Influence of Diaphragm Flexibility on the Seismic Performance of Multi-Story Modular Buildings

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Abstract

Modular buildings are those built using prefabricated volumetric units called modules. Due to modules being connected to each other at discrete locations, discontinuous structural systems are formed, where diaphragm discontinuity is a key issue and could result in diaphragms that are flexible. Multi-story modular buildings with flexible diaphragms are susceptible to higher mode influences when under the action of seismic loads, where such influences affect lateral load distribution, cause excessive drift among gravity frames and could potentially lead towards collapse. This paper presents the preliminary work conducted to classify the behaviour of diaphragms in modular buildings and to assess the effects of diaphragm flexibility on the overall seismic performance of a case study modular building. Diaphragm behaviour was controlled through axial and shear stiffness of diaphragm connections as well as the combined shear stiffness of adjacent module floor and ceiling units. Seismic analyses were conducted using 44 scaled far-field ground motions as considered within FEMA P695 and the results indicate that the case study modular building with flexible diaphragms is affected by higher modes and current code provisions are inadequate for the seismic design of modular buildings.

Keywords: Prefabrication, modular buildings, diaphragm flexibility, seismic design

1. INTRODUCTION

Modular buildings are built using prefabricated volumetric components called modules. Such modules are stacked vertically and scaled horizontally on site to form complete buildings. Modular building construction capitalises on the use of off-site manufacturing, thus resulting in reduced construction times, superior quality and reduced environmental impact (Aye, Ngo, Crawford, Gammampila, & Mendis, 2012; Crowther, 1999; Gibb, 1999; Jaillon & Poon, 2009; Kamali & Hewage, 2016; Kumar, 2016; M. Lawson, Ogden, & Goodier, 2014; R. M. Lawson, Ogden, & Bergin, 2012; Rogan, Lawson, & Bates-Brkljac, 2000; Smith, 2010). However limitations relating to technical, logistical as well as regulatory aspects have hindered its full realisation as the preferred choice for construction (Cartz & Crosby, 2007; Jellen & Memari, 2013; R. M. Lawson et al., 2012; Smith, 2010; Torre, Sause, Slaughter, & Hendricks, 1994). Of the key technical issues, this study focuses on addressing diaphragm discontinuity in multi-story modular buildings (MSMB). The diaphragms of MSMBs are essentially discontinuous due to modules being connected to each other at discrete locations. It is believed that diaphragm discontinuity aggravates diaphragm flexibility and could likely lead to diaphragm failure under the action of seismic loads due to higher mode influences. This paper proposes a method to predict diaphragm service stiffness in modular buildings and demonstrates the effects of diaphragm flexibility on the seismic performance of a hypothetical case study four-story four-by-four-bay modular steel building.

