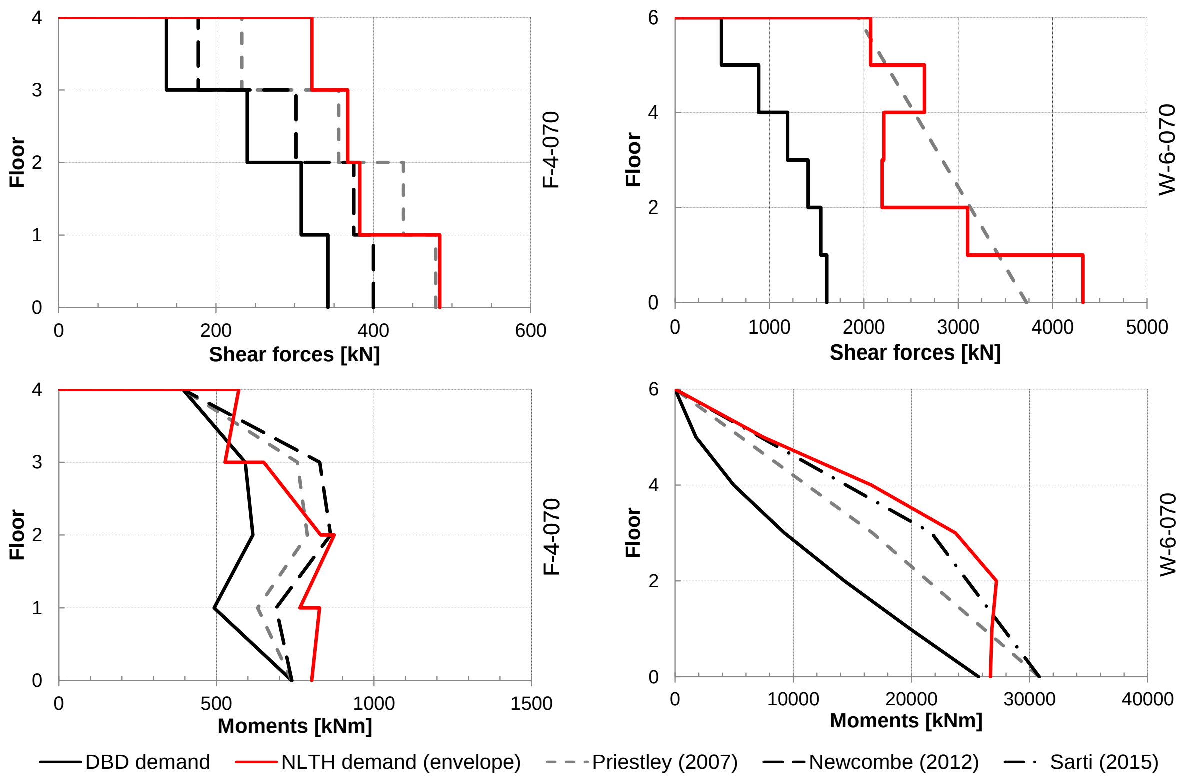
For wall structures the inter-storey drift measured at the walls was always smaller than the target drift values as representatively shown in Figure 5b for the damped 6 storey wall structure. Diaphragm stiffness had a negligible effect on the wall inter-storey drift values. However, it is worth noting that the diaphragm inter-storey drift was notably increased in the case of flexible diaphragms. For the real diaphragms in the damped 6 storey wall structure an additional 0.8% drift was observed at the mid-span of the diaphragms. This effect was largest at the lower and upper storeys and led to total drifts exceeding the target drift values. In the case of very flexible diaphragms these diaphragm inter-storey drifts reached values of 3-4%. It is therefore recommended to consider the diaphragm flexibility when checking for the allowable SLS and ULS drifts in wall structures to prevent excessive damage to non-structural elements and secondary structures.

4 PREDICTION OF FORCE DEMAND

4.1 Prediction of the storey shear and moment demand

Figure 6 shows the shear and moment envelopes from all diaphragm stiffness values for the damped 4 storey frame and the damped 6 storey wall structures, respectively. For all structures, the shear and moment values obtained from the NLTH analyses were notably higher than the values obtained through an Equivalent Static Analysis (ESA) based on a DBD design. This is due to higher modes which amplify the shear forces in the lower and upper storeys and the moment in the middle storeys.

