Earthquakes and Structures, *Vol. 11, No. 6 (2016) 1061-1076*
DOI: http://dx.doi.org/10.12989/eas.2016.11.6.1061

Behaviour of Multi-Storey Prefabricated Modular Buildings under seismic loads

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(Received February 9, 2016, Revised June 9, 2016, Accepted July 17, 2016)

Abstract. Prefabricated Modular Buildings are increasingly becoming popular in the construction industry as a method to achieve financially economical buildings in a very short construction time. This increasing demand for modular construction has expanded into multi-storey applications where the effect of lateral loads such as seismic loads becomes critical. However, there is a lack of detailed scientific research that has explored the behaviour of modular buildings and their connection systems against seismic loads. This paper will therefore present the nonlinear time history analysis of a multi-storey modular building against several ground motion records. The critical elements that need special attention in designing a modular building in similar seismic conditions is discussed with a deeper explanation of the behaviour of the overall system.

Keywords: Multi-Storey Prefabricated Modular Buildings; nonlinear time history analysis; ductility of columns

1. Introduction

Modular construction has gained a fair amount of popularity in the construction industry in recent years in many parts of the world. This technology has already been used on low to medium rise structures around the world. A great example among many is the multi-storey apartment building 'Little Hero' in Melbourne, Australia (Fig. 1) which consists of 58 single-storey apartment modules and 5 double-storey apartment modules. The authors were also involved as part of the development team for this project. The 8 modular stories were assembled with finishes within 8 days and the building was constructed in a site with a very narrow access road demonstrating many advantages of modular construction.

Although this technology with its many efficiencies and advantages is fast spreading, there is an absence of detailed engineering research that investigates into the structural behaviour of modular buildings. This lack of knowledge may result in unsafe or uneconomically over-designed structures in order to maintain structural stability and safety.

Two main types of modules can be identified in practice in the form of load bearing modules and corner supported modules. Load bearing modules by their nature are only designed for single

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http://www.techno-press.com/journals/eas&subpage=7 ISSN: 2092-7614 (Print), 2092-7622 (Online)

Fig. 1 “Little Hero”, a multi-storey modular building in Melbourne, Australia

storey and low rise applications. In contrast, corner supported modules are more suited for multi-storey applications as their connections are capable of connecting elements both vertically and laterally unlike those in load bearing modules. The taller the building, the lateral load sharing mechanisms become more critical. Therefore a high level of understanding is required for the design of corner supported modules especially to resist lateral loads.

In this regard, this paper will focus on how a modular structure made out of corner supported modules would behave under dynamic earthquake loads.

1.1 Background

Prefabricated modules (such as apartments, office spaces, stair cases etc.) are manufactured with architectural finishes and services inside a quality controlled factory environment, ready to be delivered and assembled on site. This technology is evolving at a rapid rate with a vast variety of innovative solutions that cater to the many different needs of customers.

The introduction of prefabrication into building construction has proven to reduce construction waste by up to 52% (Jaillon *et al*. 2009) mainly through means of minimized off-cuts (Osmani *et al*. 2006). This in turn will result in significantly improved energy, cost and time efficiency of construction (Aye *et al*. 2012).

Some of the benefits and features of such building units (modules) are as follows:

• The modules can incorporate all components of a building including stairs, elevator shafts, facades, corridors and services

• The modules are constructed in a quality controlled production facility. A unit’s length, width and height can vary from project to project

• There is minimal work on site to complete the buildings as the façade and interiors themselves form part of modules

• The modules can easily be removed from the main structure for future reuse or relocation

• Modular construction at present reduces construction time by over 50% from a site-intensive building (Lawson *et al*. 2012)

• Reduced construction time means that the building starts generating revenue for the client, much sooner than it does compared to a conventional construction

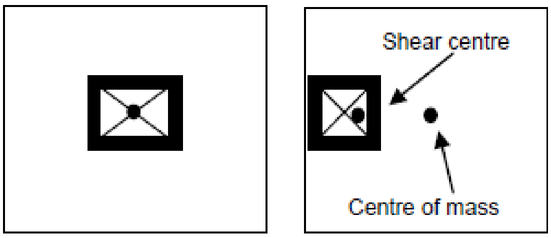
1.2 New system - A purely modular structure

Most modular buildings around the world are built as a system of modules that are eventually connected laterally to a cast in-situ or prefabricated steel core which eventually acts as the primary lateral load resisting element. Further, in most instances, the floors are poured with concrete subsequent to placing the modules. Although these methods do still add value to the construction by saving time initially, they do not still define the structure as a purely modular construction. As a result these structures do not fully enjoy the aforementioned benefits of modular construction. Gunawardena *et al*. (2016) introduced a structural system in which an assembly of prefabricated modules alone could be used to engineer a structurally feasible construction, which in result will reward the client with full benefits of modular construction. A cast in-situ or prefabricated core is not the predominant lateral load resisting component in this new structural system. The elevator core in this system is intended to be formed with steel elements as a part of some of the prefabricated modules themselves and therefore will not be the central component in the lateral load resisting system. The prefabricated modules are stacked vertically and connected horizontally through bolted plates.

The lateral load transfer mechanism is provided through these connections and improved greatly through the introduction of modules with stiff concrete walls. These stiff modules which are strategically placed in the main structure resist the majority of lateral loads and transfer them down to the foundation. As a result the structure would not require a traditional central structural core. The structure can now act as a purely modular system.

This new system provides architects with a great degree of freedom with a structure that is not limited by the placement of a core that takes critical lateral loads. In conventional structures the core would ideally be situated at the centre so that the shear centre of the structure coincides with the centre of mass thus reducing torsional forces. Fig. 2 illustrates this phenomenon.

Since the stiffer modules take the lateral loads, the elevator shaft can be placed anywhere in the building as the architect pleases considering the vertical transportation requirement. This provides a special design with a greater versatility and room for innovation.



