

Collapse Behaviour of Prefabricated Modular Buildings Under Seismic Conditions

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Declaration of Originality

This is to certify that

- i. The thesis comprises only my original work towards the Doctor of Philosophy
- ii. Due acknowledgement has been made in the text to all other material used
- iii. The thesis is less than 100,000 words in length, exclusive of tables, figure captions, bibliographies, and appendices.

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Abstract

Modular construction is becoming increasingly popular day by day due to its' advantages, that outweigh disadvantages to a great extent. With the continuous growth of modular construction, modular structures are moving more towards pure modular constructions which require minimal work onsite. Pure modular buildings require no external lateral force resisting systems (LFRS) which may require significant time and be more costly. Modular buildings are very different from conventional structures in terms of construction nature. A modular structure can be considered as an alternate arrangement of modules and connections. Hence, a modular building is discrete in terms of stiffness and strength distribution along with the building height, while that of a conventional building tend to be continuous. However, regardless of the apparent discrepancy between modular and conventional constructions in many aspects, modular structures are designed according to conventional building standards which fail to take inherent structural differences between modular and conventional structures into account.

Furthermore, most of the connections used in modular structures resemble bolted connections which are used in conventional constructions. Bolted connections cannot provide sufficient ductility to a structure due to the brittle nature of failure involved in them. As the relative strength of the module is usually stronger than the inter-modular connections, failure is expected to occur at the connections when the rest of the building may remain in the elastic state. Hence, if this most critical connection is a bolted connection or some other connections which are not with sufficient energy-dissipating capacity, under a seismic event exceeding the design limit, the structure will result in a brittle failure initiated by the failure of connections. Moreover, since the connections form a single storey with high inelasticity concentration, failure of one connection may trigger the failure of other connections of the same storey in an unzipping manner across a storey. This may result in a plunge in the damage variation of the structure, resulting in adverse effects on the serviceability of the structure. Furthermore, an unzipping of connections across one storey may leave a colossal mass of structural component comprising a stack of modules to stand freely and overturn to collapse under a further increase in ground motion intensity.

Pure modular structures are not fully realised in the current practice. However, since the current practice in modular construction is to rely on the conventional building design standards, techniques and practices, transformation to fully modular is not hindered from any of them.

There are no rules or regulations in the current practise requiring external lateral force resisting systems (LFRS) for structures built in low to moderate seismic regions. Hence, if the structure without any external LFRS can still satisfy the code requirements, the structure can exist. Given the enormous time and cost savings from eliminating any external LFRS, designers might soon opt to move towards pure modular structures while still designing the structure as per conventional design standards. Some researchers have already investigated this transition. However, the risk involved in this transition, as discussed in the previous paragraph needs to be studied to come up with additional design considerations in designing pure modular structures rather than merely adhering to conventional code requirements. Even though the highlighted risk arises under higher seismic intensities (beyond design limits) which are a result of destructive intraplate earthquakes that are considered infrequent, the location of this type of future intraplate earthquakes cannot be predicted. When pure modular buildings become famous, they will start replacing the conventional structure throughout the world. Thus, the chance of such a building getting severely affected by an earthquake increase.

The main aim of this study is to highlight the aforementioned potential risk involved in pure modular buildings that are built using conventionally available bolted connections when they undergo dynamic events exceeding their design intensity. A mid-rise (ten storeys) pure modular structure designed as per Australian design standards for Melbourne conditions was chosen as the prototype building to be considered in this study. Since the study aims at highlighting the risk involved in this construction nature when they are designed as per conventional building design standards without giving any special consideration, the study was conducted in comparison with a conventional steel frame analogical in physical dimensions to the chosen modular building. The study involved developing high-fidelity finite element (FE) models of the modular and conventional structures to study their feasibility and adherence to code requirements. Endurance time excitation functions (ETEF) were employed to study the seismic performance of the structure, especially under increasing seismic intensities. These ETEFs were developed as an alternative to conventional incremental dynamic analysis methods that are followed to study the seismic response of structures, more specifically, the collapse behaviour. The study is aimed at modelling the ultimate performance behaviour of the inter-modular connections between adjacent storeys as well as the near-collapse response behaviour of the building as a whole using numerical simulations along with shaking table experiments conducted on the shaker table for validation purposes. The simulations were scoped at investigating the behaviour of the building in a damaged state up to the onset of the wholesale

collapse. Based on the collapse response observed in the FE models and during the experimental analysis on the shaking table, further numerical and experimental models were developed to simulate and study in-depth the collapse response of the modular structure.

The study initiated with an investigation of the feasibility of the considered prototype modular and conventional steel buildings in terms of code requirements. Modal analysis, static pushover analysis and nonlinear dynamic analysis within the code specified limits proved that both the structures performed satisfactorily within the design limits. However, the modular structure was found to be failing catastrophically when the design limit exceeded, while the conventional steel frame continued to fail progressively. The discrete nature of the construction was highlighted during the analysis of prototype models, where the connections started giving up while the modules continued to remain in undamaged condition.

Analysis of experimental and numerical models, together with the observations from the prototype FE models, revealed that the failure in the modular structure initiated from the failure of edge connections of the first storey. Failure of the edge connections resulted in a sudden increase in the stresses of the internal connections and a plunge in the global damage of the structure. The sudden increase in connection forces led to a failure of the internal connections of the first storey, resulting in a failure of all the connections of that storey in an unzipping manner. This resulted in the stack of modules above the first storey to be left as a free-standing block which experienced sliding, rocking, and overturning under the rest of the ground motion.

The free-standing block thus formed (referred to as control model henceforth), analysed separately on the shaking table under the same ground motion stretch corresponding to the free-standing motion of the modular building superstructure, resulted in a pure rocking of the control model, without overturning. This observation was related to the absence of energy inputs (from the failure of connections just before the starting of free-standing phase), which results in initial conditions for the free-standing motion of the superstructure component, in the control model which was analysed on the shaking table. A numerical model of the control model, validated using the experimental results from the shaking table analysis was employed to verify the conclusion made on the cause of overturning with further analysis on it. Further, to support the conclusion, the ground displacement was checked against the half-width of the control model. The ground movement was way lesser than the half-width of the control model, supporting the fact that the energy released from the failure of the connections just before the free-standing motion was the sole reason for the overturning. Moreover, the validated numerical model of

the control model was employed to understand and elaborate on factors exacerbating and mitigating overturning of the modular building superstructure.

In contrast to the catastrophic failure that was observed in the modular structure, the conventional structure which comprised of a ductile braced frame demonstrated a failure initiation and progression through the bracing members which are not a primary component of the structure performance. Even though the prototype modular structure contained braces inside the modules, the lateral loadings that were resisted by the braces were finally transferred to the inter-modular connections. Moreover, since the braces (internal braces inside the modules) from upper and lower modules do not intersect at a single working point of the connection (due to the gap between the modules), the connections experience an unbalanced loading from the lateral loads transferred from upper and lower modules. This does not happen in the case of a conventional structure due to the continuous nature in the structure. Hence, in the conventional steel frame, the braces which have a higher ductile capacity gave up in a sequence, starting from lower storeys, moving up along the structure gradually. This sort of a distributed nature of the failure involving the failure of ductile components resulted in progressive failure of the conventional steel frame even beyond design limits.

The hazardous nature of the collapse of pure modular structures, as highlighted in this study, is a critical study that has been lacking in the modular building research area. The need for this study arises explicitly due to the current practices followed in modular construction, which involves the use of conventional bolted connections as the inter-modular connections, and the use of conventional building design codes to design modular structures. As demonstrated through this study, the conventional design codes are capable of ensuring a safer performance of the conventional structures even beyond the design limits. In comparison, adherence to the same did not guarantee a safer performance of the modular structure beyond the design limits.

Furthermore, this study brought out the unique nature of the failure initiation, progression and collapse involved in modular structures due to the discrete nature of its' construction. This component of modular structures' response has been missing in the research involved in modular structures, specifically under dynamic events. The collapse process, as studied in-depth in this research, can be utilised to improve the safety of the response of modular structures under dynamic loadings and come up with a safer pure modular system. Based on the findings from this study on the collapse response of conventional and modular structures, recommendations are presented at the end of this study for future researchers and designers to

follow in designing pure modular structures. These recommendations are made by critically analysing and comparing the performance of the modular and conventional structures and by addressing what has been missing in the response of modular structures that makes its response catastrophic.

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List of Abbreviations

ADRS	Acceleration displacement response spectrum
AEM	Applied element method
CAM	Component attenuation model
CAV	Cumulative absolute velocity
COG	Centre of gravity
CP	Collapse prevention
DEM	Discrete element method
DM	Damage measure
DOF	Degrees of freedom
ET	Endurance Time
ETEF	Endurance time excitation function
ETHA	Endurance time history analysis
FE	Finite element
FEM	Finite element method
IDA	Incremental dynamic analysis
IM	Intensity measure
IO	Immediate occupancy
LFRS	Lateral force-resisting system
LS	Life safety
MEP	Mechanical, electrical, plumbing
MMI	Modified Mercalli Intensity
MSB	Modular steel building
OSB	Oriented-strand board
PGA	Peak ground acceleration
RSA	Response spectral acceleration
SHS	Square hollow sections
THA	Time history analysis
UK	United Kingdom

Chapter 1: Introduction

1.1 Background

Modular building construction has gained popularity over the past two decades because of its high level of efficiency, particularly in terms of labour, materials, and costs [1]. Initially, the process involved prefabrication of single components, such as slab elements or any other small component of a building. It then gradually moved to prefabrication of volumetric cubicle components manufactured in controlled factory environments. These volumetric components have been gradually made to include all the equipment and fittings required for the location in the building to which they belonged. For example, kitchen modules are manufactured with everything that needs to be incorporated into a kitchen, from tiling, plumbing and electricity to kitchen equipment [1-3].

The concept of modular buildings originated from a combination of the concepts of prefabrication and shipping containers. Shipping containers have long been used as storage and temporary accommodation facilities (see Figure 1-1) [4]. When the components of a building, such as concrete slabs and ceilings, were first prefabricated, they were used together with shipping container type structures to form modules. Gradually, they came into being as modular buildings through their implementation on construction sites as temporary offices, storage facilities and emergency accommodation [5, 6].



Figure 1-1 A shipping container house <https://menafn.com/>, last accessed August 2020

The speed and ease of construction, and excellent quality of manufacturing possible in controlled factory environments have shown this type of construction to be a perfect solution to the increasing demand for construction, especially with short timeframes [5]. Modular construction has been widely adopted in the residential and commercial building sectors and

for emergency accommodation. The most recent and impressive use of prefabricated modules was in the construction of a hospital in Wuhan during the COVID-19 outbreak (Figure 1-2). This two-storey hospital contained space for 1,000 hospital beds and was fully completed over a 10-day period from 23 January to 2 February 2020. The prefabricated modules were laid on pillars to keep them off the ground [7].