2. DIAPHRAGM CLASSIFICATION IN MSMBs

Diaphragm classification as currently prescribed is based on the ratio between maximum diaphragm displacement (Δ_{dia}) relative to the lateral force resisting system (LFRS) and the corresponding average inter-story drift of the LFRS at service conditions (Δ_{LFRS}), where for rigid diaphragm behaviour this ratio is expected to be ≤ 0.5 and for flexible diaphragm behaviour > 2.0 (for all values in-between, the diaphragm is classified stiff) (American Society of Civil Engineers & Federal Emergency Management Agency, 2000; American Society of Civil Engineers & Structural Engineering Institute, 2010; Standards Australia, 2007; Standards New Zealand, 2004). For diaphragms of MSMBs, total diaphragm deformation ($\equiv \Delta_{dia}$) under service conditions due to the action of lateral loads can be considered as the sum of each individual overall diaphragm deformation due to the influence of diaphragm connection axial and shear behaviour as well as the shear behavior of the combined module floor and ceiling unit. This is demonstrated through a displacement controlled pushover analyses of a 2D diaphragm model developed using the software OpenSees (McKenna, 2011) as shown in **Fig. 1**, where (a) corresponds to the overall diaphragm deformation due to diaphragm connection axial behavior, (b) due to diaphragm connection shear behavior and (c) due to the combined module floor and ceiling unit shear behavior. For this demonstration, the diaphragm model was made simply supported and a uniform lateral load distribution was assumed. In-plane axial and shear behaviour of diaphragm connections was controlled by using two node link elements having both axial (k_A) and shear (k_V) stiffness properties, whereas in-plane shear behaviour of the combined module floor and ceiling unit was controlled via rotational springs (k_R). Through the expression $\Delta_{dia} = \alpha\Delta_{LFRS}$, the limiting diaphragm deformation for the analysis was determined using a desired diaphragm flexibility factor (α) and the corresponding LFRS drift (Δ_{LFRS}) at service conditions. The values of $\alpha = 0.5,\ 2.0\ \&\ 4.0$ was selected and would result in rigid, stiff and flexible diaphragm behaviors respectively. Surface plots were generated, to identify the minimum required stiffness properties to achieve a targeted diaphragm behavior for a particular design lateral load as shown in **Fig. 2(a)**, where for this demonstration rigid diaphragm behavior was targeted along with the use of rigid modules. The intersection of the surface plot with a particular design lateral load as shown in **Fig. 2(a)** is further detailed in **Fig. 2(b)** which indicates the numerous limiting connection stiffness combinations that exist for the considered diaphragm design force (in general, $F_{dia} = \omega l_d$, where ω is the magnitude of the uniformly distributed load and l_d the length of the diaphragm). This approach is particularly useful to ensure that a targeted diaphragm behaviour for diaphragms in MSMBs is satisfactorily achieved.

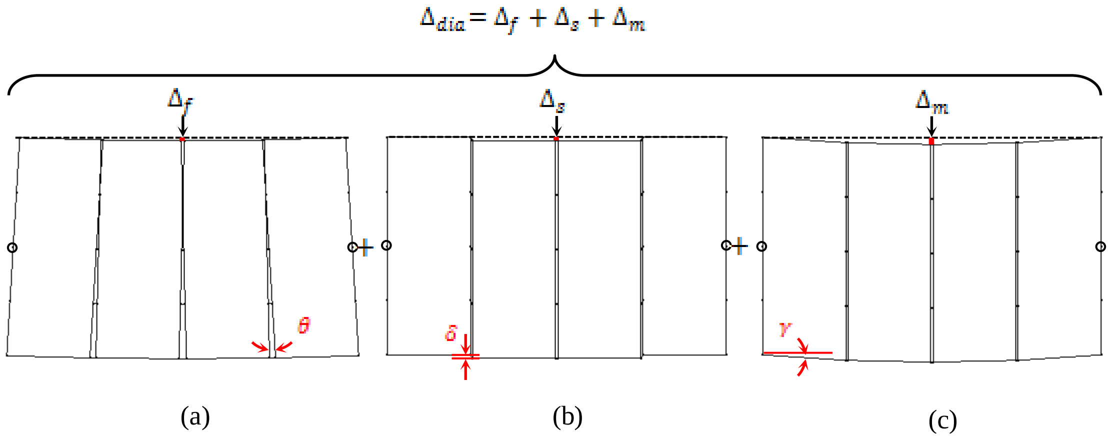


Figure 1: ***Components of diaphragm deformation as due to (a) connection axial behaviour, (b) connection shear behaviour, (c) module floor/ceiling unit shear behaviour.***