(a) (b)

Fig. 2 Placement of the core in conventional buildings; (a) concentric core where torsional forces are minimized, (b) eccentric core where higher torsional forces apply on the structure

This new structural system is equipped with the potential to further reduce construction time and cost while also adding to the range of innovation that designers would relish. It is therefore used in creating the case study for the analysis presented in this paper where the purely modular structural system is checked for its behaviour under earthquake loads.

2. Expected behaviour under seismic loads

A cross section of the module-module connection which transfers loads both vertically and horizontally, is sketched in Fig. 3. The possible pinned connections are not shown in order to prevent confusion with the possible hinge locations shown in the diagram. One such connection would connect four supporting columns from four adjoining modules.

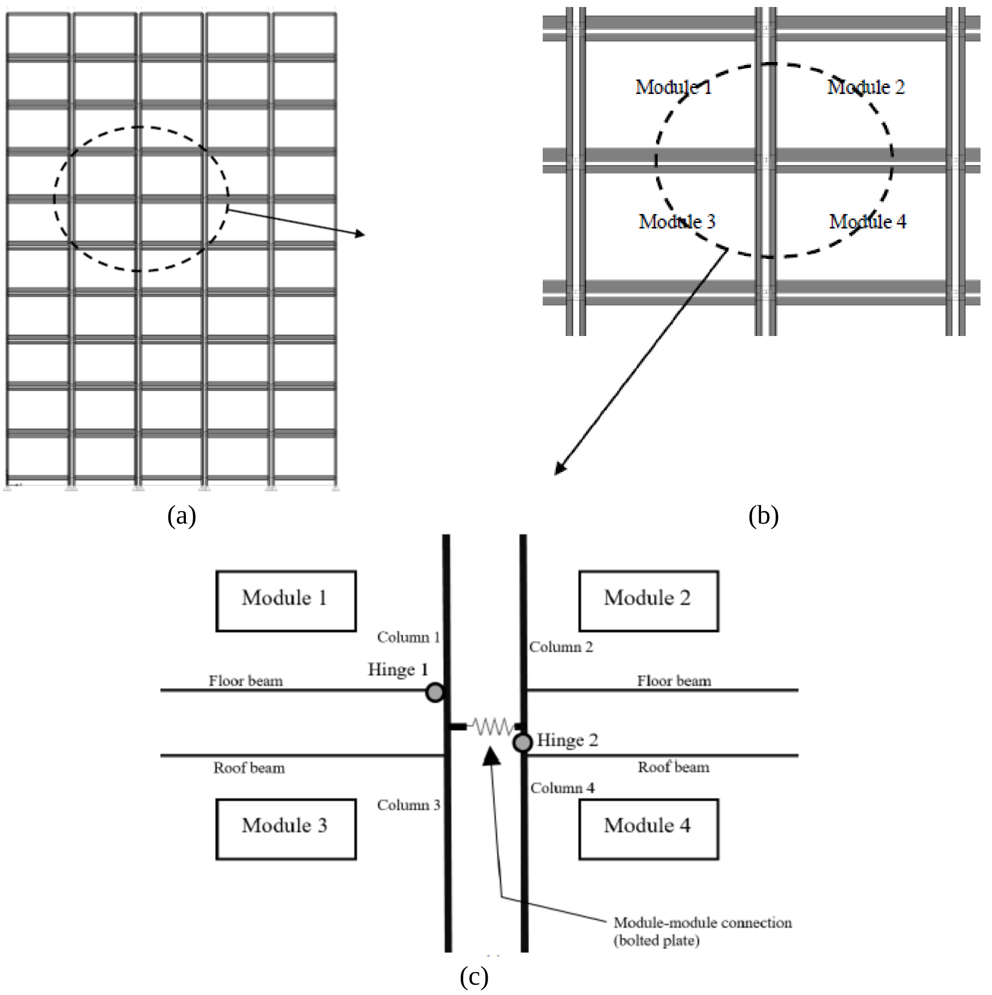


Fig. 3 An illustration of the module-module connection and possible hinge locations (c) zoomed in from the front elevation of a modular building (a) and a close-up of the elevation view of neighbouring modules (b) connected by this module-module connection

As shown in Fig. 3 the module-module connection is intended to be facilitated with the use of a bolted plate connecting all four columns. This plate will transfer lateral loads as shear forces and therefore be designed to carry the ultimate shear force that occurs at its location. It is therefore unlikely that the initial hinge formation will occur at this connection during an earthquake.

Two of the possible hinge locations are marked in Fig. 3. Hinge 1 which may form inside the floor beam of a module (or a roof beam if adequately loaded) would be a less likely scenario in the newly introduced structural system. The infill concrete walls together with the roof and floor beams of the module would act as a deep beam of which the depth is the height of the module itself. Therefore this deep beam would be too strong to yield before the main column (Columns 1 to 4 in Fig. 3). It is therefore expected that Hinge 2 which may form on the main corner column just below or above the module will be the more likely scenario of a hinge formation during an earthquake. This hypothesis is tested through the non-linear time history analysis presented in this paper.

3. Methodology for nonlinear time history analysis

A hypothetical ten-storey medium-rise building designed as a modular building with the new structural system as discussed previously is considered for the analysis. The building is modelled using the software RUAUMOKO 3D (Fig. 4) where non-linear time history analyses are carried out against six selected earthquake ground motions.

Mainly two types of modules are used in the building where one type (Type 1 - Fig. 5) has stiffer reinforced concrete walls of 100 mm thickness and the other (Type 2 - Fig. 6) without such reinforced concrete infills. In current practice the outer walls of such prefabricated modules are constructed with various lightweight materials including thin steel sheets and sandwich panels. Each storey of the ten-storey building consists of five modules which are each 4.2 m wide and 10.8 m long as shown in Fig. 4. The modules shown in Figs. 5 and 6 are corner supported modules and their corner columns are rectangular hollows sections (RHS) of the size 220 mm×150 mm×14.2 mm. Each module consists of 6 main supporting columns (Fig. 4). Since the building is only ten stories tall the columns size remains the same for all stories.