Figure 1-2 Wuhan, China, hospital construction, which was completed in 10 days

<https://www.wsj.com/>, <https://www.aljazeera.com/>, last accessed August 2020

Many countries have changed their attitude towards modularisation since the concept of modular building was developed over the last two decades [8]. Countries like the United States of America (USA), United Kingdom (UK) and China have taken great initiative in moving towards modular construction for residential and commercial buildings; more so than has Australia [8-10]. Australia has begun to adopt modular construction in recent years, as many buildings were built throughout the country that, at least to some extent, adopted the modular construction [11, 12]. The world's tallest modular construction as of June 2020 was in Singapore (Figure 1-3). Clement Canopy towers, at 140 m tall, consists of 1,899 prefabricated panels of which approximately 85% were constructed off-site prior to onsite assembly [13].



Figure 1-3 Clement Canopy towers, Singapore-the world's tallest modular building

<https://www.architectureanddesign.com.au/>, last accessed in August 2020

In addition to Clement Canopy towers in Singapore, many buildings around the world have been built using modular construction. Modular projects from around Australia include the Little Hero apartment building in Melbourne (Figure 1-4), which was built up to eight stories tall using 58 prefabricated modules; and SOHO apartments Darwin (Figure 1-5), a 32-storey building [12].



Figure 1-4 Little Hero building, Melbourne, Australia-8 storeys, 58 modules [12]



Figure 1-5 SOHO apartment building, Darwin, Australia [12]

One9 apartments (Figure 1-6) is a nine-storey modular building built in Melbourne and consisting of apartments designed for energy-efficient and comfortable liveability. The superstructure of the building was constructed in five days. It was reported that the construction site had many constraints for construction because of the presence of a large shopping mall nearby. Modular construction was advantageous in this scenario, as it allowed construction of the building with little disturbance to the surroundings, and completion of the project in a short timeframe [12].



Figure 1-6 One9 apartments, Melbourne, Australia <https://www.hickory.com.au>, last accessed August 2020

With continuous improvements stemming from research and development, modular buildings keep growing taller and are moving more towards becoming fully modular. Fully modular buildings are built without the requirement for conventional lateral or gravity load-transferring systems such as core walls, shear walls and mega braces, the construction of which is time consuming and costly [11, 12, 14]. A fully modular building would ideally consist of an arrangement of modules and connections only. Modules in such buildings would have a lateral

force-resisting system (LFRS) incorporated into them at the factory itself [12]. Modules would be transported to the site and directly connected using connections to form buildings. Without the aid of an external LFRS, the most vulnerable failure point in a modular building are the inter-modular connections [11].

Individual modules usually have greater strength than the inter-modular connections. These connections disturb the continuous nature of stiffness and strength along the building height. Typical module-to-module connections are either welded or bolted splice connections [14, 15]. Many researchers have come up with different types of connections that could be used to connect modules in modular construction. An extensive review of most connections used in current practice is presented in Section 2.1.7.3.

1.2 Research Problem

Modular buildings are different from conventional structures in terms of behaviour, detailing requirements, and method of construction. Regardless of the many differences, both types of building are designed according to prevailing building standards, which fail to take into account inherent structural differences between modular and conventional structures. Without any supplementary LFRS, in a pure modular system, the most vulnerable failure point will be the inter-modular connections. This is the opposite of the case for a conventional building where we will identify the most vulnerable failure elements opposed to the concept of most vulnerable failure points in a pure modular building. In a conventional building, failure will begin with and progress from lateral force-resisting elements such as braces or shear walls, or other primary elements of the building such as columns or beams. Failure in these elements in a conventional building will be more evenly distributed rather than being concentrated at a particular point in the building, as in the case of modular building connections.

Separate lateral force resisting elements such as RC structural shear walls, cross-braced (or k-braced) steel towers, or simply slender cross-ties have been designed and built to provide lateral stability to a building which is built of stacks of modular building units (MBUs). Practising structural engineers are used to adopting the approach of ignoring the gravity frame system whilst counting on such lateral force resisting elements when analysing the building for lateral stability. MBU stacks built of bolt connected steel units can be considered as a “gravity system” but those stacks can be many times stiffer than most lateral force resisting elements. The incompatibility issue is normally not taken into account in most codified structural design

procedures (which only requires linear elastic analyses of the building to be performed) and more so if the design of the lateral force resisting elements has incorporated a ductility reduction factor which would further exacerbated the issue. In view of the incompatibility issue which may render lateral force resisting elements to be totally ineffective this thesis deals with testing and analysing the stability of the MBU stacks in isolation when responding to seismic conditions.

All structures can be designed to adhere to code requirements. Thus, conventional, and purely modular buildings can both be designed to meet design conditions, and in the event of ground motion within the designed seismic intensity, both buildings will survive with no failure. Nevertheless, if a rare seismic event exceeding the design limit occurs, a modular building might behave completely differently from a conventional building. Such a distinct response is expected due to the construction nature, which puts the inter-module connections more vulnerable to failure. In this scenario, the discontinuous nature of the building due to the alternate arrangement of connections and modules may place the comparatively less strong connections (Compared to modules) more vulnerable to failure. This failure may result in a discontinuity in the lateral and gravity load-flowing path between modules.

The conventional welded or bolted spliced connections that are currently used do not cater to the unique structural characteristics of modular structures. Welded connections are not preferred for pure modular construction because of the amount of work involved on-site and the reduced access possible during welding. Bolted connections behave in a brittle manner once an initial slip occurs and they cannot be made ductile, although they can be made very strong. A good connection requires a sufficient balance of ductility and strength to protect the structure from loadings beyond design conditions. In addition, most inter-modular connections provide insufficient and limited energy-dissipating capacity to the structure.

For a conventional structure, a seismic event exceeding the design limit will not cause the hazard expected for a pure modular building. Conventional buildings undergo failure of many components, such as braces, that cannot be considered as critical as the inter-modular connections in a pure modular building. Even with increasing seismic intensities, a failure within a conventional structure will be more evenly distributed throughout the building; conversely, that in a modular building will be concentrated at the relatively weak connections. Failure of one connection may amplify the concentration of inelasticity in other connections of the same storey or other storeys in the building, making the other connections more vulnerable

to failure. This sort of failure progression may result in sudden progression of damage in a modular building, creating more hazardous modes of collapse than for a conventional building. In conventional structures, a failure might occur more progressively, element by element, distributing the damage more evenly throughout the building.

Further, since the connections are more prone to failure than are the modules, if the seismic intensity increases beyond a certain limit, the collapse in the structure will initiate and propagate in a distinct manner. The vulnerable connections will begin to fail, allowing single modules or blocks containing many modules to collapse catastrophically. Analysis of the collapse of this nature may include the rigid body motion of free-standing blocks of modules or free-falling single modules. This sort of potential destructive collapse behaviour in modular construction has not been studied because modular construction is such a new form of construction in terms of research and industry practices. Most studies to date on the dynamic behaviour of modular buildings have been limited to analysis within design limits, where the structural response under anticipated design dynamic conditions is studied using a simple nonlinear time history analysis (THA).

1.3 Aims and Objectives

The main aim of this research is to highlight the potential risk that may be involved in modular buildings constructed without any external LFRS (pure modular systems) when they are subjected to seismic events exceeding the design limit. It is understood that because of the nature of their construction, modular buildings are entirely different from conventional buildings in terms of their structural configuration and, hence, behaviour. The following objectives are established in this regard:

- 1) Develop high-fidelity finite elements models of a prototype modular and conventional steel building. Each of these models was identical to each other in terms of the gross dimensions and distributions of mass, and was designed to be code compliant for a return period of 500 years. However, these models were to be subject to the intensity of ground shaking surpassing the code design limits.
- 2) Design and build scaled-down experimental models of the modular building, a control model representing a free-standing stack of modules, and a second control model representing a typical conventional steel building for dynamic experimentation on a shaking table.

- 3) Validate numerical models of the scaled-down modular building and the conventional steel building to ensure that the simulated static and dynamic response behaviour were consistent with that observed in the physical experiments.
- 4) Conduct parametric studies using the validated numerical models to study the behaviour of the modular building when experiencing large displacement on the verge of collapse and to study factors that exacerbate or mitigate the ultimate behaviour of the building.
- 5) Derive recommendations (based on experimental and numerical results of the study) to develop safer pure modular building structures.

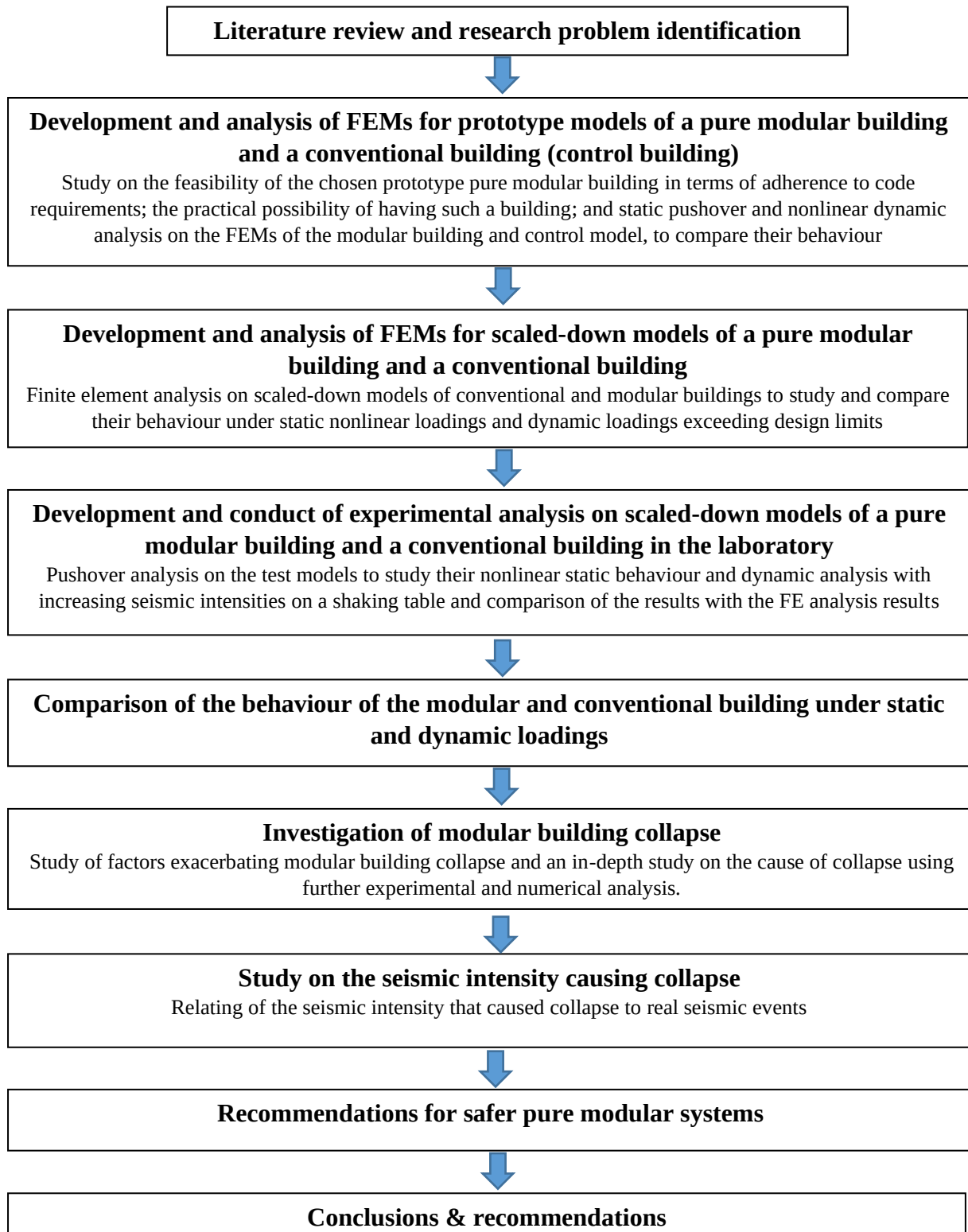
1.4 Research Scope and Methodology

1.4.1 Research Scope

This study focusses on a modular building and a conventional building of the same dimensions and consisting of two bays and 10 storeys. The buildings considered are assumed to be in Melbourne, Australia; to stand on site class C soil; and to be designed to withstand a 500-year-hazard return-period earthquake. The structures are studied and compared for their response behaviour, moving from rest through the linear elastic phase to the nonlinear elastic phase and collapse.