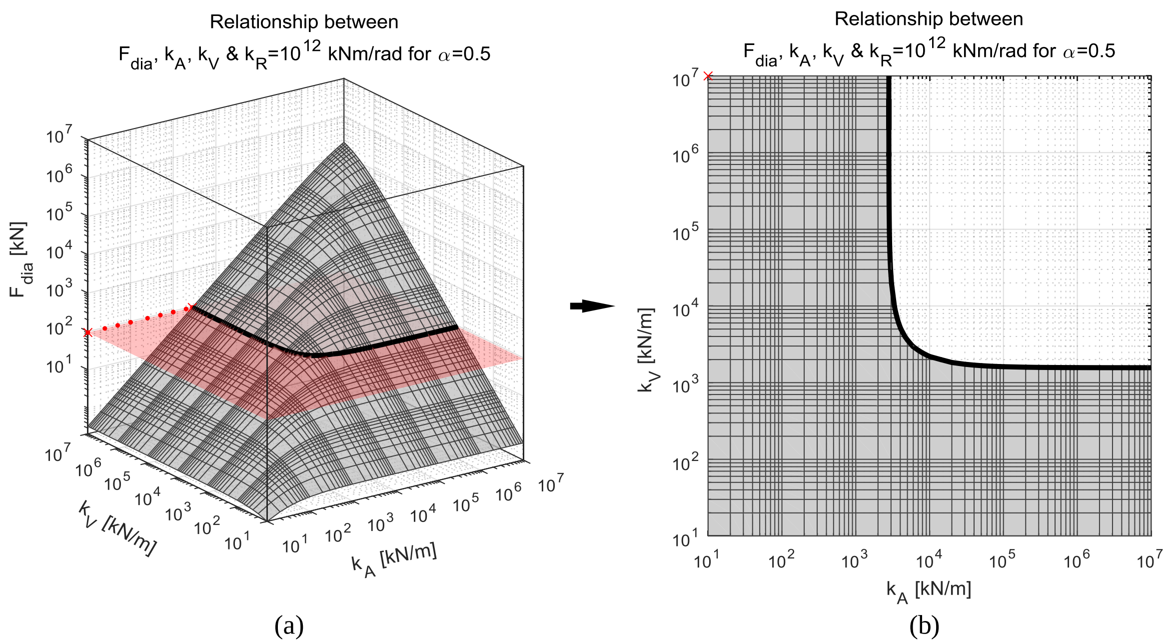


Figure 2: ***(a) Sample surface plot and (b) the range of satisfactory connection stiffness values***

3. INFLUENCE OF DIAPHRAGM FLEXIBILITY

As shown in the previous section, diaphragm connection as well as module stiffness are useful for establishing diaphragm behaviour in modular buildings. The inherent discontinuity of diaphragms in modular buildings as a result of discrete inter-module connectivity could result to diaphragms that are flexible if the provided stiffness for each diaphragm component is inadequate. Such flexible diaphragms in modular buildings are likely to cause higher mode influences when subjected to seismic loads and could potentially be catastrophic if unaccounted for. Such higher mode influences have been reported for typical buildings with flexible diaphragms when under the action of seismic loads (Lee, Aschheim, & Kuchma, 2007; Lee, Kuchma, & Aschheim, 2007; Zhang & Fleischman, 2016). Gravity frame collapse have also been reported for prefabricated parking structures during the 1994 Northridge earthquake due to the lack of rigid diaphragm action as a result of inadequate tying of prefabricated panels between themselves and to their LFRS (Iverson & Hawkins, 1994). Moreover, diaphragm design requires the determination of the diaphragm design force, where current practice mostly relies on the use of the equivalent lateral force (ELF) methodology. The ELF method establishes the vertical distribution of the elastic design base shear under the assumption that a building is of elastic rigid diaphragms and has a 1st mode dominant vibration pattern similar to that of a cantilever structure (American Society of Civil Engineers & Structural Engineering Institute, 2010; Chopra, 2012; Standards Australia, 2007; Standards New Zealand, 2004). However, analytical studies have shown that for typical low-rise buildings, the design of diaphragms for uniform strength over the entire height of the building based on an amplified top level design force, as determined through the ELF method, better accounts for the large inertial forces that may occur at lower stories and has been termed the constant strength design approach (CSD) (Fleischman & Farrow, 2001). However, similar to the CSD approach, a uniform distribution of the elastic design base shear was adopted and was termed the "modified ELF" method.

