Safety of Modular Buildings in Seismic conditions

Mohamed Mafas M.M[1], Nelson T.K. Lam[2], Siddhesh Godbole[3], Pat Rajeev[4]

1. Corresponding Author. PhD student, Department of Infrastructure Engineering, The University of Melbourne, Parkville, VIC 3010, Australia.
   Email: mmohamed7@student.unimelb.edu.au
2. Professor, Department of Infrastructure Engineering, The University of Melbourne, Parkville, VIC 3010, Australia. Email: ntkl@unimelb.edu.au
3. PhD student, Department of Infrastructure Engineering, The University of Melbourne, Parkville, VIC 3010, Australia. Email: godboles@student.unimelb.edu.au
4. Associate Professor, Faculty of Science, Engineering & Technology, Swinburne University of technology, Hawthorn VIC 3122. Email: prajeev@swin.edu.au

Abstract

In seismic conditions modular buildings may behave very differently to buildings that are constructed in a conventional manner. Code compliant design of a modular building would not guarantee its resilience to earthquake ground shaking of an intensity exceeding the design limit. Intra-modular connections might have been designed in such a way which resulted in the connections failing prematurely thereby triggering collapse. This study is aimed at modelling the post-elastic performance behaviour of connections between adjacent modules as well as the response behaviour of the building using finite element modelling in combination with shaking table experiments of a scaled down model of the building. Analysis is scoped at investigating post-elastic behaviour (up to the onset of collapse) of modular buildings that feature the use of bolted splice for the intra-modular connection. The outcome of the investigation is expressed in the form of seismic performance indices characterizing the extent of damage and destabilizing actions to the building. The issue of progressive collapse has also been investigated by considering cumulative effects.

1. Introduction

1.1 Modular building construction

The concept of Modular Building Construction (MBC) has been gaining popularity over the last two decades due to the high efficiency involved in it (Azhar, Lukkad et al. 2013).Apart from the many advantages of MBC, achieving 100 percent modular construction has some challenges such as 1) the requirement of a concrete core, 2) the unavailability of generic connection system between modules, 3) lack of continuous diaphragm action etc. . These requirements could be fulfilled through a proper inter-modular connection system which would have the capability to transmit lateral and gravity loads between the modules efficiently(Lawson, Ogden et al. 2005). Most of the proposed connection systems involve bolted splice joint activity in their connection (DiMartino Sr 1986, Locke 1987, De La Marche 2005, Lawson, Ogden et al. 2014, Doh, Ho et al. 2016, Farnsworth 2016)

1.2 Hazards and unforeseen collapse (Problem definition)

The construction nature (alternate arrangement of modules and connections) of modular buildings makes the behaviour of modular buildings very different compared to that of a conventional building, especially under dynamic loadings. A homogeneity in vertical stiffness up the height of a conventional building facilitates an even distribution of capacity throughout the building in a seismic event. However, in a modular building, an alternate occurrence of framing elements and connections may distort the capacity distribution up the height of the building unlike a conventional form of construction. This uneven force distribution could prospectively be considered during the design phase of the building. Even then, there remains a chance that the building might experience an earthquake which would exceed the design limit. The consequence of occurrence of such a ground motion would be worse in the case of an MBC due to its irregularity and discontinuity in the stress distribution. The connections that attracted higher stresses could fail causing dislocation of modules which in turn will impart higher P-delta effects on the entire building. Failure of one or more connections in a single storey may also amplify the stresses in the connections of the same storey.

This study is focussed on analysing modular buildings under seismic events exceeding the design limit. Numerical model of a modular building was deployed to analyse the progression of collapse of the MBC in conditions of very strong ground shaking. This numerical model was then compared with a model of a conventional building to highlight differences in the post-elastic response behaviour of the building. To validate the numerical models developed in this project, scaled down experimental testings were performed by setting up simplified conventional and modular building models.