1.4.2 Research Methodology

The methodology followed in this research is summarised below.



1.5 Thesis Layout

The thesis consists of eight chapters. A brief introduction to these eight chapters is presented below.

Chapter 1: Introduction and Structure of the Thesis. This chapter provides a brief introduction to modular construction and discusses its development over time. The chapter also introduces examples of modular constructions across the world and their significance in terms of benefits achieved from their being modular. The chapter also presents the aims, objectives, scope, and methodology of this research.

Chapter 2: Literature Review. This chapter provides a detailed report on the literature review carried out during this study. The chapter begins with a literature review of prefabricated modular buildings. The history of modular buildings, which starts with prefabrication, is explained in detail. A full review of all components of a modular building (mainly the modules and the connections employed in modular buildings) is also included in this chapter.

Moreover, the chapter includes the advantages of modular construction and its challenges in terms of achieving full modularity (pure modularity). Since the study aim is related to the seismic performance of modular buildings, this study also includes a comprehensive literature review of studies conducted to date on the seismic performance of modular buildings. The literature related to other essential theories and techniques used throughout this study is also reviewed in this chapter.

Chapter 3: Development and Analysis of Prototype Models. This chapter begins with an investigation into the possibility of having a small to mid-rise pure modular building without the requirement for any external LFRS. Accordingly, prototype modular and conventional steel buildings are developed in the FE packages OpenSees and SAP2000. The developed FE models are subjected to modal, static pushover and nonlinear dynamic analyses to study and compare their behaviour. Results from this chapter are used as a basis for making judgements on the feasibility of a pure modular structure. The comparative responses to dynamic events exceeding design limits observed for the modular and conventional structures motivate the development of scaled-down models to conduct the experimental analyses in Chapter 4 and 5.

Chapter 4: Experimental Tests on Scaled-down Models. This chapter presents an experimental analysis and comparison of the response of the modular and conventional

structures introduced in Chapter 3. The chapter explains in detail the scaling laws employed, modelling assumptions made and development of the experimental models for both the modular and conventional buildings. Further, shaking table experiments on a free-standing stack of modules formed by the failure of the connections, to study its response from starting from rest conditions under the same ground motion stretch is also presented. Finally, the experiments conducted on both models (conventional steel frame and modular building) (static and dynamic tests on the shaking table) are presented together with the results.

Chapter 5: Collapse Analysis and Comparison with Recordings from Physical Testings.

In this chapter, details are presented on the development of FE models of the scaled-down experimental models of modular and conventional steel buildings. All analyses are repeated on the developed FE models to validate them. Validated FE models are employed to conduct further analysis of both models to understand and compare each stage of the structure response (failure initiation, propagation, and collapse). Further, the results for scaled-down models are compared with those for the prototype models to verify the effectiveness of the modelling assumptions made during the development of the scaled-down models.

Chapter 6: Collapse Simulation of The Modular Building. In this chapter, an in-depth study on the response of the modular structure during the rock, slide and overturning phase is studied. Further numerical models developed in MATLAB software are employed to model the rock, slide and overturning behaviour observed. Moreover, the developed numerical model is validated with the experimental observations from Chapter 4 to use the numerical model to study factors exacerbating and mitigating overturning.

Chapter 7: Recommendations for Safer Pure Modular Systems. In this chapter, recommendations are provided for future researchers and designers to consider in researching and designing pure modular structures. The recommendations are aimed at avoiding the hazardous nature recorded during the experimental and numerical studies discussed in earlier chapters. Further, a novel modular configuration that can resist lateral loadings through bundle tube action and avoid most of the catastrophic failure modes observed during the experiments, is introduced along with an analysis of the conceptual feasibility of the introduced system.

Chapter 8: Conclusions and Recommendations. This chapter presents conclusions from the full study and makes recommendations for future research. Further, the limitations of this study are also discussed.

Chapter 2: Literature Review

2.1 Prefabricated Modular Buildings

2.1.1 Prefabrication

Prefabrication can be considered a manufacturing process that involves a specialised facility at which different components are assembled to form a final installation [16]. Prefabrication started out for individual components of buildings such as slabs, walls, and panels. It gradually expanded to prefabricate these components together in volumetric units. It can be done off-site (in a factory environment) or on-site [3]. The ‘prefab’ category includes many subcategories of construction grouped under its umbrella. Each subcategory represents a different type of prefabrication of building components at off-site locations that are ready to be fixed into their allocated position at the site. Panelisation and modularisation are two common subcategories of prefabrication [17].

Panelisation involves off-site manufacturing of individual components required for a structure, such as prefabricated wall panels, precast floor, and roof systems. These are transported to the construction site to be assembled to form a finished structure. Typically, prefabricated panels are constructed with the structural requirement of the component completed off-site, and their finishing left to be done at the site after assembly. In this method of construction, wall finishes, plumbing fixtures, electric and other infrastructure are all done at the site premises rather than at the manufacturing plant. Unlike modularised construction as discussed below, panelised construction requires a significant amount of onsite work. Nonetheless, panelisation offers significant cost savings in terms of labour, cost and time compared with conventional construction without any form of prefabrication [1, 9, 17].

Modularisation is when volumetric components that form a significant part of the building are manufactured in off-site factory environments, rather than the prefabrication of only individual parts as in the panelised method. In terms of onsite construction time, modular units have lower costs than any other construction type. This is because of the nature of manufacturing for the finished building in terms of mechanical, electrical, plumbing (MEP) and exterior finishing at the off-site premises. This means that tasks remaining to be done at the site are limited to assembling the modules to form the completed structure [3, 5, 17].

2.1.2 Modular Buildings

Concepts of off-site manufactured housing have been evolving since the late 1920s and 1930s; through architects such as Peter Behrens and Walter Gropius in Germany and Richard Neutra and Buckminster Fuller in the USA. Motor cars are manufactured in large numbers using an assembly line approach. Initially, this approach was adopted to mass manufacture prefabricated components (panelised method), but it has since developed into mass manufacturing of prefabricated building modules [1]. In the USA, the modular industry came into being with the caravan industry; modularisation was widely used during the Second World War to construct caravans used as accommodation for troops. The demand for caravans as permanent housing increased after the war [1]. The construction industry catered for this demand by manufacturing ‘mobile homes’ that endeavoured to maintain a balance between utility as a vehicle and a house [1].

Modular buildings are one of the most outstanding achievements of the prefabricated construction industry, where volumetric structural units are manufactured in a highly controlled factory environment and shipped to the site ready to be fitted into their positions [10]. A typical module is almost finished when it is shipped out of the manufacturing facility. The type of module manufactured differs according to its intended location in the modular building [18]. For example, bathroom modules are built with all fittings, including a shower, washbasin and toilet at the factory premises itself [19]. Finally, the task required on-site is to assemble the modules in their allocated positions. These volumetric components are usually manufactured using concrete, wood and steel [8, 9]. Despite the practical and technical difficulties involved in the usage of each material, all these materials have been used during the past few years [1]. However, steel is currently the most commonly used material in the manufacture of modules because of its many advantages, including its low weight and high strength [2]. This study is focussed on steel modular building construction.

Apart from the motivation from the increased usage of prefabrication, the history of modular buildings can also be attached to shipping containers and caravans. Caravans, as a mode of temporary housing, have a history dating back more than 100 years. This form of portable housing has been very popular as temporary recreational houses. Caravans are very similar to modular buildings except that they are not stacked or joined to form large spaces; nor are they made for permanent housing. A caravan is analogous to a single module (Figure 2-1) [4-6, 17, 20].



Figure 2-1 A caravan used as temporary accommodation

<https://www.timetoroam.com.au/>, last accessed September 2020

Shipping containers have been used as temporary accommodation and storage spaces for a very long time. They are still widely used in many countries because of their proven advantages for this sort of arrangement. Modular construction is more related to shipping containers than to caravans, because of their long-term usage and wider range of applications [4, 17]. Shipping containers have been used as storage facilities, where many container boxes are stacked to form a larger storage space. They have also been used with their sides open and combined to form larger spaces for accommodations, office areas and general use. Prefabrication technology, combined with the motivation from pre-existing shipping container houses, has provided a strong foundation for the evolution of modular buildings [17].



Figure 2-2 Shipping containers used for residential and commercial purposes

<https://africahousingnews.com/>, last accessed September 2020

2.1.3 Classification of Modules

Several classification systems have been introduced by researchers to classify modules available in current practice. A generalised module classification system based on available

classifications and their popularity is presented in this study. Although a material-based classification is possible, because of the wide use of steel, this study focusses on steel modules [3].

2.1.3.1 Non-structural Modules

Non-structural modules are used in buildings when the modules make no structural contribution, such as gravity or lateral load transfer. Since the modules do not take part in such structural behaviour, buildings constructed with this type of module require external support from structural systems such as braces or core/shear walls to resist lateral and gravity loadings [1, 2, 18].

Modules supported by a primary structural frame are an example of non-structural modules. With this approach, the supporting frame is made of steel or concrete. As in conventional buildings, the supporting frame is the most critical component providing lateral and gravity load resistance [1, 2]. Compared with traditional structures, this type of construction saves time because less work is required on site. However, the approach is still inefficient compared with the ideal fully modular system desired by researchers because the structural frame needs to be made prior to the assembly of modules. This system can be advantageous because of the reduced strength and stiffness requirements imparted to modules from not requiring resisting lateral and gravity loads. The seven-storey MOHO Manchester apartments in the UK is an example of a building constructed on a steel frame (Figure 2-3) [2].



Figure 2-3 MOHO Manchester, UK apartments [2]

There are also cases where concrete frames have been equipped with prefabricated modules to save time and costs [2]. Figure 2-4 shows the installation of bathroom pods in a concrete-framed building. Bathroom pods manufactured separately are another type of prefabrication

that has been occurring for a long time. In addition to bathroom and kitchen pods, many other different types of area-specific prefabricated volumetric components manufacturing has evolved to make construction work faster and more cost-effective [1, 2].