Inconsideration of the above, the following section presents the study conducted to assess the influence of diaphragm flexibility on the seismic performance of a typical hypothetical four-story four-by-four-bay modular steel building assumed to be built using rigid modules that are of $16\,m$ in length, $4\,m$ in width and $4\,m$ in height along with an inter-module horizontal spacing of $0.1\,m$. The building was simplified to reduce the computational demand involved in ground motion analysis using the software OpenSees (McKenna, 2011) through the MATLAB programming language interface. The diaphragms were modelled similar to that described in the previous section and linear elastic force-deformation properties were assigned to the two node link elements. Rigid diaphragm constraints were applied to the corner nodes of each combined module floor and ceiling unit with respect to mass nodes located at their geometric centres. The perimeter LFRS of the case study building was also simplified to provide for only the expected shear resistance and was represented by two node link elements as well with the elastic perfectly plastic force-deformation property, where yield strength (V_{yLFRS}) and deformation at yield (Δ_{yLFRS}) were determined using the relevant seismic response modification factors prescribed within the New Zealand seismic code (Standards New Zealand, 2004). The simplified model and key diaphragm nodes are shown in **Fig. 3(a) & 3(b)**.

The reference building (B_0) was selected to have perfectly rigidly diaphragms to which the modified ELF distribution was assumed and a displacement controlled pushover analysis was conducted where the values corresponding to the design base shear as determined using the New Zealand seismic code were considered as the reference set of values. Variations of the simplified model were established considering the expected behaviour of all diaphragms to be either rigid, stiff or flexible for ground motion analyses. The required stiffness properties for each diaphragm behaviour were found through the 2D diaphragm model and approach described in the previous section using the average LFRS inter-story drift (Δ_{LFRS}) at the 1st story level of the reference building (B_0). However, as indicated earlier, though numerous combinations for connection stiffness values exist, selections were made based on having approximately equal axial and shear stiffness for diaphragm connections. Following from this earlier described procedure, target diaphragm displacements were set accordingly, in consideration of the appropriate diaphragm flexibility factor (α) and the considered average LFRS inter-story drift (Δ_{LFRS}). Therefore connection stiffness values for building B_1 were established such that rigid diaphragm behaviour is guaranteed (where $\alpha = 0.5$ will correspond to connections having stiffness denoted by k_{A_1} & k_{V_1}), those of building B_2 for stiff diaphragm behaviour (where $\alpha = 2.0$ will correspond to connections having stiffness denoted by k_{A_2} & k_{V_2}) and those of building B_3 for flexible diaphragm behaviour (where $\alpha = 4.0$ will correspond to connections having stiffness denoted by k_{A_3} & k_{V_3}). Each of these three building types were subjected to 44 scaled ground motions derived from the suggested far-field earthquake record set considered within FEMA P695 obtained from the PEER ground motion database and were applied in the direction of the weak axis of the diaphragms (Applied Technology Council & Federal Emergency Management Agency, 2009). These ground motions were selected for use within FEMA P695 in consideration of satisfying the objectives set out therein, being (a) be consistent with requirements of ASCE/SEI 7, (b) be a representative collection of strong ground motions, (c) be adequate in number for statistical sufficiency, (d) be applicable to a variety of structural systems and (e) be applicable to a variety of sites. These records were used in this study by undertaking ground motion scaling by anchoring the pseudo-acceleration spectrum of each ground motion to the design spectrum corresponding to shallow soil type in Christchurch, New Zealand at the 1st mode vibrational period of each building type. 1st and 3rd modal frequencies of each building type were used for assigning Rayleigh damping considering a 5% damping ratio. A summary of key parameters used is shown in **Table 1** and the list of ground motions in **Table 2**.