Each of Type 2 and Type 1 modules are made up with a self-stabilised steel frame. The modules need to be structurally stable by their own since it is necessary for transportation and on-site handling. This internal frame includes internal columns made up of parallel flange channels (PFC) of the type 180 PFC. All of the floor beams in each module are spaced at 1.8 meters from each other and are made up of 300 PFC sections. The roof beams are also spaced at 1.8 meters from each other and are made up of 180 PFC sections. All steel members are considered to be with a yield strength of 350 MPa.

All frame elements such as columns and beams were modelled using the element type ‘BEAM’ (Carr, 2010). All such ‘BEAM’ type elements follow the ‘Giberson one-component beam model’ defined by Sharpe (1974). The stiff walls in Type 2 modules were modelled using the element type ‘QUADRILATERAL’ (Carr 2010) which follows a shell element definition. An extract of the input file that was written for this model is presented in Appendix A.

Columns and beams that make up the load transferring system are part of the prefabricated module, and their continuity is ensured through the stiffness of the bolted connection plates. The lateral connections are modelled as spring elements with equivalent stiffness and pinned at the common joint to allow independent rotation of the connected modules (Annan *et al*. 2009).

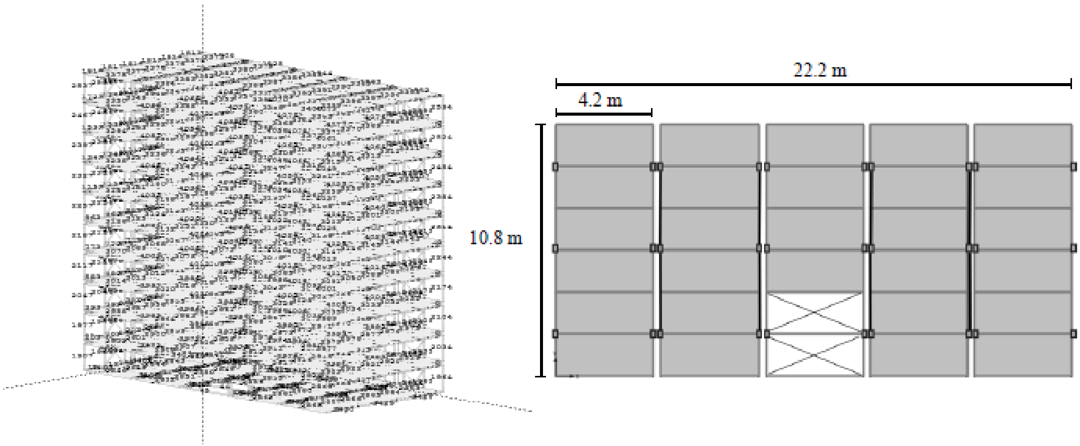


Fig. 4 The RUAUMOKO model created for the nonlinear time history analysis and a sketch of the plan view of a typical floor. (An extract of the RUAUMOKO input file is attached in Appendix A)

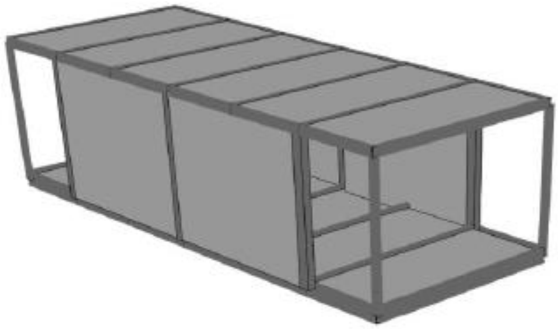


Fig. 5 Type 1 module with 100mm thick RC infill wall

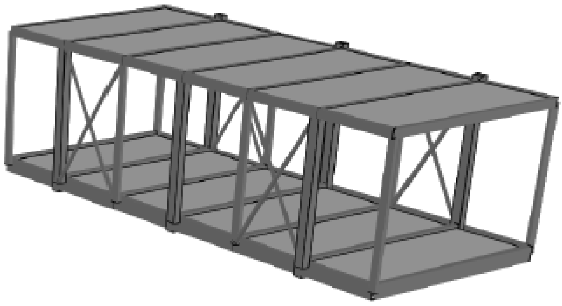


Fig. 6 Type 2 module (without infill walls)

These module-module connections are assumed to be rigid connections for the purpose of forming the computer model as proposed by Annan *et al*. (2009). The stiffness of the module-module bolted connection was calculated following the method proposed by Wileman *et al*. (1991).

The nonlinear time history analysis was carried out in order to identify which of the expected hinges would occur first and also to observe the moment curvature relationship of the critical main corner columns (Fig. 3). This would result in a more thorough understanding of the ductility of the columns under such earthquake loadings. All steel structural elements were specified with a bilinear hysteresis profile as per the RUAUMOKO 3D Manual (Carr 2010). The ductility of the column frame member shall be found out in the form of curvature ductility (Carr 2010) which is defined as

$$\text{Ductility } (\mu) = \text{Maximum curvature } (\phi_m) \text{ / Yield curvature } (\phi_y) \quad (1)$$

In addition to the nonlinear time history analysis, a pushover analysis was also carried out in order to obtain the maximum yield curvatures of the columns where hinges were expected. The generated pushover curve was further used to carry out a Capacity Spectrum Analysis.