Figure 2-4 Installing bathroom pods in a concrete framed building [2]

2.1.3.2 Structural Modules

Unlike non-structural modules, structural modules in a building take part in the structural performance of the building. They may have the capability to transfer gravity loadings, lateral loadings, or both. There are two main types of structural modules based on their gravity load-transferring mechanism [1, 2].

2.1.3.2.1 Load-bearing Modules

These types of modules are also known as wall load-bearing modules. As the name implies, they have their full perimeter to transfer gravitational loading. In buildings equipped with these modules, the walls and columns of the module carry the gravity load, which is finally transferred to the foundations. Since the walls of the module participate in load transfer, they are usually designed with ‘C’ sections placed at constant intervals along the wall length. Diaphragm action of the panel or board material used on walls, together or by themselves, is adapted to transfer the lateral loads acting on the modules [1, 2, 19].

Steel sandwich panels are also commonly employed in load-bearing walls. The calculated gravitational loading and estimated lateral loading imparted on the wall section are taken into account in calculating the strength of the wall panels, and spacing of the profiles used [2, 19]. Figure 2-5 shows a typical load-bearing module used in current practice.



Figure 2-5 A typical load-bearing module [1]

2.1.3.2.2 Corner-supported Modules

In these modules, the corner and intermediate columns carry the gravity loads transmitted by the edge beams of the modules. Lateral load-transferring systems in these types of modules are similar to load-bearing modules, where bracings or the diaphragm action of wall and floor elements can be utilised as required. Corner columns are usually made up of square hollow sections (SHS) [1, 2, 21]. Figure 2-6 shows a typical corner-supported module used in the industry.



Figure 2-6 A corner-supported module [2]

A critical performance aspect of these alternately arranged connections and modules is their behaviour under lateral loads. Based on the relative stiffness of the ceiling beam, the floor and their connections to the corner columns, the behaviour of the column at the column-to-column connection may differ. However, this kind of behaviour also depends on the lateral load-transferring system used in the building [11, 14, 22].

Load-bearing modules must be stacked on top of each other with no discontinuity around the perimeter. However, corner-supported modules are stacked with only corners and intermediate columns connected, leaving a gap between modules. This gap can be utilised as a space for Mechanical, plumbing, or electrical (MEP) installation. Corner-supported modules are more commonly used than load-bearing modules because not requiring strong wall elements makes them cheaper; they are lightweight; and the gap between them is useful for MEP purposes [1, 2, 11, 12, 18, 23].

2.1.4 Features of a Typical Module

This section explores the composition of a generally used module available in the industry.

2.1.4.1 General Skeleton

A typical module manufactured using current construction practices is 3–4 m wide, 3–4 m high and 6–16 m long, according to its intended use. Transportation regulations and restrictions impose constraints on module width. Modules are commonly made up of SHS corner posts and Parallel flange channel (PFC) sections as primary and secondary beams for ceiling and roof [1, 2, 24]. The composition of an example (corner-supported) module is shown in Figure 2-7.

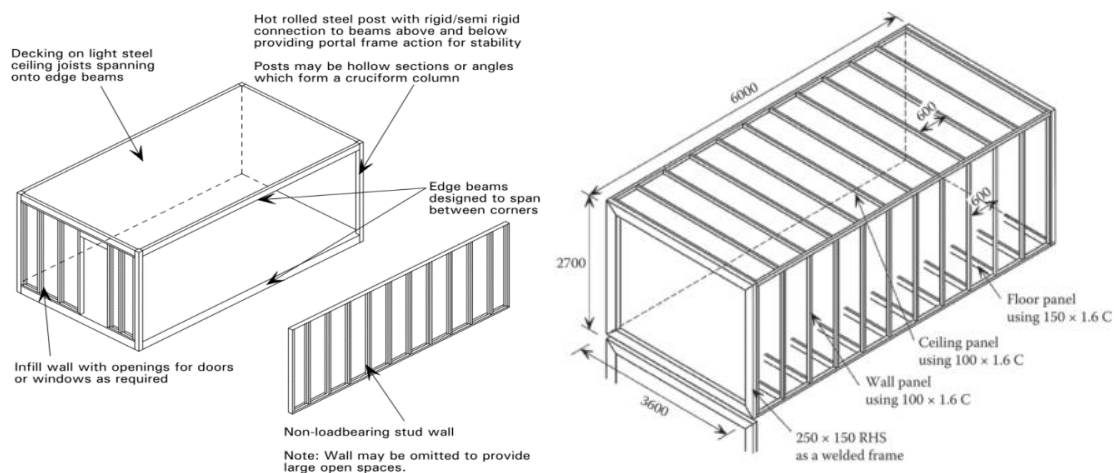


Figure 2-7 Composition of a sample module [2]

The dimensions of the sections that make up the skeleton of the modules vary based on the building design being considered. Section sizes are usually chosen based on the gravitational and imposed loads on the structure, as per recommendations from design codes.

2.1.4.2 Floor

Floors of modules carry the dead and live loads imposed directly on them. A module floor usually consists of steel C sections 150–200 mm deep and 1.2–1.5 mm thick, spaced around 400 mm apart (varies depending on the calculated loading). The spacing for the C sections is also selected to be compatible with the floorboards that are used. The top flange of beams is attached to the floorboards to prevent local buckling. In contrast to conventional design considerations, the design of floor beam C sections is mostly governed by the perceptible vibration or deflection rather than bending resistance [2]. An example module floor section is shown in Figure 2-8.

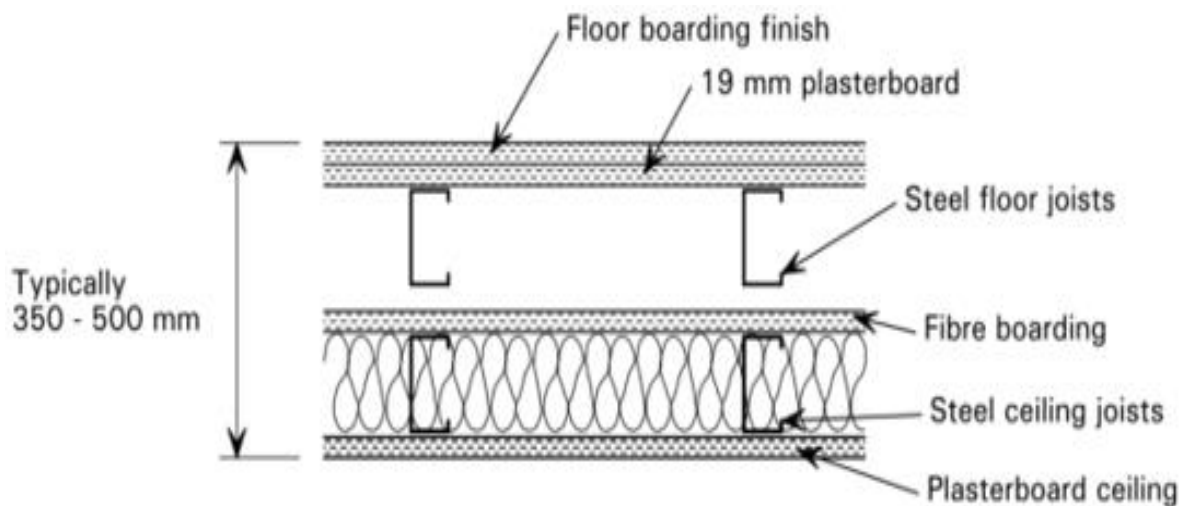


Figure 2-8 Typical floor section of a module [2]

Apart from floorboards, precast concrete slabs are also used as flooring and ceilings inside modules [12].

2.1.4.3 Walls

Outer load-bearing walls are usually made up of concrete infill panels connected by a gusset plate and an anchor in the infill wall [25], as shown in Figure 2-9.

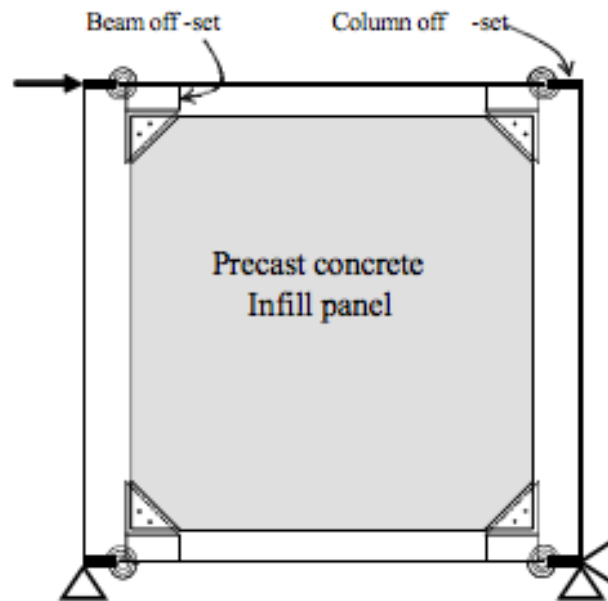


Figure 2-9 Precast concrete infill panel connected to the frame [25]

Other internal walls that do not adequately take part in the gravitational load-bearing process are usually made up of plasterboards or oriented-strand boards (OSBs). Exterior walls are covered with OSBs or cement particle boards, which are involved in weatherproofing and some diaphragm action in the wall. These boards are usually fixed to the walls using air-driven pins strengthened by glued joints. Internal walls are of double skin construction, which provides excellent acoustic insulation. Use of these boards in the walls is also effective for intruding vertical diaphragm action in each module so that it will act as a lateral load-resisting system [1, 2].