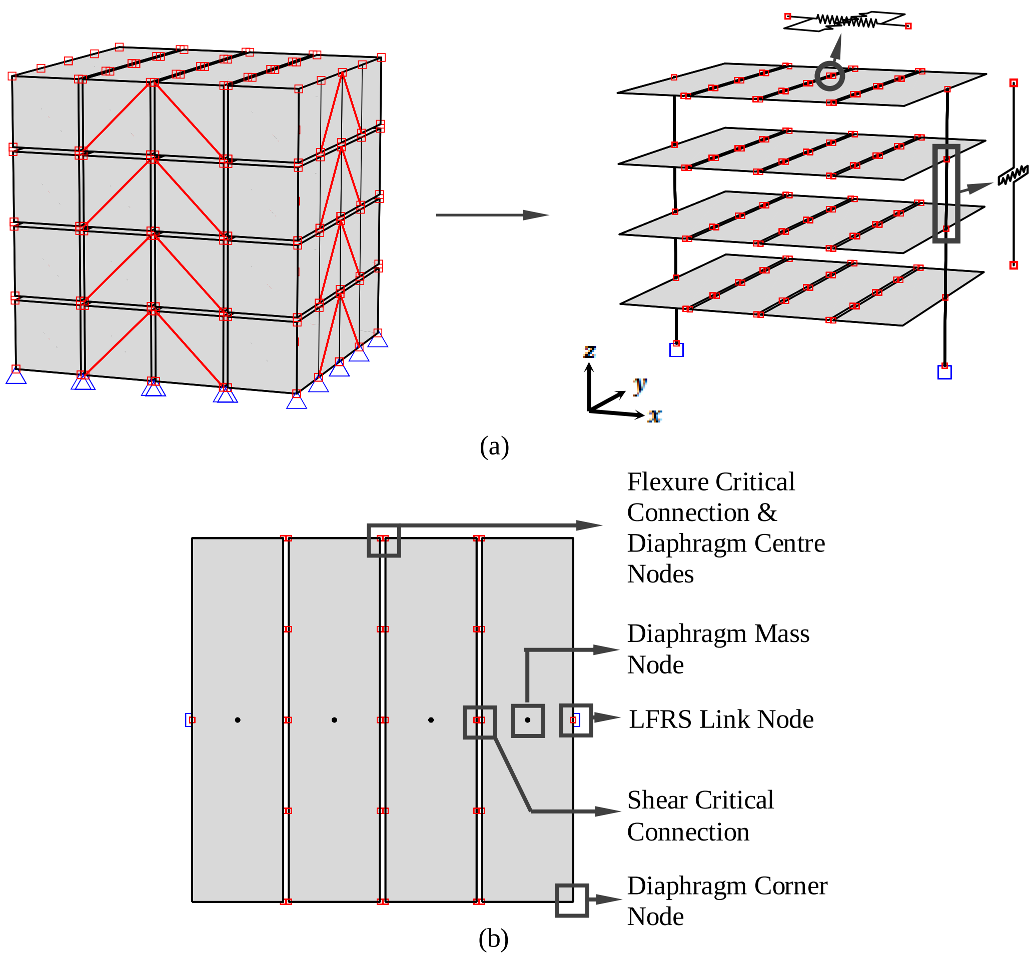


Figure 3: ***The simplified case study modular building and (b) key diaphragm nodes***

Table 1: ***Key parameters used for this study***

Parameter		
Floor & ceiling mass per module	$m_f\ [\times 10^3\ kg]$	15.0
Roof mass per module	$m_r\ [\times 10^3\ kg]$	7.5
1st mode period B_0	$T_{1_0}\ [s]$	0.494
1st mode period B_1	$T_{1_1}\ [s]$	0.533
1st mode period B_2	$T_{1_2}\ [s]$	0.649
1st mode period B_3	$T_{1_3}\ [s]$	0.801
Structural performance factor	$S_p = 1/\Omega_{LFRS}$	0.7
Ductility factor	μ_{LFRS}	1.706
Elastic base shear	$V_b\ [kN]$	375.639
LFRS shear stiffness	$k_{LFRS}\ [kN/m]$	31320
LFRS yield force	$V_{yLFRS} = V_b/2S_p\ [kN]$	268.33
LFRS yield deformation	$\Delta_{yLFRS} = V_{yLFRS}/k_{LFRS}\ [m]$	0.0086
Connection axial stiffness B_0	$k_{A_0}\ [kN/m]$	10^{12}
Connection shear stiffness B_0	$k_{V_0}\ [kN/m]$	10^{12}
Connection axial stiffness B_1	$k_{A_1}\ [kN/m]$	4625.1
Connection shear stiffness B_1	$k_{V_1}\ [kN/m]$	4000.0
Connection axial stiffness B_2	$k_{A_2}\ [kN/m]$	1194.8
Connection shear stiffness B_2	$k_{V_2}\ [kN/m]$	1000.0
Connection axial stiffness B_3	$k_{A_3}\ [kN/m]$	576.9
Connection shear stiffness B_3	$k_{V_3}\ [kN/m]$	500.0