As per Eurocode 8 (EC8: 2004) six earthquake records were selected to be used for the nonlinear time history analysis where the average response can be considered for results. These six records were selected

Table 1 Earthquake records used for the Nonlinear Time History Analysis

Earthquake Record	Magnitude (Moment Magnitude Scale)	Duration	Time Step	PGA	Scale
Chi chi	7.63	90.0 s	0.005 s	0.278 g	1
Düzce	7.14	25.8 s	0.005 s	0.535 g	1
Imperial Valley	6.90	40.0 s	0.005 s	0.519 g	1
Kocaeli	7.40	27.5 s	0.005 s	0.312 g	1
Loma Prieta	7.10	60.0 s	0.005 s	0.371 g	1
Tabas	7.40	16.0 s	0.005 s	0.852 g	1

to match the intensity of Type 1 earthquake as per EC8: 2004 with an assumed peak ground acceleration (PGA) of 0.4 g and soil type C where the standard penetration test (SPT) number of blows ranges from 15 to 50. The six short duration earthquake records that were chosen for the time history analysis are shown below in Table 1.

4. Discussion of results

4.1 Hinge formation

It was observed through the nonlinear time history analysis that as expected, the hinge formation occurs first on the supporting columns below the floor beam. In this regard it was observed that the columns below the 1st storey Type 1 modules were the most critical. The results are presented here for one of those columns (Column 3411 as per the model).

The hinge stiffness variation through the duration of the applied earthquakes indicates that the column forms hinges while the module-module connections and the floor beams remain elastic. Thus as per Fig. 3, “Hinge 2” is where the hinge formation has occurred.

The stiffness curves are plotted for the most critical column (Columns 3411) for each of the applied earthquake time histories in Fig 7. A continuous curve at 100% stiffness would show an element remaining elastic through the duration of the applied earthquake time history (Carr 2010). However, the plotted curves show the stiffness of the column at “Hinge 2” location dropping close to 0% which indicates that it has yielded.

4.2 Curvature Ductility

The moment curvature relationships that were generated for the critical column (of which the stiffness variation was shown through Fig. 7) against the six earthquake time histories are shown in Fig. 8.

The gradient of the curve while the element is still elastic equals the value of elastic modulus (E) times the second moment of area in the bending direction (I). Therefore, where the curve changes its gradient shows where the value of EI has changed thus indicating yielding.

The column has deformed following the specified bilinear hysteresis rule and where the bi-linearity starts in each curve indicates the yield moment and yield curvature (ϕ_y) of the column (Bouzid and Kassoul 2016). The value for yield curvature as observed is taken to be 0.02 rad.

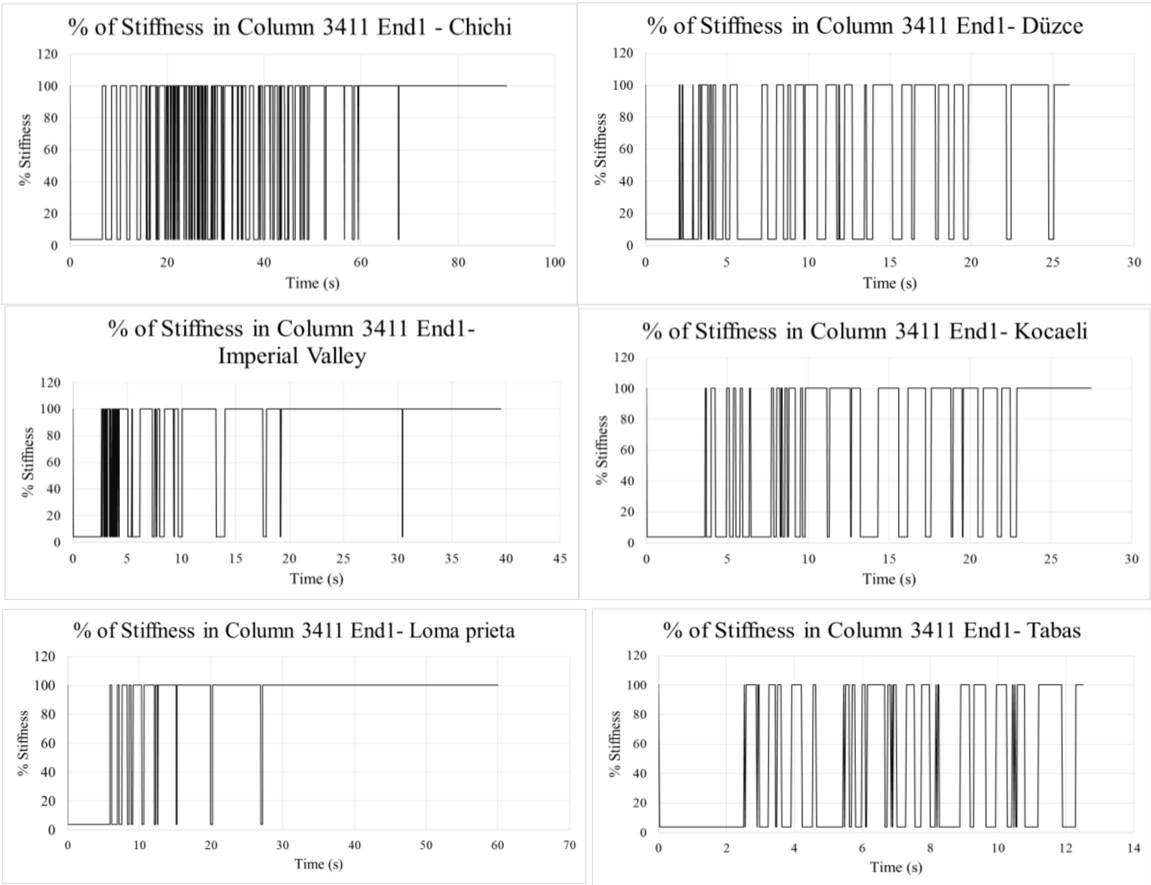


Fig. 7 Stiffness variation with time compared between the most critical column and the module-module bolted connection plate subject to the six earthquake time histories