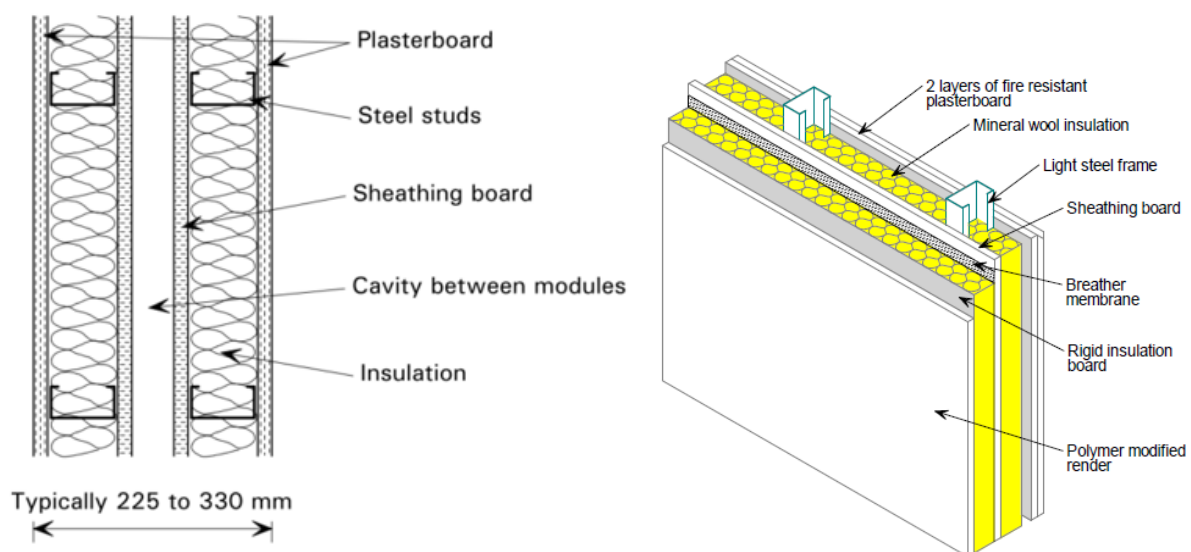


Figure 2-10 Internal wall cross-section [1]

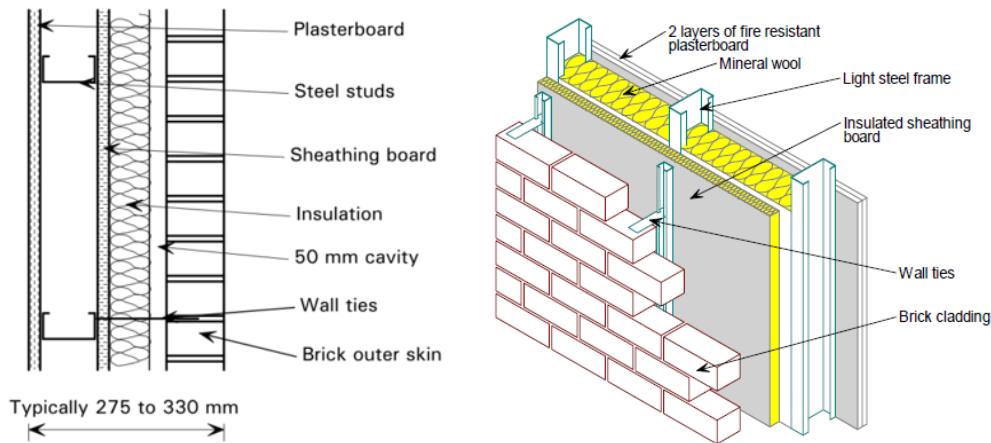


Figure 2-11 External wall cross-section and side view [1]

2.1.5 Advantages of Modular Construction

The advantages of modular building construction very much outweigh the disadvantages. This has been the motivator for the growth in demand for modular buildings and research involved in this sector. Some of the primary benefits involved in modular building construction are as follows:

- Economies of scale due to manufacturing multiple identical units. Modules are usually manufactured in large numbers at once to cater for different customers. This large-scale manufacturing has the advantage of economies of scale [5, 26].
- The speed of construction on site. Since the modules are prefabricated and the only on-site task is assembly, this brings time saving on-site. Moreover, assembly is made easier with efficient connection technologies [8, 27]
- Improved quality and accuracy in manufacturing and installation. Since the modules are manufactured in a controlled factory environment, more supervision and technology can be employed to improve the quality of the manufacturing process. Installation of modules also requires less effort and modules can be manufactured to simplify the process to make it more accurate [1].
- Less environmental impact during construction. This is one of the most important advantages of modular building construction. Since the construction work on site is reduced by more than 80%, pollution from construction is significantly lower compared to conventional construction. This has benefits for projects in terms of sustainability [2].

- Detachable/relocatable. Modular building manufacturing nowadays aims to provide flexibility for customers to detach their modules and reinstall them elsewhere [28].
- Safety. Compared with conventional construction, modular building construction provides a much safer working environment because it requires specially trained labour at sites and factories, and less work is involved on-site [2].
- Design flexibility. Modules of different sizes and with different sides open can be used to achieve more architectural flexibility [2].

2.1.6 Challenges Involved in, and the Future of, Modular Building Construction

Several challenges need to be overcome to achieve the desired benefits and achieve full modularity in modular construction. Overcoming the most expensive and time-consuming components of traditional construction is one challenge in modular building construction. Some of these challenges are as follows:

1. Necessity for a concrete core or external LFRS for mid- or high-rise modular buildings

The core or any other external LFRS is one of the most essential parts of any high-rise building. Construction of the core is very time consuming and costly. One goal of modular construction is to reduce both time and cost. Hence, it is vital to develop a way to replace the concrete core and other external LFRS with an alternative form of load-transfer mechanism that is cheaper, less time consuming, and more specifically, can be incorporated to the modules at the factory premises itself [11, 29].

2. Lack of availability of a generic connection system [30]

Modular buildings, unlike traditional buildings, do not have an internationally agreed code or standard design protocol, especially for the connections used. A generic global connection design would allow any two modules from two manufacturers to be assembled easily.

3. Size of modules and vibrations on modules during transport

Current module size is restricted to the maximum dimension of available transport trucks and transportation size limits imposed by governments. Modules are manufactured at factory premises with all required attachments and equipment in place. The attachments/connections inside the module as well as the module frame itself, including its components, may be damaged as a result of vibrations caused by the uneven texture of roads and impact from sudden stops and driving over speed humps.

These issues should be considered in the design of modules to prevent damage to any parts of modules [30-32].

4. Stability of modules and behaviour of inter-module connections during lateral loading

Earthquake and wind loads create critical loads for most structures. Modular buildings should have an efficient way of transferring and dissipating these lateral loadings. Inter-module connections should be capable of transferring these loads safely. Because of the discrete nature of such constructions, their behaviour under seismic events exceeding design limits may be catastrophic, and it is the objective of this research to explore this issue [11, 24, 33-36].

Pure modular buildings are those without any external LFRS such as concrete core wall, shear walls or mega braces. This sort of external LFRS is the most expensive component, and the most time-consuming component to build, in both conventionally built steel and modular buildings. Eliminating these partially or entirely will impart greater stresses to connections that would otherwise have been supported by a conventional LFRS. Most research developing connections for modular buildings aims to develop connections that can transfer gravity and lateral loadings effectively so that the requirement for external gravity and lateral load-transfer mechanisms can be gradually removed in the move towards a pure modular system. Lateral load-transfer systems in modular and conventional buildings, and the requirements in this regard for pure modular buildings, are discussed in detail in the latter part of this chapter.

2.1.7 Modular Building Connections

Modular building connections are the most critical component of a modular building. They have an enormous influence on the performance of the building under static and dynamic loadings [11]. Due to the nature of construction, these module-to-module connections play a vital role in the stability of these buildings. Hence, connections need to be designed appropriately to transfer loads safely from one module to another. These connections should have the capacity to carry static loads, which are common in all buildings, while also being able to withstand extreme wind and earthquake loadings, which are of dynamic nature [12].

Since the conception of prefabrication, developing an appropriate connection has been a major concern for researchers. Such a connection has the potential to considerably improve the efficiency of the building in terms of performance under static and dynamic loadings and loadings during assembly of modules. The efficiency of prefabrication increases more with less

work at the site. Research has focussed on developing a connection system that can eliminate the requirement for additional work on sites, such as the requirement for a concrete core wall, shear wall or mega braces. If not for this work at the site, modular construction would take times in the order of a few hours or few days. Three types of connection can be identified in modular buildings (Figure 2-12) [18]:

1. Inter-modular connections
2. Intra-modular connections
3. Module-to-foundation connections.

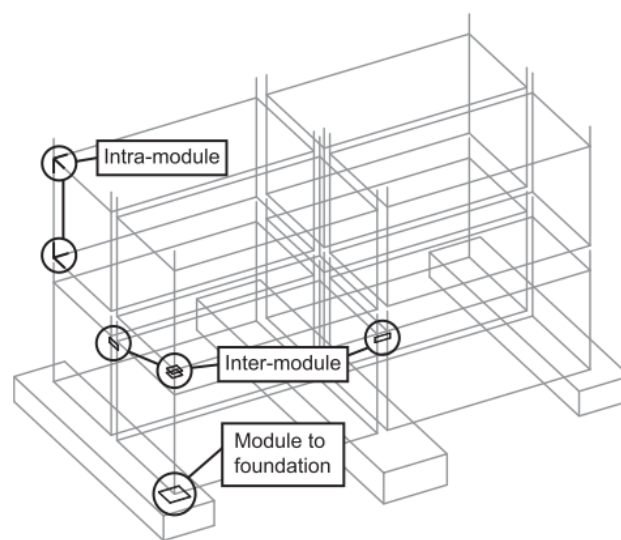


Figure 2-12 Connection types in a modular building [18]

2.1.7.1 Intra Modular Connections

Intra-modular connections are those within a module forming its skeleton. These are similar to connections used in conventional constructions. Depending on the location and relative significance of the connection area, either welding or bolted connections, is used. Single web (or fin) plates [2, 21, 26], double-angle cleats [37], and bolted endplates [21] are commonly used types of bolted connection for column-to-beam connection. Moment-resisting connections comprised of an endplate or a deep fin plate have been found to be sufficient to provide the necessary lateral stiffness in low-rise structures [2]. This is atypical, as fin plate connections usually fall into the category of simple shear connections; their use is restricted to low-rise buildings (maximum of three storeys) because of their lower ductility, rotation capacity and moment capacity [2]. Nonetheless, the risk of progressive collapse increases in

open modules when this type of connection is adopted [2]. When it comes to modular buildings, fin plates alone may not have adequate capacity, so, they may require strengthening.

Conversely, Annan et al. [24] researched the potential for welding secondary beams directly onto main beams to form steel floor frames. Analogously, traditional steel frames may use clip angles allowing higher rotation capability. Steel members should be designed to stand axial forces and hogging moments generated on them as a result of the fixed nature of welded connections [38]. Annan et al. reported that linear elastic analysis is appropriate for examining this issue and proposed a process to be adopted in design.

2.1.7.2 Connectivity Between Module and Foundation

Despite substantial change in the superstructure component of modular buildings, the foundations used in modular buildings remain almost identical to those used in conventional buildings [18]. Conventionally used foundations such as augured steel piles, in situ or precast concrete footings, bored piles or combinations of these may be used for modular buildings [18].

Adequate foundations are essential to protect low-rise modular buildings constructed in high-dynamic loading areas from sliding and overturning failures. Prefabricated modules are generally connected to the foundations by welding to concrete or steel piles, chains, keeper plates or cables [18]. All these methods have pros and cons, including tensioning requirements for cables and chains.

Foundations are a concerning issue, especially in high- and medium-rise structures. Modules can be manufactured with base plates included and welded on-site to accessible cast-in plates or fitted to cast-in anchors.

Figure 2-13 illustrates an embedded column connection system reported in Ref. [39] that replaces conventional post-fixed or cast-in steel-bearing plate connections. This proposed embedded column connection is aimed at delivering better ductility and utilising the complete column strength. However, a shortcoming is the necessity for welding on site between the columns of the modules and the endplates [18].

A discussion has been carried out in Chapter 7 as a recommendation to use partially restrained foundations that could allow uplift and rocking of the structure to protect the structure under extreme seismic events.