Table 2: ***The list of far-field earthquake records used as considered within FEMA P695 from PEER***

ID	**Name**	**Recording station**	**M**	**PGA$_{dir-1}$ (g)**	**PGA$_{dir-2}$ (g)**
12011	Northridge	Beverly Hills - Mulhol	6.7	0.416	0.516
12012	Northridge	Canyon Country-WLC	6.7	0.410	0.482
12041	Duzce, Turkey	Bolu	7.1	0.728	0.822
12052	Hector Mine	Hector	7.1	0.266	0.337
12061	Imperial Valley	Delta	6.5	0.238	0.351
12062	Imperial Valley	El Centro Array #11	6.5	0.364	0.380
12071	Kobe, Japan	Nishi-Akashi	6.9	0.509	0.503
12072	Kobe, Japan	Shin-Osaka	6.9	0.243	0.212
12081	Kocaeli, Turkey	Duzce	7.5	0.312	0.358
12082	Kocaeli, Turkey	Arcelik	7.5	0.219	0.150
12091	Landers	Yermo Fire Station	7.3	0.245	0.152
12092	Landers	Coolwater	7.3	0.283	0.417
12101	Loma Prieta	Capitola	6.9	0.529	0.443
12102	Loma Prieta	Gilroy Array #3	6.9	0.555	0.367
12111	Manjil, Iran	Abbar	7.4	0.515	0.496
12121	Superstition Hills	El Centro Imp. Co.	6.5	0.358	0.258
12122	Superstition Hills	Poe Road (temp)	6.5	0.446	0.300
12132	Cape Mendocino	Rio Dell Overpass	7.0	0.385	0.549
12141	Chi-Chi, Taiwan	CHY101	7.6	0.353	0.440
12142	Chi-Chi, Taiwan	TCU045	7.6	0.474	0.512
12151	San Fernando	LA - Hollywood Stor	6.6	0.210	0.174
12171	Friuli, Italy	Tolmezzo	6.5	0.351	0.315

4. RESULTS AND DISCUSSION

Fig. 4-6 show the averages of the observed variations in lateral displacement, inter-story drift, developed inertial forces and diaphragm connection forces for each building type compared with the results of the reference building which was subjected to a pushover analysis under the assumed modified ELF distribution. Lateral displacement measurements were taken at the corners of each diaphragm as well as at the centre of longitudinal edges. Measurements for connection forces were taken at flexural as well as shear critical connections located respectively at the centre of longitudinal edges of diaphragms as well as at the centre of the lengths of combined floor units nearest to the LFRS. Developed inertial forces within diaphragms were calculated based on force measurements taken at the LFRS link elements.

As shown in **Fig. 4,** it is noticeable that with increasing diaphragm flexibility (a) lateral deformation at diaphragm corner, (b) inter-story drift ratio at diaphragm corner, (c) the lateral force required to be resisted by the LFRS and (d) the required LFRS ductility demand have all reduced. Furthermore, the gradual reduction of observed values with story height indicate a likely 1st mode dominant behaviour for the LFRS regardless of diaphragm behaviour. However, as expected, almost all measured values were larger than the reference building which would generally be considered for design. Also, the required LFRS ductility demand for building 01 was found to be larger than the considered design value at the 1st story level.