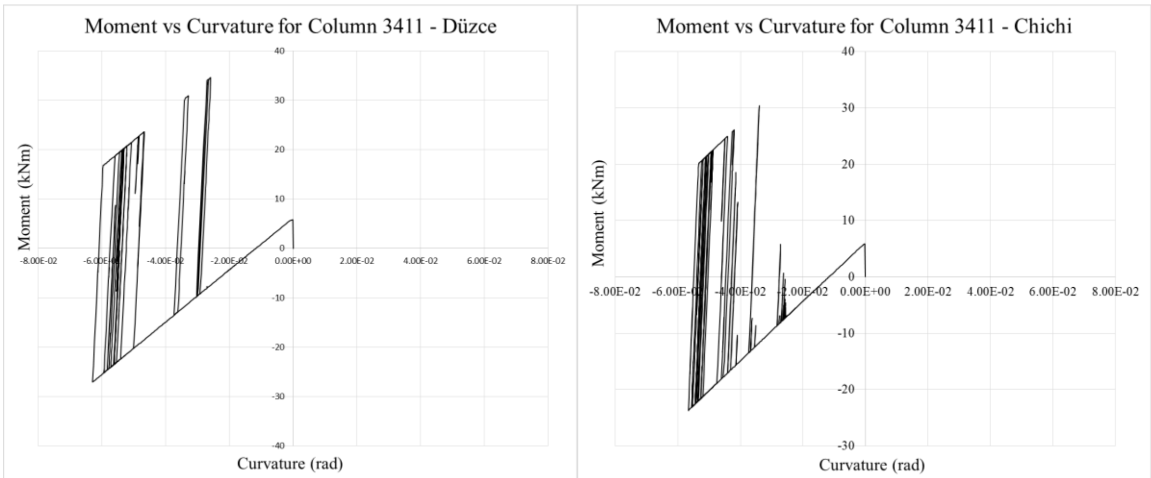


Fig. 8 Moment-curvature relationship for the most critical column subject to the six earthquake time histories