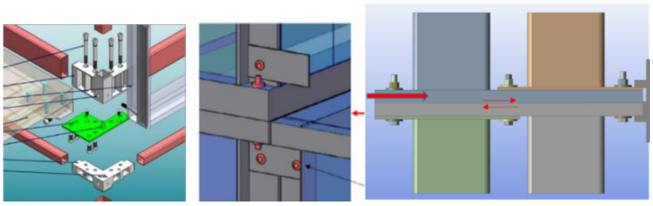
2 Structural System of Modular Buildings

2.1 Lateral force resisting system in modular buildings

Lateral Force Resisting Systems (LFRS) such as central reinforced concrete cores, internal and external bracings that are used in conventional steel and concrete building constructions are also used in MBC (Gorgolewski, Grubb et al. 2001).To increase the efficiency of MBC, bracings and LFRS are typically installed in the factory. Diaphragm action of the floor is a very important part of the lateral load resisting system in the building. Unlike conventional building constructions, floor diaphragms are also constructed offsite and individually for each module. The diaphragm action of the whole storey is maintained by the connections between the modules.

2.2 Inter-modular connections

Inter-modular connections play a very important role in the modular buildings. There have been many types of inter-modular connections till date starting from welded fixed connections to detachable connections. (DiMartino Sr 1986, Locke 1987, De La Marche 2005, Annan, Youssef et al. 2008, Heather 2013, Doh, Ho et al. 2016, Sharafi, Mortazavi et al. 2018). A common feature among mostly used connections is the bolted splice. Some of the connections introduced by different researchers are shown in figure 1.



(a) (b) (C)

Figure 1 Different types of Modular building connections(Gunawardena 2016, Chen, Liu et al. 2017)

Cyclic behaviour of different modular building connections has been studied by many researchers.(Annan, Youssef et al. 2009, Doh, Ho et al. 2016, Chen, Li et al. 2017). But research focusing on the behaviour of the modular buildings due to the presence of these connections in between modules are very rare. Fatheih et al. conducted nonlinear dynamic analysis on 2D and 3D modular buildings modelled in Opensees (Finite element solver developed by the University of California Berkley(California 1999/2000). The study was aimed at capturing interactions between modules, shear stresses and moments in the horizontal connections within the modules, and displacements of the horizontal connections (Fathieh 2013). It is shown in this study that the concentration of inelasticity (Shear & axial stresses) in these buildings was mainly in the first few story connections between the modules.

Static pushover testings on steel braced modular buildings of different number of stories which were designed under the assumption of pinned connections for horizontal and vertical connections have been undertaken (Annan, Youssef et al. 2009). Inter-modular connections which were welded (on site) to connect two adjacent modules were of interests. Welded structural elements were found to behave in a similar manner to a continuous element thereby giving welded modular units similar structural characteristics as that of a conventional steel building. In reality, access required for welding two adjacent modules was a big issue. The permanent nature of a welded connection is another issue. Finite element models have been developed assuming inter modular connections as pinned connections where the flexibility of the connections has not been accounted for.

3 Methodology

3.1 Open Sees Models of scaled down buildings

After conducting preliminary analysis on a 2D model of a modular building in SAP2000 finite element package, the authors developed a scaled down model of a modular building and a conventional steel building designed to same seismic conditions to study and compare their behaviour under increasing seismic intensities. The finite element models of the Scaled down building models were validated against physical models tested in lab using shaking table facility. Finite element models of both scaled down buildings were developed in Opensees. A numerical simulation algorithm was developed to monitor the state of every connection in the modular building. Algorithm was used to check the strain state of every connection at each time-step and to remove the connection upon its failure. Dimensions of the test models were determined based on the scaling laws which are discussed in Section 4. Both the models were 1.635m in height, 0.315m long along “X” direction and 0.45m wide in “Y” direction.