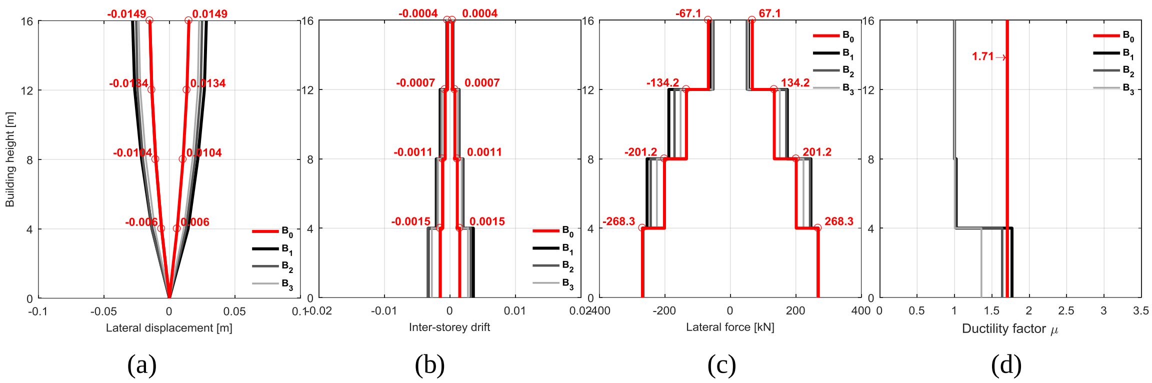
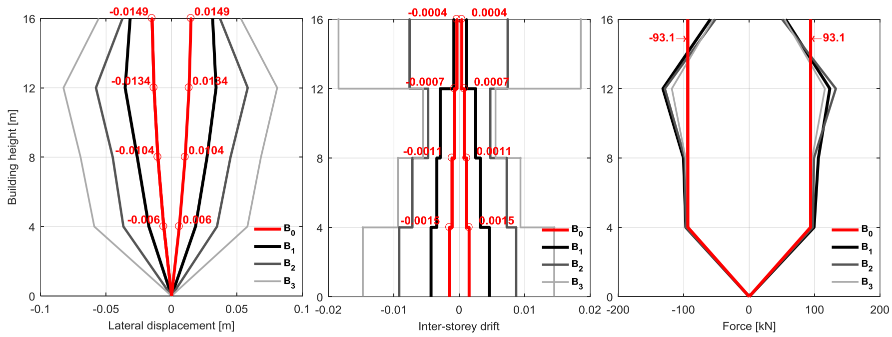


Figure 4: ***Observed LFRS (a) lateral displacement, (b) inter-story drift, (c) resisted lateral force and (d) required ductility***

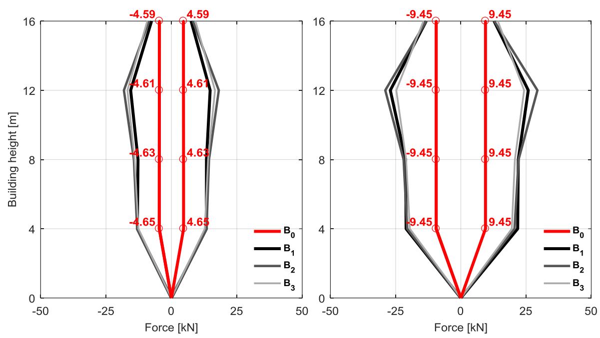
When considering **Fig. 5(a) & 5(b)**, diaphragm central deformation and inter-story drift ratios determined at diaphragm centres have increased as a result of increasing diaphragm flexibility. The largest lateral deformations are observed at the third story level for each building type and the inter-story drift ratio corresponding to the 3rd and 4th stories drastically increase as diaphragms are made more flexible. These observations indicate the influence of higher modes on the response of the overall building and more particularly the response of the diaphragm. Also, as noticeable in **Fig. 5(c)**, diaphragm inertial forces reduce with increasing diaphragm flexibility except at the third story level where the measured inertial forces for building 02 are the largest. The observed overall reduction of inertial force at the roof level as well as the increase of inertial force at the third story level for each building type with increasing diaphragm flexibility could likely be the result of out of phase motions of the reduced roof level mass to that of the story below. Moreover, it is evident that in comparison to the ELF methods prescribed within current seismic codes, the modified ELF better accounts for the large inertial forces developed at lower levels within the considered modular steel building, which could potentially avoid the likely formation of a soft first story within the building, however, it underestimates the inertial forces developed at the third story level (**Fig. 7**). This issue could be alleviated through the use of appropriate generalised seismic response modification factors for the seismic design of such diaphragms in MSMBs.