Priestley et al. (2007) provided simplified methods to estimate the shear and moment envelopes for concrete wall and frame structures based on building over-strength and dynamic amplification due to higher mode effects. Newcombe (2011) and Sarti (2015) revised these procedures for post-tensioned timber frame and wall structures, respectively. Figure 6 shows that the proposed envelopes provide relatively accurate envelopes for the inter-storey shears and moments. It is strongly recommended that one of the proposed procedures to account for the increased demand in multi-storey timber structures be adopted.



a) 4 storey damped frame structure b) 6 storey damped wall structure

Figure 6: Maximum storey shears and moments from the equivalent static analysis based on a DBD design, envelopes from NLTH analyses including all diaphragm stiffnesses and suggested prediction envelopes for the damped 4 storey frame and damped 6 storey wall structures

Since the analyses were carried out using the same material specifications and moment-rotation behaviour specific to Pres-Lam frames and walls as adopted in the design procedure, an over-strength factor of 1.2 was selected based on the rounding of section sizes and reinforcing dimensions, as well as the material reduction factor used in the design. This value is lower when compared to designs where strength values of connections are provided by design codes, which normally introduce larger sources of over-strengths.

4.2 **Diaphragm force demand**

The diaphragms of multi-storey timber buildings are the structural element subjected to the highest demand within the whole structure, because they are subjected to the maximum forces occurring at any instant during an earthquake. This results in much higher demands than in the lateral load resisting system where inertial forces can act in different directions up the building's height.

Figure 7 shows the peak floor inertial forces at each floor level for the damped 4 storey frame and the damped 6 storey wall structures. Values shown are enveloped and do not consider the fact that they have occurred at different times during the ground motion. These results are used to determine the largest inertia force in any of the diaphragms in the structure when subjected to a design earthquake.

It can be observed that the diaphragm force demand diverged significantly from the assumed first mode distribution with magnitudes between 2 and 5 times the values predicted by an equivalent static analysis. Diaphragm flexibility did not significantly influence the force demand in frame structures; in wall structures diaphragm flexibility increased the force demand in some cases. The nearly constant force pattern up the structure's height can be attributed to higher mode effects, which mostly impact on the lower half of the structures.

Figure 7 also shows the maximum diaphragm demand based on the amplified top storey shear values, determined with current methods in accordance to the New Zealand Concrete Standard NZS 3101 (Standards New Zealand 2006) and the approach proposed by Priestley et al. (2007) and its modification by Newcombe (2011) for frame structures. It was assumed that the maximum diaphragm demand can be determined as the top storey floor force, including dynamic amplification and over-strength, applied up the whole building height. The diaphragm force demand envelopes from the modified parts and components method as suggested by Cowie et al. (2014), the pseudo Equivalent Static Analysis (pESA) (Bull 2004; Gardiner et al. 2008) and the First Mode Reduced Method by Rodriguez et al. (2002) for wall structures are also shown. None of these methods were able to predict the peak diaphragm demand for the analysed frame or wall structures.

4.3 **Prediction of the diaphragm force demand**

In order to predict the peak diaphragm inertial force demand in timber structures, two formulations, given in equations (1) and (2) for frame and wall structures respectively, are proposed. Both methods are based on the approach proposed by Priestley et al. (2007). All diaphragms up the height of the structure should be designed based on the maximum diaphragm demand determined by one of the following equations.

For frame structures the diaphragm demand can be determined as

$$E^*_{diap\,(frame)} = \phi^o V_{E,top} + 0.2\mu V_{E,base} \tag{1}$$

where E^*_{diap} = diaphragm force demand; ϕ^o = over-strength factor of the lateral load resisting system; μ = ductility of the structure; $V_{E,top}$ = shear value at the top storey from the equivalent static analysis; $V_{E,base}$ = base shear value.

Equation (1) has been modified from the original formulation for the top storey shear force as defined in Priestley et al. (2007) by increasing the multiplier of the base shear value from 0.1 μ to 0.2 μ.