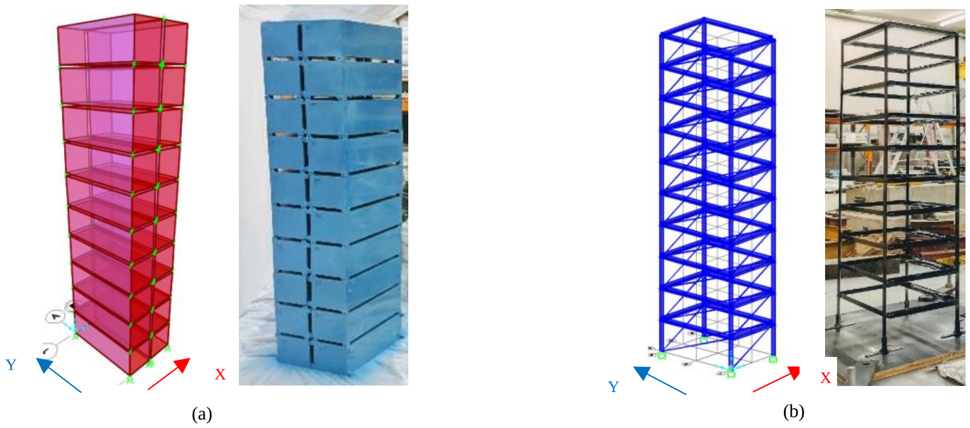


Figure 2 FEM & Experimental Models of (a) Modular building (b) Conventional steel building

3.2 Experimental Models

3.2.1 *Scaling laws used in analysis*

The models were developed based on the scaling laws adhering to dynamic similitude. Scaled testing has been undertaken in many research studies as in Refs. (Meymand 1998, Lu, Li et al. 2004, Crosariol 2010, Moss, Crosariol et al. 2010, Far, Far et al. 2015).The governing equation for dynamic similarity between a model and a prototype building can be expressed as follows.

$$S_m = \frac{S_E \times S_L^2}{S_a} \tag{1}$$

Here the notations carry the following meanings,

S_m - Mass Scaling ratio between Prototype and Model

S_L - Length (Geometric) Scaling ratio between Prototype and Model (λ)

S_a - Acceleration Scaling ratio between Prototype and Model

S_E - Material Property (Youngs Modulus) Scaling ratio between Prototype and Model

Based on the above relationship, the scaling relationship in terms of geometric scaling factor (λ) can be expressed as shown in below.

Table 1 Scaling relations in terms of geometric scaling factor (λ)

Length	λ	Acceleration	1
Time	$\lambda^{1/2}$	Frequency	$\lambda^{-1/2}$

In this study the geometric scaling factor was chosen to be λ= 20.

3.2.2 *Conventional and Modular building scaled models.*

The conventional steel structure model was fabricated using aluminium flat bars of different thicknesses for columns and braces and aluminium angle sections for beams. Appropriate amount of mass was added to each storey to achieve the desired period as per the scaling laws.

Modular structure was fabricated using aluminium boxes with 163.5mm height, 150mm width and 450mm length which corresponds to a prototype module of 3.2m height, 3m width and 9m length. Connections between these modules were chosen in such a way that the dynamic properties of the building will satisfy the design requirements.

4 Ground Motions

4.1 Seismic intensity considered in the study

Both buildings were designed for site class D at seismic hazard value of 0.08g (consistent with Melbourne conditions) for 500-year return period as per Australian Standard AS1170.4-2007 (R2018).

4.2 Loading protocol

4.2.1 *Dynamic Time history analysis of structures*

The most common method used for performing a comprehensive assessment of the behaviour of structures under seismic loads is Incremental Dynamic Analysis (IDA). Conducting an IDA could get computationally expensive. Experimental IDA of structures on shaking table can get cumbersome and expensive owing to the number of different intensity trials one might need to perform to evaluate the trend of the damage extent with increasing loading intensity.

To overcome these challenges in practise, researchers such as Estekanchi et al. (2004) came up with the concept of Endurance Time History (ETH) analysis. Since the introduction of the concept, there have been significant improvements in this method over the past decade.

4.2.2 *Endurance Time History Analysis (ETHA)*

Endurance time method is a seismic structural analysis method using intensifying dynamic loads. The structural response is monitored as the intensity of excitation is increased. Structural performance is assessed based on the response of the building as a function of the excitation intensity (Estekanchi and Vafai 2018). As the applied excitation progressively intensifies, the "time" taken to reach a certain limit state since the commencement of excitation can be taken as the measure of the ground shaking intensity and can be mapped to a more commonly used intensity measure such as Peak Ground Acceleration (PGA). The response parameter can be any measure of structural response such as top displacement, drift and similar parameters. This is very similar to an IDA in terms of output obtained but the load application process is very simple compared to an IDA.