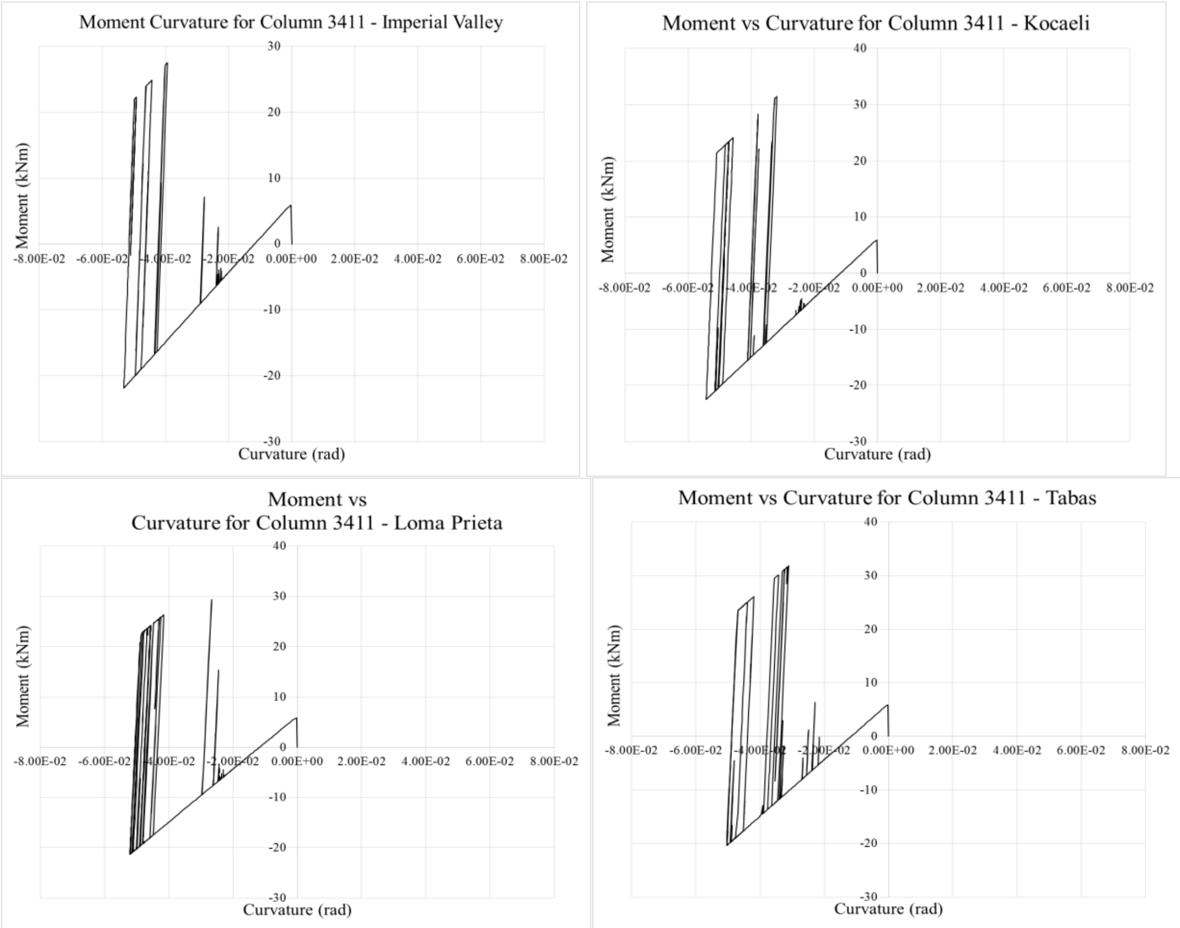


Fig. 8 Continued

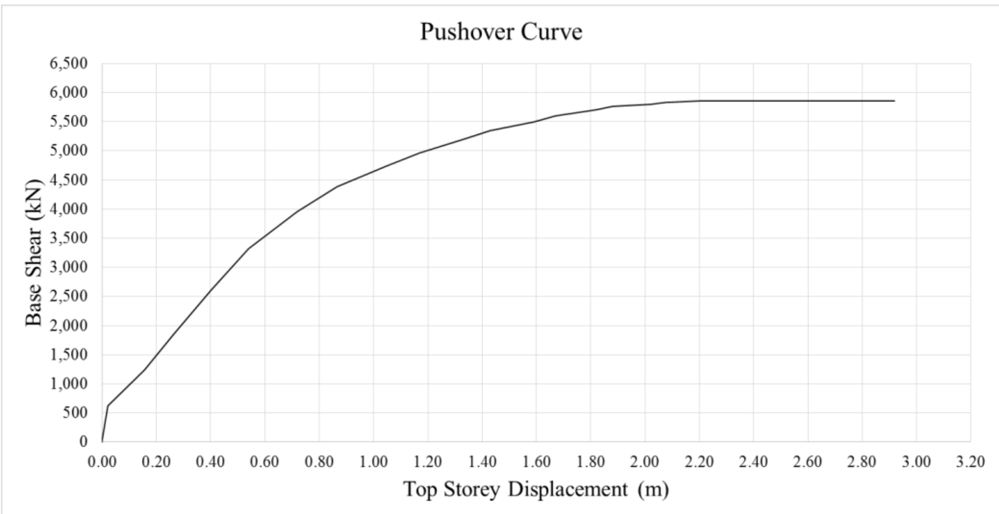


Fig. 9 Pushover Curve generated from the nonlinear static pushover analysis for the 10 storey building

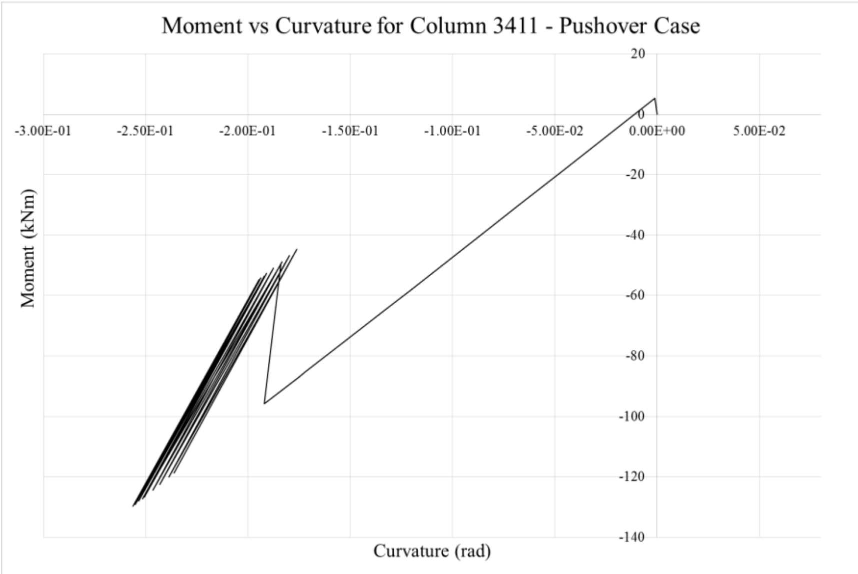


Fig. 10 Moment-curvature relationship for the most critical column generated from the nonlinear static pushover analysis

Although ultimate failure cannot be observed here, a common maximum curvature can be observed at approximately 0.05 rad.

Using the above observations a ductility value of 2.5 (0.05 rad/0.02 rad - from Eq. (1)) can be observed for the critical column. As per AS 1170.4: 2007 the Australian standard for earthquake loading, this value of ductility could be categorized as limited to moderate ductility.

The pushover curve generated is shown in Fig. 9 and the moment curvature relationship generated for the same critical column is shown in Fig. 10. It was observed that the yielding started to occur at a curvature of approximately 0.18 rad and had a maximum curvature of approximately 0.26 rad.

Similarly a ductility value of 1.4 (0.26 rad/0.18 rad) can be derived for the critical column. As per AS 1170.4: 2007 this ductility falls below the limited ductile range, however since this is a static loading scenario a lower ductility than the outcome of the time history analysis may be expected.

4.3 Capacity spectrum analysis

The Capacity Spectrum Method which was first developed by Freeman *et al*. (1998a, b) is one of the simplest ways to estimate the seismic performance of a structure by comparing its capacity against its demand for applied earthquake forces. The pushover curve that was generated from the nonlinear static analysis as presented in Fig. 9 was converted to a capacity curve in the ADRS (acceleration displacement response spectrum) format by following the conversion formulae (Eqs. (2) to (5)) proposed by Chopra and Goel (1999). Similarly the response spectra of the applied earthquakes which are typically in the acceleration vs time format were also converted to the ADRS format by following Eq. (6).