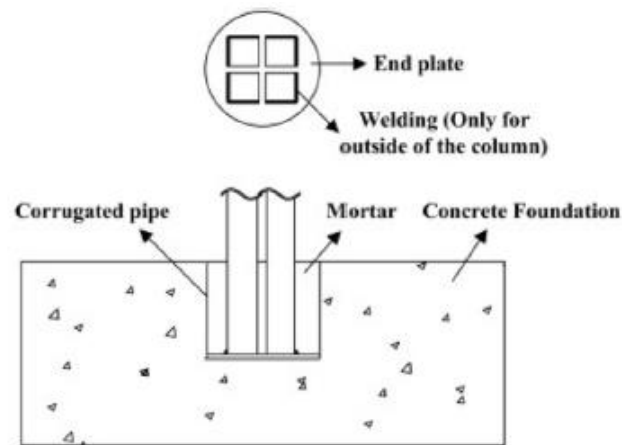


Figure 2-13 Embedded column connection [39]

2.1.7.3 Inter-modular Connections

Inter-modular connections are one of the most essential components of a modular building. These form connectivity between the modules in both horizontal and vertical directions, allowing the transferring of gravity and lateral loadings between modules [11]. The most basic type of connections for modular buildings come from the most elementary use of prefabrication, shipping container storage and housing [4]. When shipping containers are placed on top of each other to create extra space, they are connected with a twist lock that connects corner casts available at the corners of container boxes (Figure 2-14) [20, 40].



Figure 2-14 Twist lock and corner cast arrangement used in container houses

<https://adaptainer.co.uk/>, last accessed August 2020

Over time, as shipping container-type modules began to be manufactured with the specific purpose of providing accommodation and office space, techniques used in steel construction came into practice, and such construction became known as modular construction [5]. Moreover, because of the absence of specific regulations or guidelines that include design codes, those for conventional steel or concrete buildings were used to design modular buildings [11]. With further influence from conventional construction practices, the bolted connection,

which is the most commonly used connection type in conventional steel buildings, was used in different forms as a connection between modules.

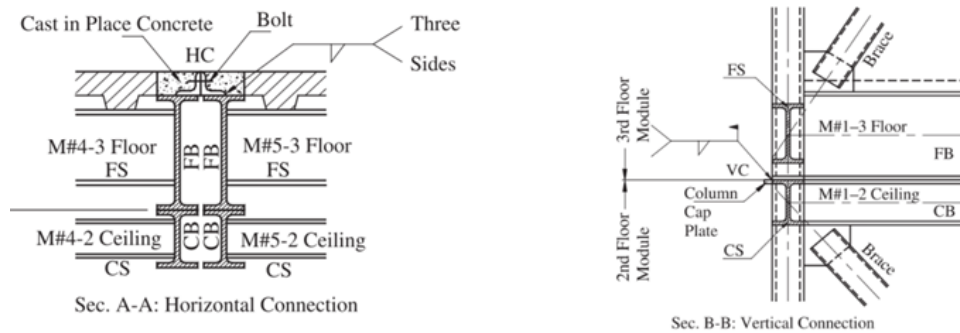


Figure 2-15 Horizontal and vertical connections proposed by Annan et al. [24]

Figure 2-15 shows the horizontal and vertical connections proposed by Annan and co-workers, as reported in Ref. [24]. The proposed horizontal connection consists of angle sections welded to the beams of the modules; connected via bolts and poured concrete as shown in section A-A of Figure 2-15. The proposed vertical connection consists of a column cap plate welded to the columns of the top and bottom modules [24, 33, 38]. This connection was introduced during the early stages of the modular construction boom, more than a decade ago. The approach used in this connection is more like the element-to-element connection in conventional steel buildings. Such connections drastically reduce the efficiency of modular construction because of the amount of work involved in developing the connection and the permanent nature of the connection as a result of welding and concreting. As mentioned in previous discussions, the efficient modular construction of the future would preferably require lower cost, less work on-site and detachable modules with no permanent connections.

Figure 2-16 depicts a connection system proposed by Gunawardena, that can serve as both horizontal and vertical connection between modules. It involves bolts as used in conventional building connections. The authors proposed three arrangements for the connections based on three different bolt diameters [12]. Their findings were beneficial to later research because the proposed connection was studied in detail for its stiffness, obtained through numerical simulations, experimental analysis and mathematical calculations based on the literature. Moreover, experimental analysis of the behaviour of the introduced connections under shearing actions using the set-up shown in Figure 2-17 provided the researchers with a better idea of their strength, stiffness and failure patterns [12].

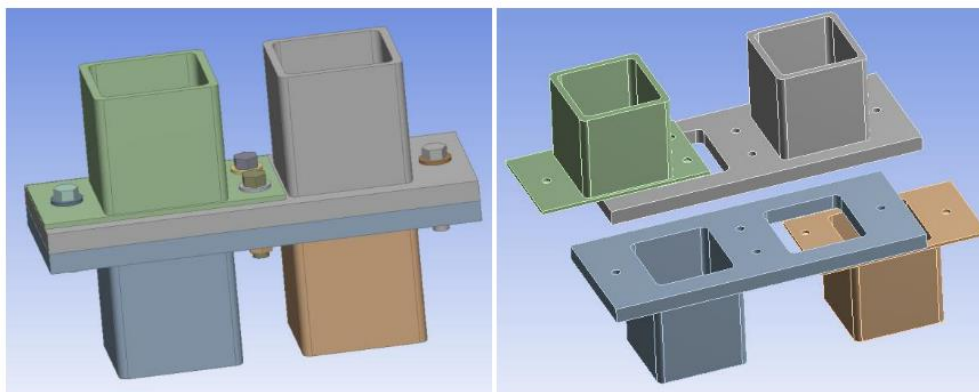


Figure 2-16 Connection system proposed by Gunawardena et al. [12]



Figure 2-17 Experimental set-up for testing the connection [12]

Figure 2-18 shows the load-deformation curve obtained for one type of connection following experimental and numerical analyses. It is shown that the connection deforms in two main stages. The flat line at the beginning of the curve in Figure 2-18 results from the slippage of the bolts. The second stage, corresponding to the increased slope of the plot, was reported to be due to the shearing and bearing of the bolts against the plates [12]. The results of Gunawardena et al.'s study have helped many researchers develop new connections and use the properties of these connections to conduct analyses of FE models of modular buildings. Although experiments under monotonic loadings have led to satisfactory performance of these connections, their behaviour under cyclic loadings may not be satisfactory because of the hazardous nature of a bolted splice connection, which may be seen under cyclic loadings. This is discussed in detail in Section 2.6.

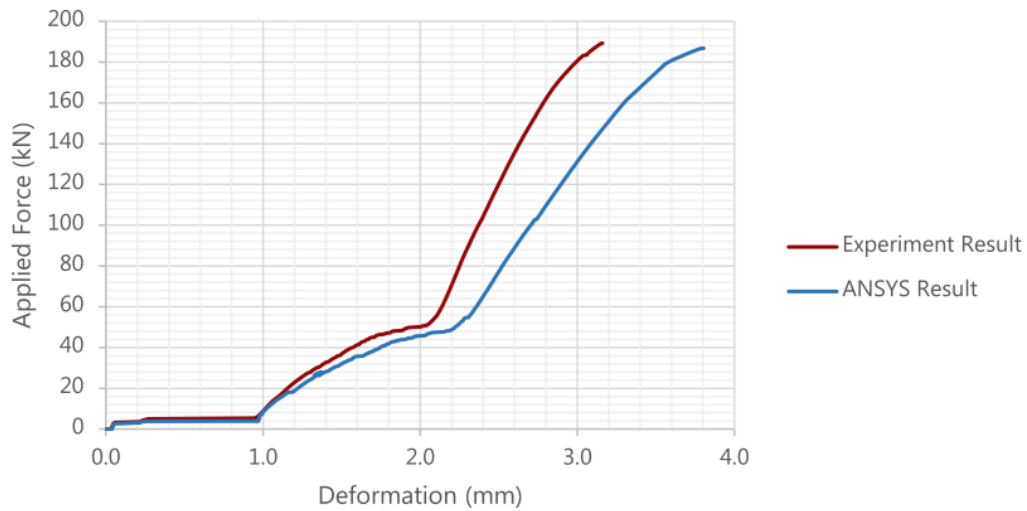


Figure 2-18 Load-deformation curves for one type of connection [12]

A connection reported in Ref. [41] has been used in practice in the industry. It involves a steel bracket connected at the corner of the module, as shown in Figure 2-19. This steel bracket serves as the horizontal and vertical connection system. Brackets at each corner are bolted to the brackets of other modules. Importantly, this connection eliminates the requirement for permanent connectivity between modules (welds and concrete). This makes it possible to consider detachable modules, which is one of the aims of a pure modular system. Despite these advantages, this connection has disadvantages in relation to the strength of the connection itself during lateral loadings and the poor access available to fix the connections where the four corners of four modules meet. Another disadvantage is that the gravitational loading will be taken purely by the bolt shank in shear [42], which may reduce the gravitational load-carrying capacity of the entire building.



Figure 2-19 Steel bracket connection introduced by Doh et al. [41]

Authors conducted experiments on this connection to study its behaviour under shear, tension and combined tension and shear. They concluded that the connections performed satisfactorily under the experimental conditions they used [41]. However, no testing was done to understand the performance of this connection under cyclic loading, which would affect the seismic performance of buildings consisting of these connections.

A new connection (Figure 2-20) system that includes beam-to-beam-connecting, high-strength bolted connections as vertical connections and cast plug-in devices as horizontal connections was reported in Ref. [43]. These tension bolts are hidden behind the baseboard in the infill walls, which does not require creation of slots/holes that will attract too many stresses and lead to scarification of the strength of structural members [43].

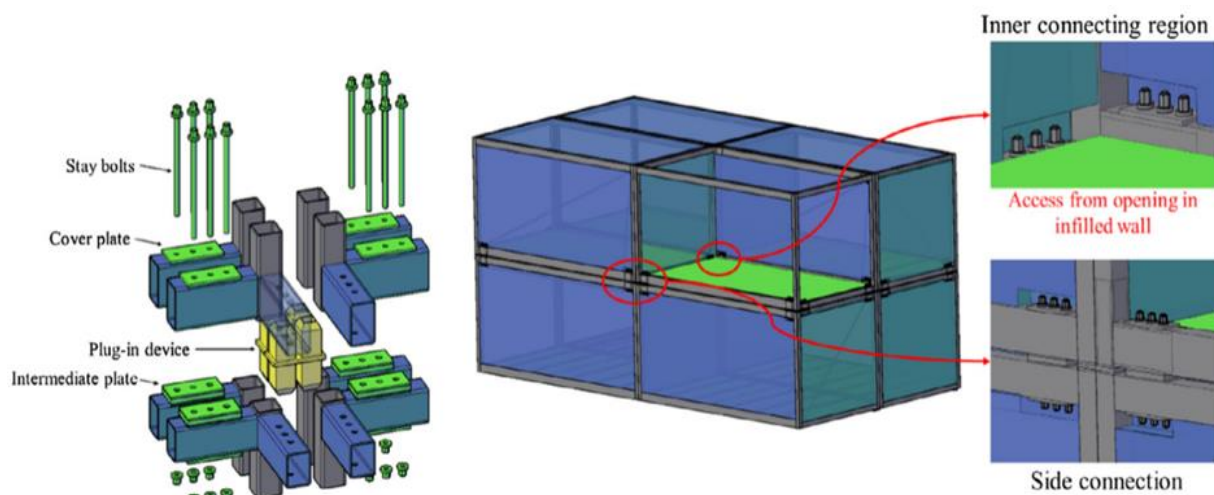


Figure 2-20 Connection system introduced by Chen et al. [43]

An advantage of this connection is that it is made up of longer bolts that are different to those used in conventional systems and there are no access issues with slots/open space available for fixing internal connections.