Generating meaningful Endurance Time Excitation functions (ETEF) is essential in obtaining more realistic results that are compatible with time history analysis results. The response spectrum well represents the important characteristics of earthquake ground shaking and has therefore been adopted as the concept in developing ETEF. ETEFs are produced by considering a seismic code-based design spectrum as a template (target spectrum). A progressively intensifying excitation time-history which has the feature of producing a response spectrum that always remains to be proportional to a target spectrum at all times has been generated (Estekanchi, Riahi et al. 2008, Riahi, Estekanchi et al. 2011, Maleki-Amin and Estekanchi 2018, Mashayekhi, Estekanchi et al. 2018).

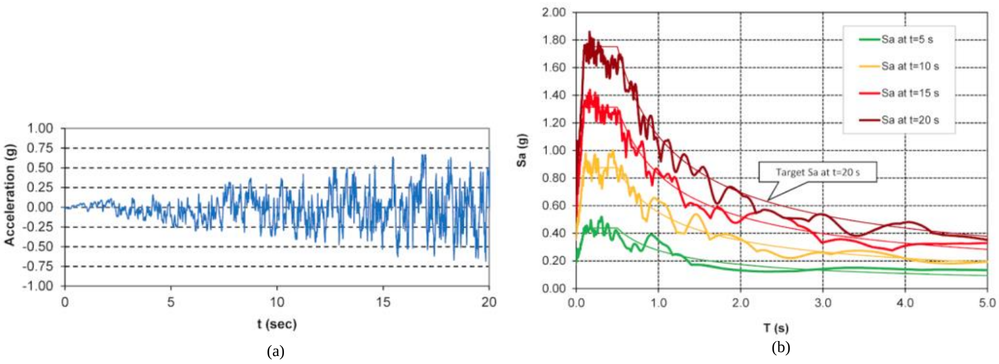


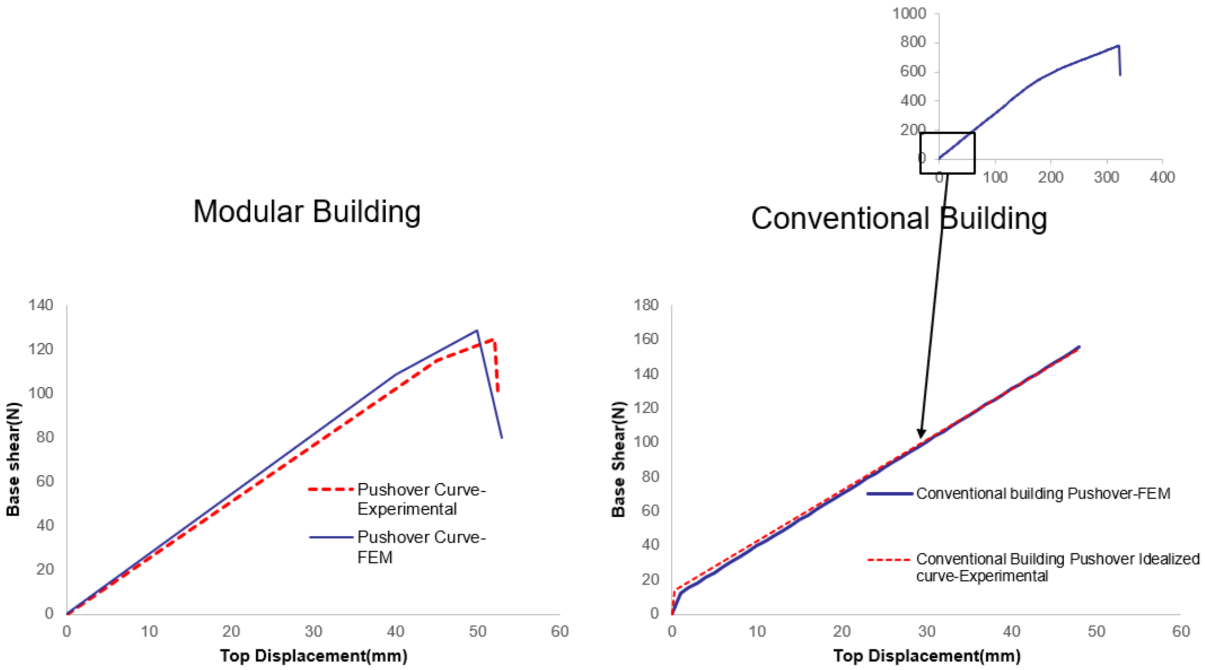
Figure 3 (a)An ETEF (b) Response spectra corresponding to different time stretches (Estekanchi and Vafai 2018)