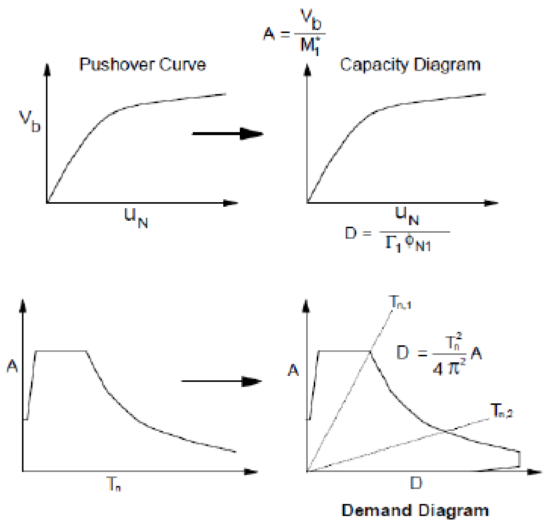


Fig. 11 Converting pushover curve to a capacity diagram and acceleration response spectrum to a demand diagram (Chopra *et al.* 1999)

$$A = {V_b}/{M_1^*} \tag{2}$$

$$D = {U_N}/{\Gamma_1 \phi_{N1}} \tag{3}$$

$$\Gamma_1 \phi_{N1} = \frac{\sum m_j \phi_{j1}}{\sum m_j \phi_{j1}^2} \cdot \phi_{N1} \tag{4}$$

$$M_1^* = \frac{\left[\sum m_j \phi_{j1}\right]^2}{\sum m_j \phi_{j1}^2} \tag{5}$$

$$D = \frac{T_n^2}{4\pi^2} A \tag{6}$$

Where;

V_b = Base Shear
U_N = N^{th} Storey deflection (N^{th} storey is taken as the roof)
m_j = Lumped mass at the j^{th} floor level
ϕ_j = j^{th} floor level element of the fundamental mode in the considered direction
N = Number of floors
M^*_1 = Effective modal mass of the fundamental mode in the considered direction
D = Deformation spectrum ordinate
T_n = Natural period

The response spectra for all considered earthquakes (Table 1) was converted to demand curves as described above considering 5% viscous damping. A combined diagram of the 5% damped demand curves for all six considered earthquake records alongside the capacity curve of the building is shown below in Fig. 12.

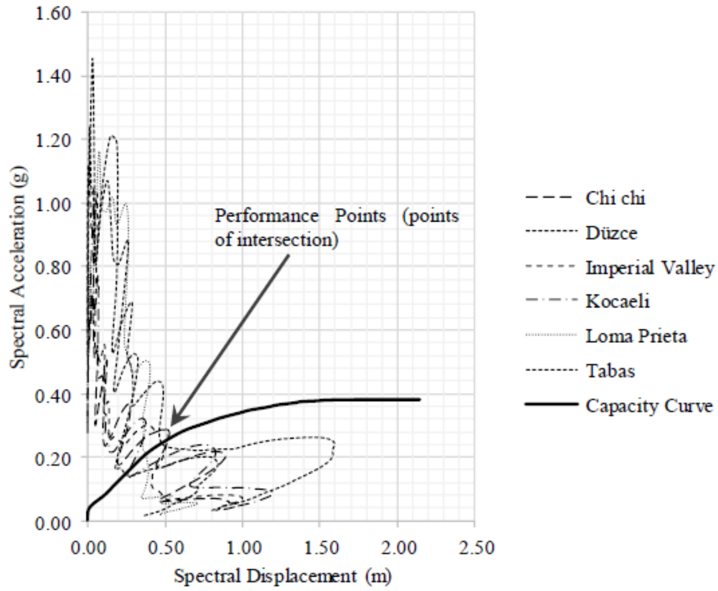


Fig. 12 Combined 5% damped demand curves against capacity spectrum

As per Fig. 12 it appears that the structure is past its linear deformation zone at its performance points against all six earthquake time histories. The performance points are also far below the full capacity of the structure. Therefore, it can be deduced that the structure performs approximately in the ‘Immediate Occupancy’ to ‘Life Safety’ zone as per the performance levels described in FEMA 356 (2000). This is a valuable preliminary estimate of the performance of this structure against the earthquakes applied.

4.4 Health monitoring for better seismic performance

Subsequent to a major earthquake it is highly important to investigate the condition of a structure to ensure the safety of occupants and establish the suitability of the building for future use. It is also important to monitor the response of the structure in an actual scenario to establish the reliability and accuracy of analytical data. In this regard, more than 250 structures in the USA have been fitted with seismic sensors to improve the understanding of the effect of earthquakes on the behaviour of structures (USGS 2015).

As far as corner supported modular buildings are concerned, the monitoring of corner supporting columns and the module-module connections is very important. Their behaviour and the actual deformations during a real earthquake shall provide valuable information in forming more safe and optimised design solutions. Romo *et al*. (2015) have proposed a similar health monitoring system that can be placed to assess the damage caused by earthquakes on structural dampers. Similar methods can be applied on modular buildings as well.

In addition to the critical structural elements, the state of non-structural elements would also be highly important and require a thorough check subsequent to a major earthquake. However, due to the unique advantages of modular buildings, any damaged components be they structural or non-structural, can be easily removed and replaced provided the replacements are readily available.

5. Conclusions

• The elements that join at the module-module connection play a critical role in how lateral loads are transferred and resisted in a modular building. The bolted steel plates that connect neighbouring columns are the key component in transferring lateral loads. They also act as the key connections in which the columns are vertically connected. Therefore, these connections need to be designed to take the full shear force of the lateral loads.

• By the results observed from the nonlinear time history analysis, a hinge formation in the main corner column just under the floor beam or above the roof beam ("Hinge 2" as per Fig 3) proves to be the most likely scenario out of the possibilities discussed. The formation of such hinges in a main column is not a desirable scenario as it may lead to collapse during a large earthquake. However, prevention of these hinges from forming may not always be possible under large earthquakes. Alternatively the structure could be designed in such a way that the hinge formation occurs elsewhere, possibly on the floor beams of less stiff modules that do not carry concrete walls. A unique advantage of a purely modular building is the ability to remove damaged modules and have them replaced. The removed modules can also be repaired and reused elsewhere on other suitable projects.

• The capacity spectrum analysis provides a valuable preliminary assessment that this particular structural concept can perform within acceptable levels. Although this assessment will vary depending on the site conditions and severity of considered earthquakes for a given project, the initial assessment shows that this structural system can be constructed to perform without collapsing by properly designing connections and the supporting columns.