Moreover, because of the greater thickness of the plug-in device, these connections can act very strongly in the lateral direction [43]. As shown in Figure 2-21, Authors also compared their proposed connection system with an analogous conventionally available connection system.

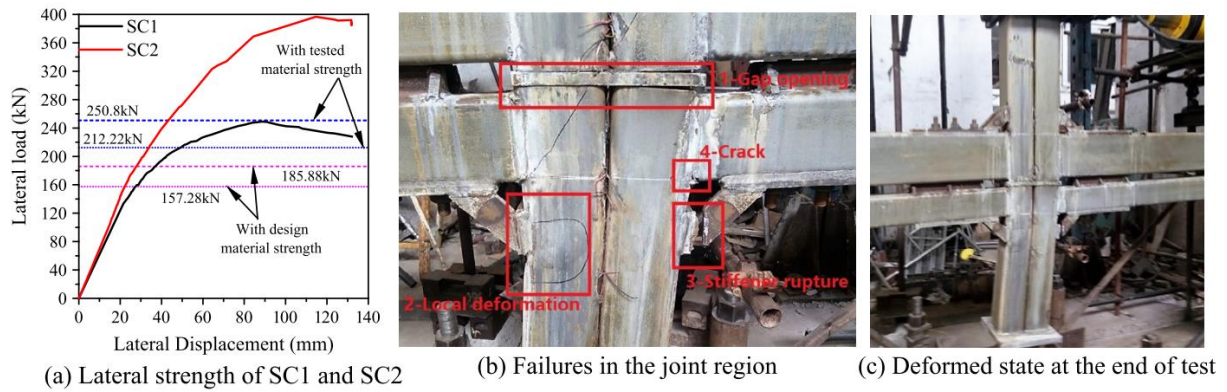


Figure 2-23 Observations during monotonic tests (connection with stiffener) of Chen et al. connectors [43]

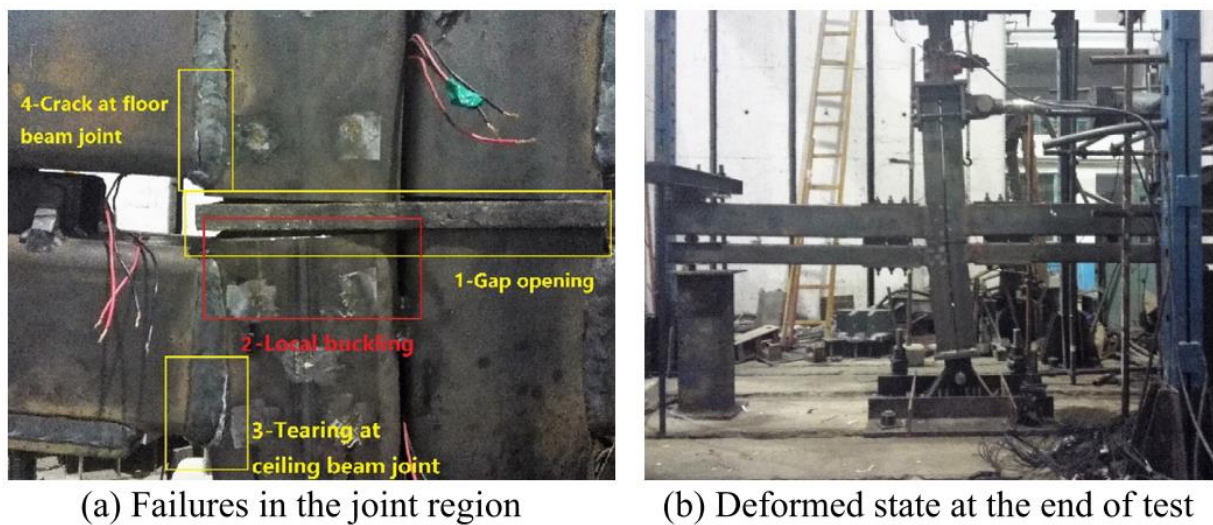


Figure 2-24 Observations during static monotonic tests (connection without stiffener) of Chen et al. connectors [43]

Figure 2-23 and Figure 2-24 depict observations under monotonic pushover testing with and without stiffeners, respectively. The primary mode of failure was due to a gap opening up in the main connectivity area between the upper and lower modules. It was also reported that local failure in beam-column connection areas was observed under both cases, with and without stiffeners [43].

Figure 2-25 and Figure 2-26 shows the observations for the cases with and without stiffeners, respectively, for test specimens under cyclic loading. It was reported that the failure patterns observed were similar to those observed under quasistatic pushover loading [43].

This connection system has some advantages over others introduced previously; for example, access is possible through the infill wall. Even though the plug-in device provides greater

lateral strength to the connection which pushes the ultimate failure to tearing of the beam-column connection or fracture of the stiffener, if the cyclic loadings were simultaneously applied axially and laterally, the connection would be in a more vulnerable situation.

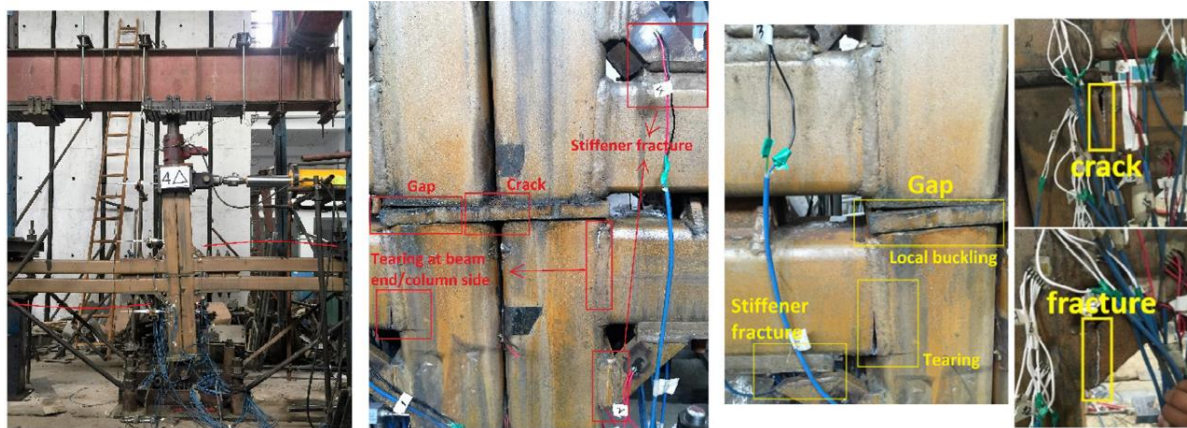


Figure 2-25 Behaviour of test specimens with Chen et al. connectors and stiffener under cyclic loading [43]

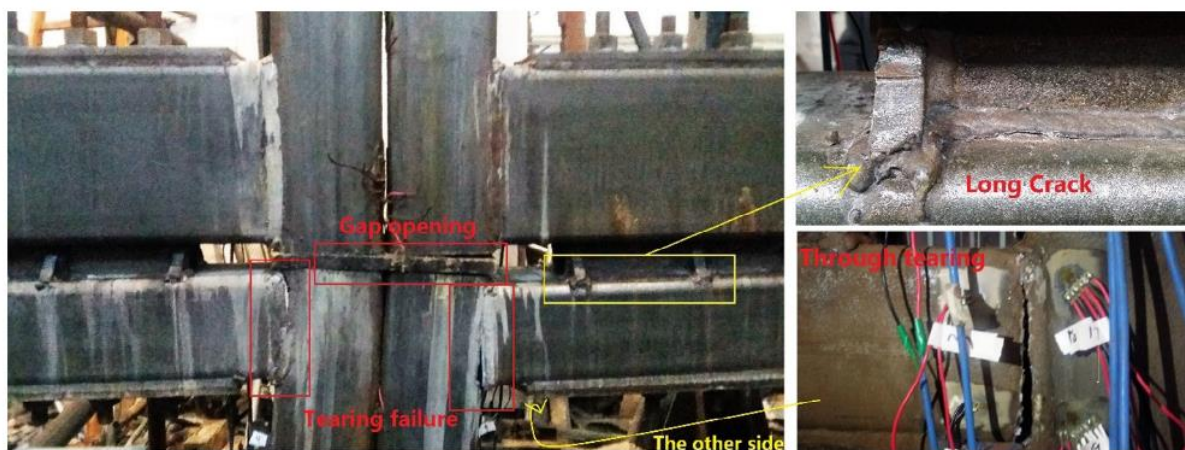


Figure 2-26 Behaviour of test specimens with Chen et al. connectors without stiffeners under cyclic loading [43]

VectorBloc [44] introduced a patented connection that involves bolts and gusset plates, as illustrated in Figure 2-27. This connection system has long bolts connected via gusset plates, which can be expected to behave safely during cyclic loadings. The need for individual connections via bolts and assembly of the gusset plates and other connected components may have an adverse effect on the efficiency of the construction process, which is not conducive to an efficient modular construction system. The features of a fully modular building are not reflected in the VectorBloc system modules because of the requirement to assemble each component part by part, as in a conventional building.