At the time stretch: $t = 0$ to $t = T_{target}$, the response spectrum would match with the target response spectrum that had been used to generate the ETEF. In this study, ETA40lc01, being one of the ETEF's available on the Endurance time method website (Estekanchi 2007) was adopted. The FEMA P-695 far field ground motions average spectra has been used as the target spectra for generating this ETEF. The chosen ETEF was scaled in time domain as per the scaling laws introduced in section 3.2.1.

5 Results and Discussion

5.1 Static pushover analysis and FEM models validation.

Pushover analysis was conducted on the Opensees models and the experimental models developed of the buildings. The experimental and FEM pushover curves for the modular building are shown in figure 4(a) whereas that for the conventional building are shown in figure 4(b).



(a) (b)

Figure 4 Pushover curve for the (a) Modular building. (b) Conventional building.

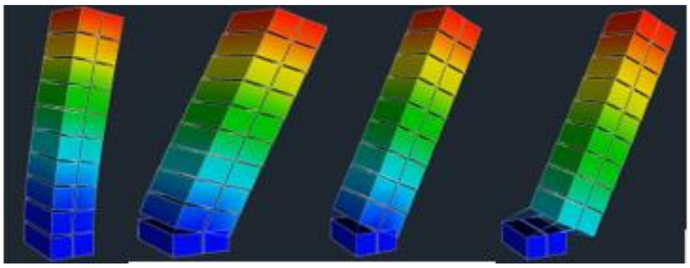


Figure 5 Failure of Modular building during pushover analysis.

The data revealed that the failure of the modular building during pushover analysis occurred due to the failure of connections in the first storey. Corner connection failure at the first storey initiated subsequent connection failures in that same storey forming a complete collapse mechanism. Whereas in the conventional building, failure started from the buckling of first storey bracings, buckling (yielding) of the braces progressed up the height of the building.

It was seen from the pushover curve of the modular building that the building yielded at a base shear of 120 N. Model weight in simulation was 47 kg meaning that the corresponding response spectral acceleration was 0.25g (at PGA of 0.08g, which is the code recommended PGA consistent with Melbourne conditions for a return period of 500 years)(Standard 2007). Similarly, the conventional building also experienced yielding at the base for the same level of earthquake intensity.

5.2 Damage variation in the buildings Under Endurance Time Excitation Function

The scaled ETEF was applied to simulation models in Opensees and to the experimental models on the shaking table to study the progressive collapse behaviour of the buildings. Displacement measurements of the building at its top were recorded as the damage measure. whereas the time was taken to be proportional to the intensity of the applied base excitation. Incremental Dynamic analysis (IDA) curves were also obtained based on the time history analysis results.