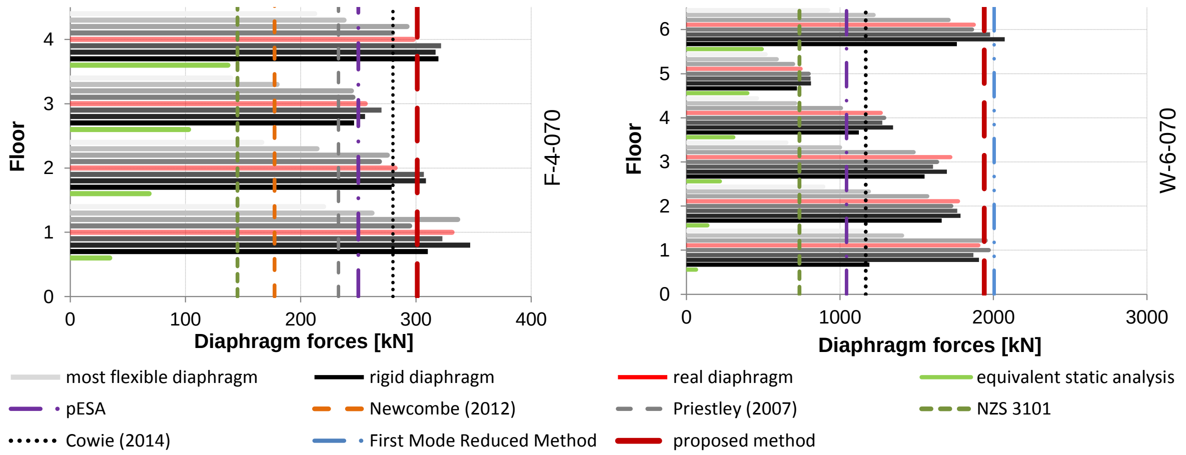
For wall structures the diaphragm demand can be determined as

$$E^*_{diap\,(wall)} = V^o_n \tag{2}$$

where E^*_{diap} = diaphragm force demand; V^o_n is the amplified design shear force at the top of the building at over-strength as defined in Priestley et al. (2007) for concrete wall structures.

Until over-strength factors for timber structures are available, conservative assumptions based on engineering judgement or preliminary experimental testing should be used. For both the non-damped frame and wall structures the ductility μ needs to be taken as 1.

The diaphragm forces suggested above do not occur simultaneously and should not be applied to the structure to determine compatibility forces in the diaphragms, as it would provide very conservative designs. Diaphragm compatibility forces are closely linked to the inertial forces, as they are created by displacement incompatibilities between different lateral load resisting elements acting in parallel, but they do not necessarily occur at the same instant in time. If compatibility forces are expected in the diaphragms, specific analysis should be carried out in order to understand the magnitude of these forces.



a) 4 storey damped frame structure   b) 6 storey damped wall structure

Figure 7: Diaphragm forces from the NLTH analyses for the damped 4 storey frame and damped 6 storey wall structures with varying diaphragm stiffnesses and suggested design values

Since only a limited number of structures have been analysed in this research, further analyses are required to assure that the suggested formulations are appropriate to estimate peak diaphragm forces. It is worth noting that the measured peak forces tend to occur for very limited time intervals only. It needs to be determined whether these peak values are caused by numerical errors during the time history analyses and if such large but short-duration forces have the actual potential to damage the diaphragms. Large diaphragm force amplification might lead to uneconomical designs; it would therefore be of interest to investigate the possible beneficial effect from ductility in diaphragm connections. Ideally, peaks in the diaphragm force demand would lead to localized yielding, without compromising the diaphragm behaviour or the dynamic response of the structure as a whole.

5 CONCLUSIONS

The following conclusions can be drawn from the analyses carried out on multi-storey frame and wall structures with rigid and flexible diaphragms:

- Higher mode effects need to be taken into account when determining the shear and moment demand in structures with flexible lateral load resisting systems regardless of diaphragm flexibility;
- Maximum shears and moments for both frame and wall structures were not significantly influenced by the diaphragm flexibility;
- Current methods were capable of predicting the shear and moment demand in multi-storey timber frame and wall buildings;
- For flexible diaphragms in wall buildings, the diaphragm inter-storey drift values were notably higher than the target wall drifts and need to be checked in design to protect structural and non-structural elements;
- Diaphragm forces for frame structures were not significantly influenced by the diaphragm flexibility; for wall structures flexible diaphragms have the potential to increase the diaphragm

demand at lower storeys;

- For both frame and wall structures all diaphragms up the height of the building should be designed for the same peak force demand which can be determined with equations provided in the paper;
- Peak floor diaphragm forces occur at different moments in time and should not be applied simultaneously to determine the displacement incompatibility forces.

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