• The ductility of the corner columns is also a critical criterion to maintain in order to achieve a better performance under high seismic loads. Although yielding may occur, high ductile columns may still prevent collapse and assist the horizontal members to redistribute the loads among neighbouring modules.

• However, in a severe earthquake, it may prove very difficult to prevent the columns from forming hinges. As such, the immediate investigation into the condition of the structure after a large earthquake is highly important. Structural Health Monitoring (SHM) therefore plays a key role in improving the seismic performance of such structures and also in improving the understanding of the behaviour of these structures under dynamic earthquake loads.

• In conclusion it must be noted that this structural system has many advantages which ensure highly profitable outcomes to both builders and developers. However, where such buildings are required to be constructed in high seismic areas special care must be taken to ensure that the critical columns are not vulnerable to hinge formations. The unique reusability characteristics of modular construction allows for a better chance at designing to sacrifice less critical elements in a large earthquake and safeguard human lives by preventing collapse.

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Appendix A

An extract from the input file for the RUAUMOKO model is given below for further reference. (Since the text entered for the entire model is too long for the nature of the paper, only key inputs are shown)

```
10 Storey Modular Building with Earthquake Time Histories
2   1       0       0       1       0       0       0       1       0       ! Control Parameters
0   0       1       ! EQ Directions
1700        4078    11      12      1       2       9.81    5       5       0.005   26      1       ! Input
5   5       10      1.1     10      10      10      0       0       0       1       0       ! Output
DEFAULT
0   0       0.00001 0       0       0       0       0       0       0       0       ! Iterations

NODES       0
1   0       0       1.8     1       1       1       0       0       0       0       0
2   0       0       5.4     1       1       1       0       0       0       0       0
3   0       0       9       1       1       1       0       0       0       0       0
4   4.2     0       1.8     1       1       1       0       0       0       0       0
5   4.2     0       5.4     1       1       1       0       0       0       0       0
.
.
.
.
PROPS
1   BEAM "180 PFC"
1   4       0       0       0       2       0       0       0       0
2.00E+08    7.69E+07        2.66E-03        8.10E-08        1.41E-05        1.51E-06        1.65E-03        1.08E-03
    6.42E-02        0       0.20748
0
0   0       0.04    0.04
0   0       0       0
1197        -1197   0       0       2       0
81.9-81.9   24.2    -24.2

2   BEAM "300 PFC"
1   4       0       0       0       2       0       0       0       0
2.00E+08    7.69E+07        5.11E-03        2.90E-07        7.24E-05        4.04E-06        2.88E-03        2.40E-03
    7.47E-02        0       0.39858
0
0   0       0.04    0.04
0   0       0       0
2069        -2069   0       0       2       0
253.8       -253.8  52.7    -52.7

3   BEAM "180 PFC-col"
1   4       0       0       0       2       0       0       0       0
2.00E+08    7.69E+07        2.66E-03        8.10E-08        1.41E-05        1.51E-06        1.65E-03        1.08E-03
    6.42E-02        0       0.20748
0
0   0       0.04    0.04
0   0       0       0
1197        -1197   0       0       2       0
81.9-81.9   24.2    -24.2

4   BEAM "100x75x8 UA"
1   4       3       3       0       0       0       0       0       0
2.00E+08    7.69E+07        1.31E-03        2.80E-08        1.30E-06        6.26E-07        5.85E-04        7.80E-04
    3.340E-02       -4.530E-02      0.10218
0
```

```
5    QUADRILATERAL     "Slab 125"
0    0    3.30E+07    0.2    0.125    40.25    2    0    0    0

6    QUADRILATERAL     "Slab 100"
0    0    3.30E+07    0.2    0.100    25.2    2    0    0    0

7    BEAM “100 infill Wall”
1    4    0    0    0    0    0    0    0    0
3.3E+07    1.38E+10    0.18    5.79E-04    1.5E-04  4.86E-02    0.15    0.15    0    0    4.32
0

8    SPRING      "Connection"
2    1    0    0    0    0    0
700000    500000  700000  700000  700000  700000  3.06    1    1
480 -480    480    -480    480    -480    1
4146    -4146    613    -613    4146    -4146    2

9    BEAM "220x150x14.2 SHS Column"
1    4    0    0    0    2    0    0    0    0
2.10E+08    8.08E+07    9.82E-03    6.80E-05    6.30E-05    3.53E-05    4.37E-03    6.25E-03
     0    0    0.77087
0
0    0    0.04    0.04
0    0    0    0
3435.25    -3435.25    0    0    1.5    0
248.75    -248.75  192.045 -192.045

10   BEAM "220x150x14.2 SHS Column"
1    4    0    0    0    2    0    0    0    0
2.10E+08    8.08E+07    9.82E-03    6.80E-05    6.30E-05    3.53E-05    4.37E-03    6.25E-03
     0    0    0.77087
0
0    0    0.04    0.04
0    0    0    0
3435.25    -3435.25    0    0    1.5    0
248.75    -248.75  192.045 -192.045

11   BEAM "220x150x14.2 SHS Column"
1    4    1    1    0    2    0    0    0    0
2.10E+08    8.08E+07    9.82E-03    6.80E-05    6.30E-05    3.53E-05    4.37E-03    6.25E-03
     0    0    0.77087
0
0    0    0.04    0.04
0    0    0    0
3435.25    -3435.25    0    0    1.5    0
248.75    -248.75  192.045 -192.045

WEIGHTS   0    ! ‘S.Dead + 0.3 Live’ has been added to the weight of the slabs
1    0    0    0    0    0    0
1700    0    0    0    0    0    0

LOADS
1    0    0    0    0    0    0
1700    0    0    0    0    0    0

EQUAKE
6    4    0.005    1    -1
```