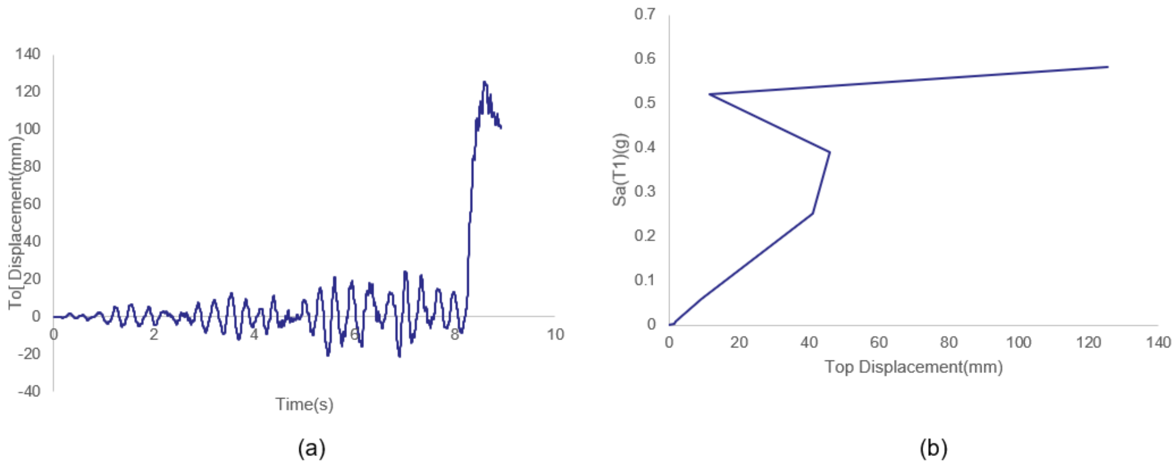


Figure 6 FEM of Modular building (a) Top Displacement variation (b) Corresponding IDA

Figure 6(b) shows the IDA curve (FEM results) of the modular building which was obtained by replacing the time axis with corresponding RSA_{max} (Response Spectral acceleration) value. As shown in Figure 7, Both the FEM building and the experimental model followed similar IDA pattern, failing under same intensity and same pattern. It was observed that the first storey corner connections (marked red in figure 7) failed at time 8.2s which corresponds to a RSA value of 0.49g. Immediately following the failure of the corner connection at the first storey, the opposite corner connection at the same storey level failed as well thereby resulting in a state of dynamic instability of the building (The rapid increase in displacement of the IDA curve occurred at this point when the connections failed).

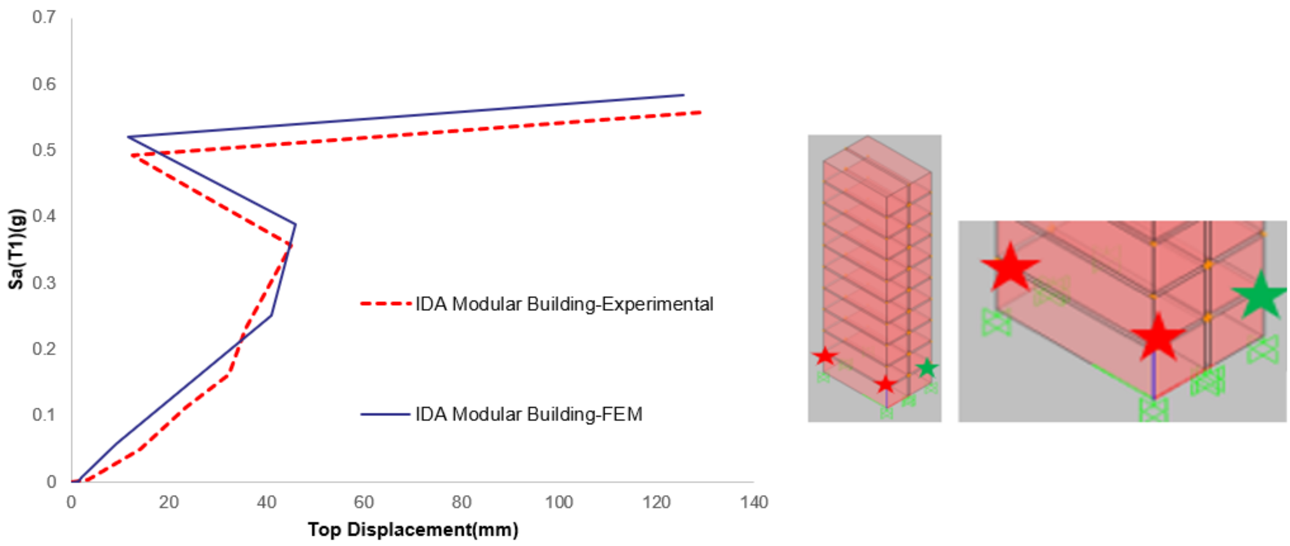


Figure 7 IDA curves from experimental and FEM analysis of modular building

Throughout the entire duration of the ETEF, only yielding (by buckling) of bracings (primary LFRS) was observed in the conventional building. Starting from the bottom most storey bracings, all the bracings up the height of the building yielded in the manner of an "unzipping" manner. This style of progressive failure is distinguished to a sudden failure of a connections in the building.