(a) (b) (c)

Figure 5: ***(a) Lateral deformation at diaphragm centre, (b) inter-story drift ratio at diaphragm centre and (c) developed inertial forces***

Fig. 6 shows diaphragm connection forces and there are noticeable similarities since these forces are dependent on the developed diaphragm inertial forces. Therefore, consequently the 3rd story flexure and shear critical connections experience greater forces than those at other stories and building 02 (B_2) experiences the largest of connection forces. Moreover, all observed values for diaphragm connection forces are significantly larger than estimates of the reference building (B_0).



(a) (b)

Figure 6: ***(a) Shear force at shear critical diaphragm connection and (b) axial force at flexure critical diaphragm connection***

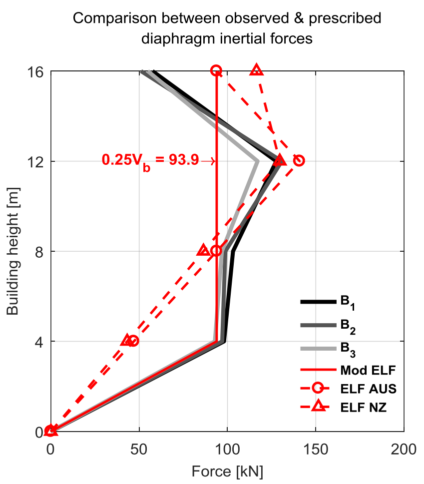


Figure 7: ***Comparison between observed diaphragm inertial forces to that determined by different ELF methods***

5. CONCLUSIONS

This paper highlights a method to determine the behaviour of diaphragms in modular buildings and presents the study conducted to assess the effects of diaphragm flexibility on the seismic performance of a case study four-story four-by-four-bay modular steel building built using rigid modules. Rigid, stiff and flexible diaphragm behaviours were achieved through the manipulation of diaphragm connection axial and shear stiffness. Each resulting building variant was subjected to a suite of 44 scaled ground motions to excite each building at the expected design conditions specified within the New Zealand seismic code for Christchurch. The results were compared with those obtained for the reference building having perfectly rigid diaphragms and being subjected to the modified ELF distribution of the elastic design base shear through a displacement controlled pushover analysis. The following conclusions are drawn from this study.

1. The increase of diaphragm flexibility has induced higher mode participation and had resulted in the increase of diaphragm deformation for the case study building, which would likely lead to increased gravity frame drifts for modular buildings in general and intensified second order effects.
2. The seismic response modification factors used for the LFRS seems to be inadequate and requires to be re-evaluated for standardised use for the seismic design of modular steel buildings with rigid diaphragms.
3. The modified ELF distribution (similar to the CSD approach) better approximates the vertical distribution of the elastic design base shear for modular steel buildings than those described within current seismic codes.
4. Diaphragm inertial forces and consequently diaphragm connection forces were largest when diaphragm behaviour was at the limit of being classified as stiff ($\alpha = 2.0$).
5. The modular steel building with diaphragm behaviour at the limit of being classified rigid ($\alpha = 0.5$), was not affected by higher modes. However it had experienced lateral deformations as well as inertial forces that were larger than those of the reference building which was based on the adherence to current design practice requirements. New design strategies for diaphragms in MSMBs are therefore required.

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