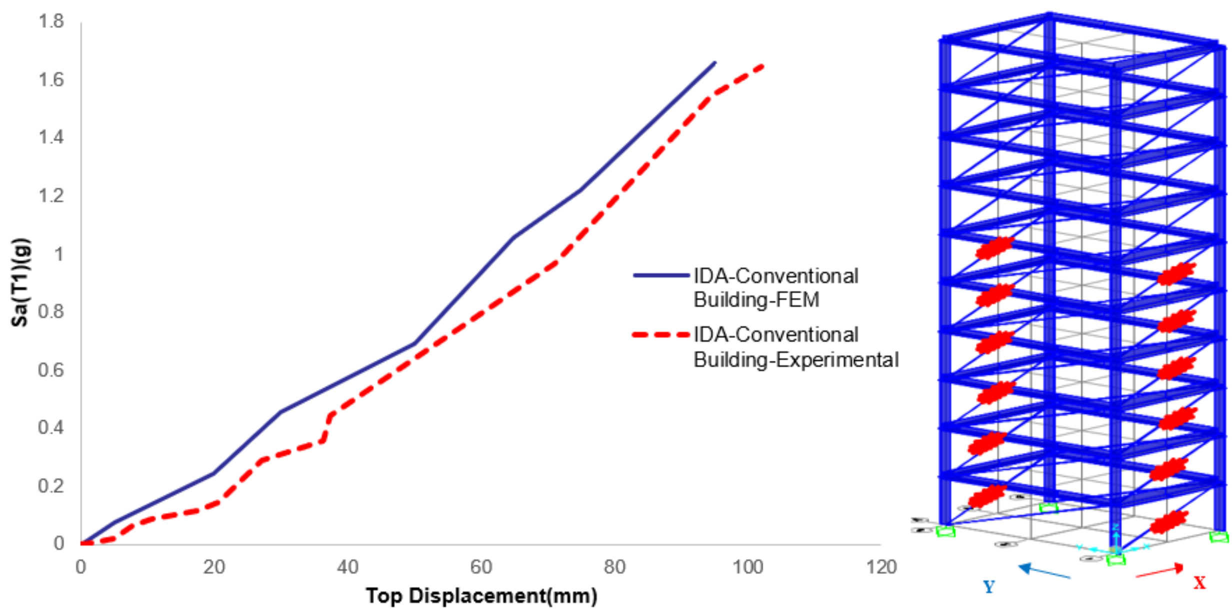
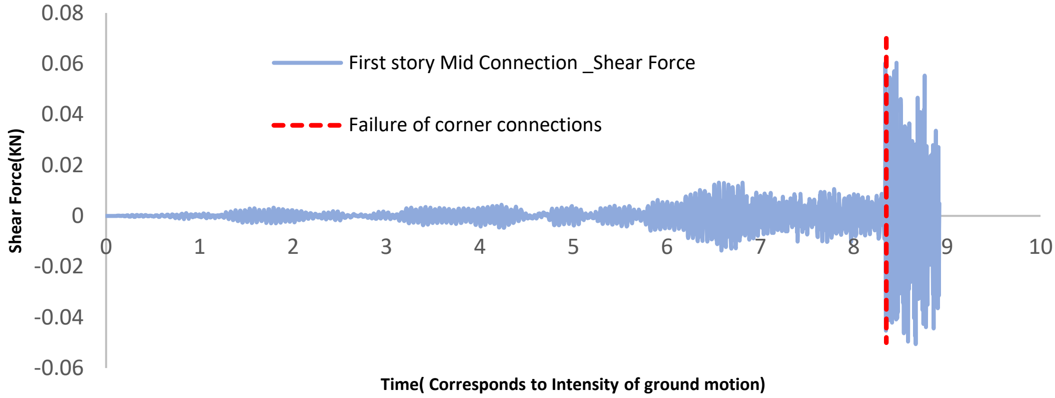


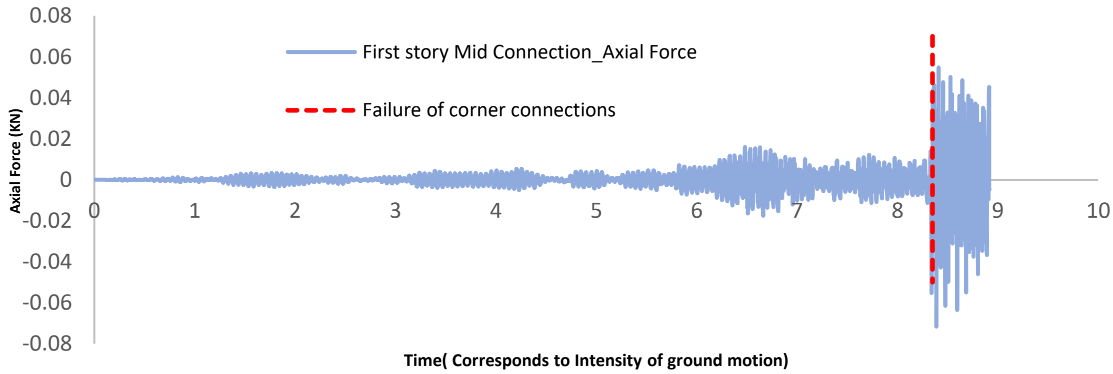
Figure 8 IDA curves from experimental and FEM analysis of modular building

5.3 Progressive collapse behaviour of the Buildings.

Lower storey connections of the modular building which were known to be most critical in a pushover tests have also been found to be prone to failure in an ETEF analysis. It was found that the first storey (left) corner connections-initiated failure which triggered a redistribution of stresses to the other connections at the same story level and eventually resulting in failure of more connections. The sudden change of the state (axial and shear force) of the connections in one story due to the failure of one or more connection in that story can be observed in figure 9 (a) & (b).



(a)



(b)

Figure 9 Sudden Increase in (a) Shear force (b) Axial force of first storey middle connections due to corner connections failure.

It is also noted that, for the building in consideration, the failure of the connections was initiated due to significant axial forces (compared to shear forces) in the connections in the building.

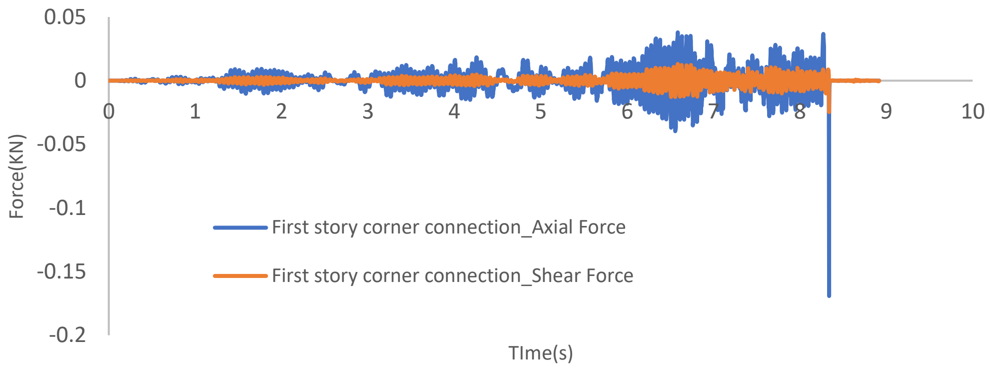


Figure 10 Comparison of axial and shear forces in a critical connection.

5.4 Future Scope

Authors are currently expanding the study to buildings with different aspect ratios to generalize the results to different buildings with different aspect ratios. Moreover, high fidelity FEM are been developed to validate the results for unscaled building.

6 Conclusions

- The first few stories were found to be very critical for both conventional and modular buildings under seismic loading.
- The progression of damage in a conventional building was up the building height whereas in a modular building, the progression of damage was mostly spanning across the storey which is indicative of a higher risk of collapse in a MBC in comparison with a conventional building.
- Failure of a corner connection in an MBC triggered failure of other connections at the same storey level, and this has resulted in a sudden jump in the IDA curve. It could be interpreted that yielding, or complete failure, of an MBC could occur suddenly without much reserved capacity in the structure. Whereas, a conventional steel structure showed a progressive failure of the structural members. Axial forces in connections of the MBC were more critical than the shear and moments in the connection.
- Endurance Time History Analysis method has proven to be a very good approach in predicting the seismic behaviour of structures compared to complicated processes like IDA.

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