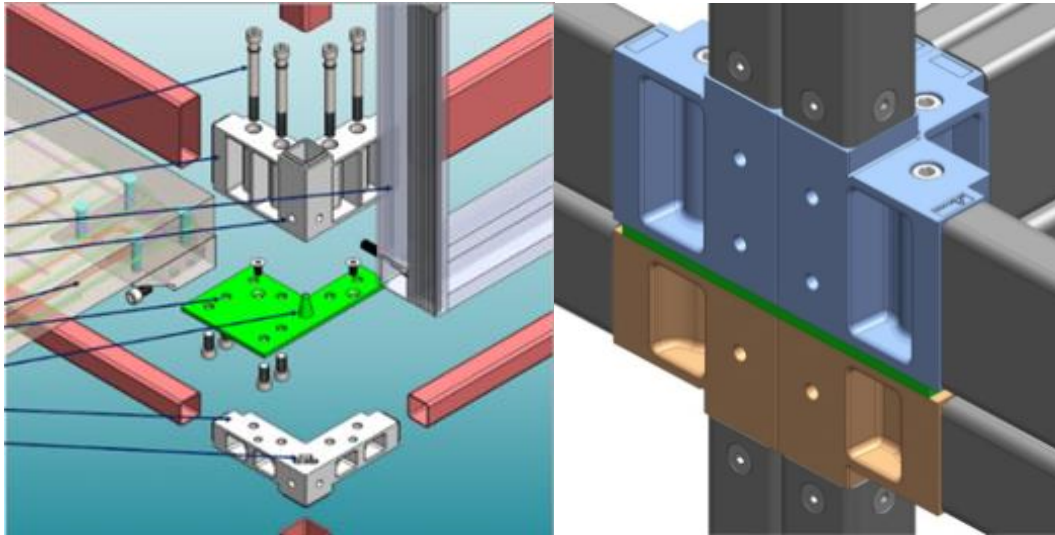
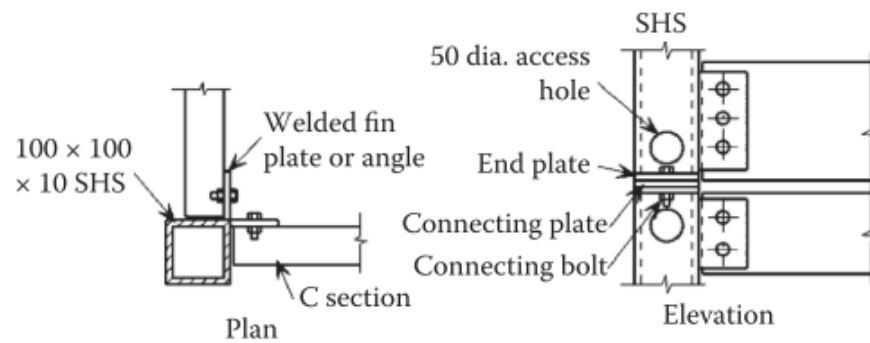
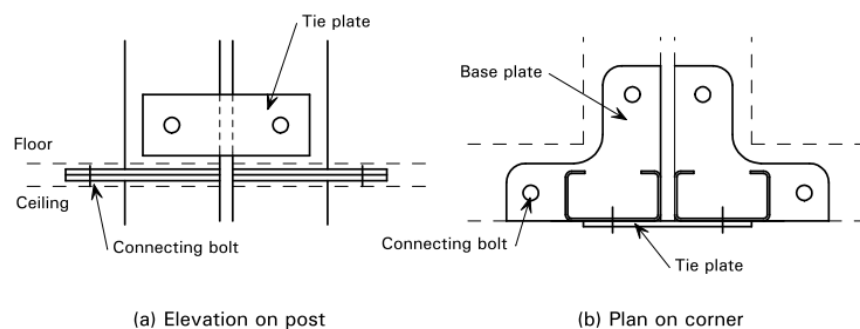


Figure 2-27 VectorBloc connection system [44]

A connection system used in practice in the UK, as shown in Figure 2-28 has been reported in Ref. [2]



(a)



(b)

Figure 2-28 (a) Vertical and (b) horizontal connection system proposed by Lawson et al. [2]

In this connection, vertical connectivity is provided by the column end plates connected through the bolts and clamped together (as shown in Figure 2-28 (a)). Horizontal inter-module connectivity is obtained using a tie plate that connects between the corner columns of adjacent modules using bolts, as shown in Figure 2-28 (b). These connections incorporate the purest forms of bolted connections that might put the connection at risk through the failure of bolts during the splicing action under cyclic loadings, as discussed in detail in Section 2.6. Moreover, the availability of access holes in the column section may require additional focus on localised strengthening of framing elements [2].

The connection system shown in Figure 2-29 comprised of a gusset plate connecting four module corners, as reported in Ref. [45]. While the gusset plate is capable of stitching together the modules horizontally, a long bolt running through the module's height is used to connect modules vertically [45].

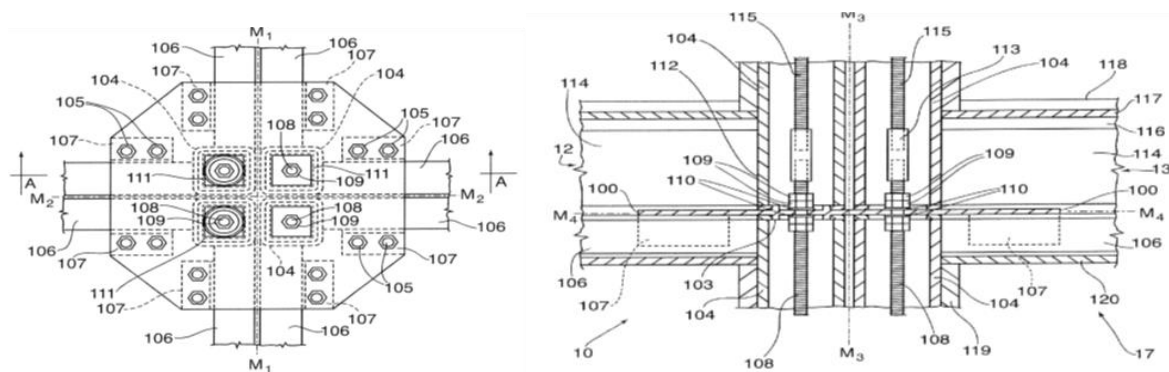


Figure 2-29 Connection system proposed by Farnsworth et al. [45]

Figure 2-30 shows a connection system used in the construction of the Latrobe Tower, a 44-storey modular building in Melbourne [46]. This connection has some of the features of conventional steel connections used in practice. It is an excellent example of connections used in current modular building construction around the world. The connection assembly shown in Figure 2-30 represents an intermediate connection, but other connections follow a similar pattern [46]. It can be seen that the column end plates connected via bolts provide vertical and horizontal connectivity between the modules. These connections might also undergo simultaneous shear and axial loadings during seismic events, putting the connections at risk of failure through failure of the bolts. The risk involved in this sort of connection, which involves a pure form of bolted connection assembly, is discussed in detail under Section 2.6.

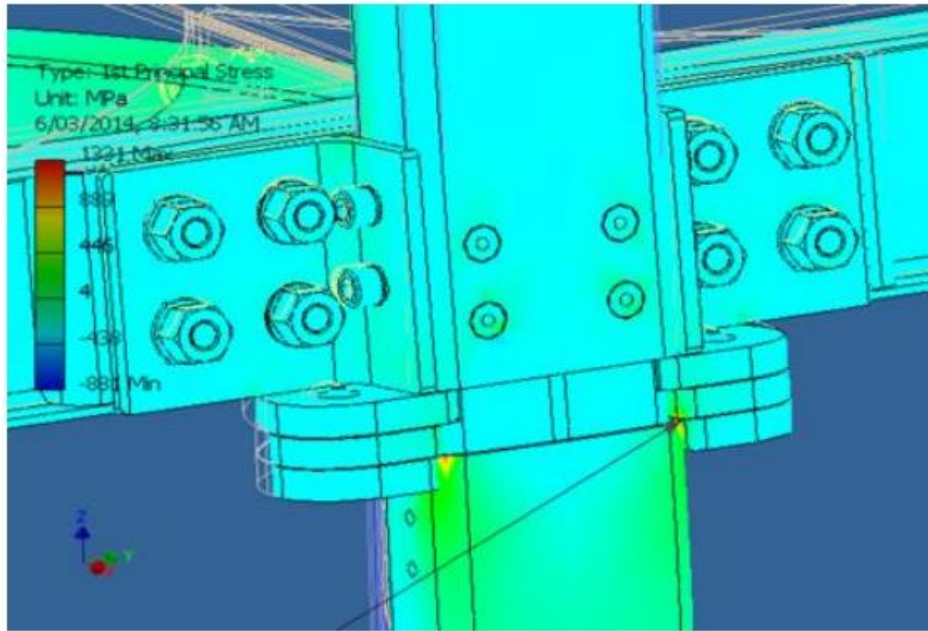


Figure 2-30 Connection system used in Latrobe Tower, Melbourne, Australia [46]

Figure 2-31 depicts a connection method reported in Ref. [47], in which vertical and horizontal connectivity is governed by bolts that connect through the beams and columns of upper and lower modules [47]. This connection has bolts that might act as weak links in the connection under cyclic loading. The presence of access holes in the columns of the connection area might make the column section weaker, necessitating localised strengthening of the column elements.

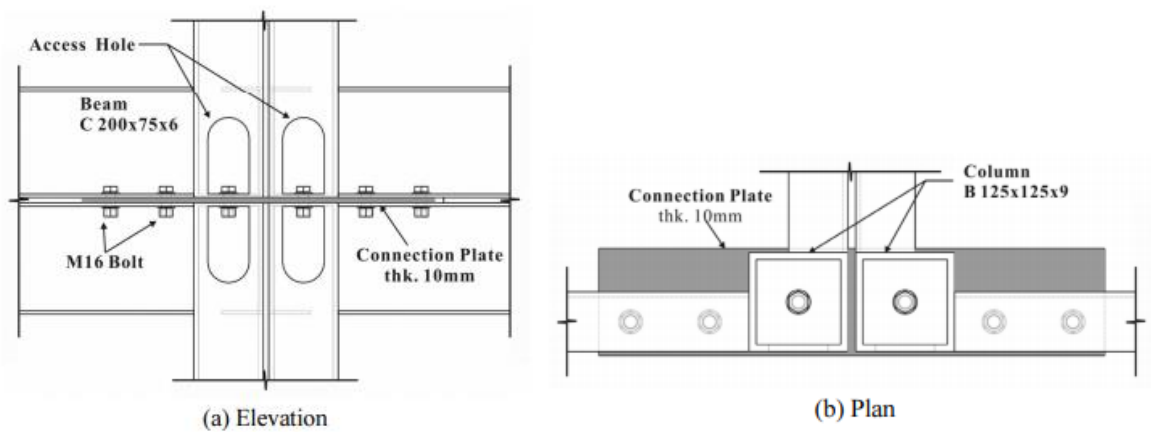


Figure 2-31 Connection system introduced by Choi et al. [47]

Figure 2-32 presents a connection system reported in Ref. [37], where vertical connectivity is provided by a generic column-column connection using bolts. Side plates connected using bolts are employed to provide horizontal connectivity and tying action between adjacent modules [37].

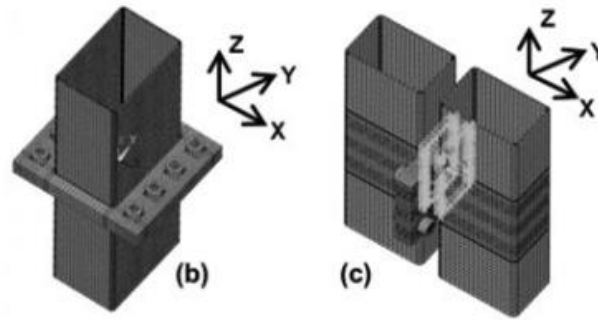


Figure 2-32 Horizontal and vertical connections proposed by Styles et al. [37]

Figure 2-33 illustrates a connection system reported in Ref. [48]. The connection has pre-stressed strands between stiffened sealing plates at the ends of columns, along with plug-in bars and shear blocks, which take part in providing racking resistance and vertical connectivity between the stacked modules. The shear block was introduced to assist in aligning the modules properly during assembly. In contrast to generally used modules, the columns of the modules consist of concrete-filled tubes. The plugin-bars inside these columns are useful in providing extra ductility and preventing the concrete from crushing [48].

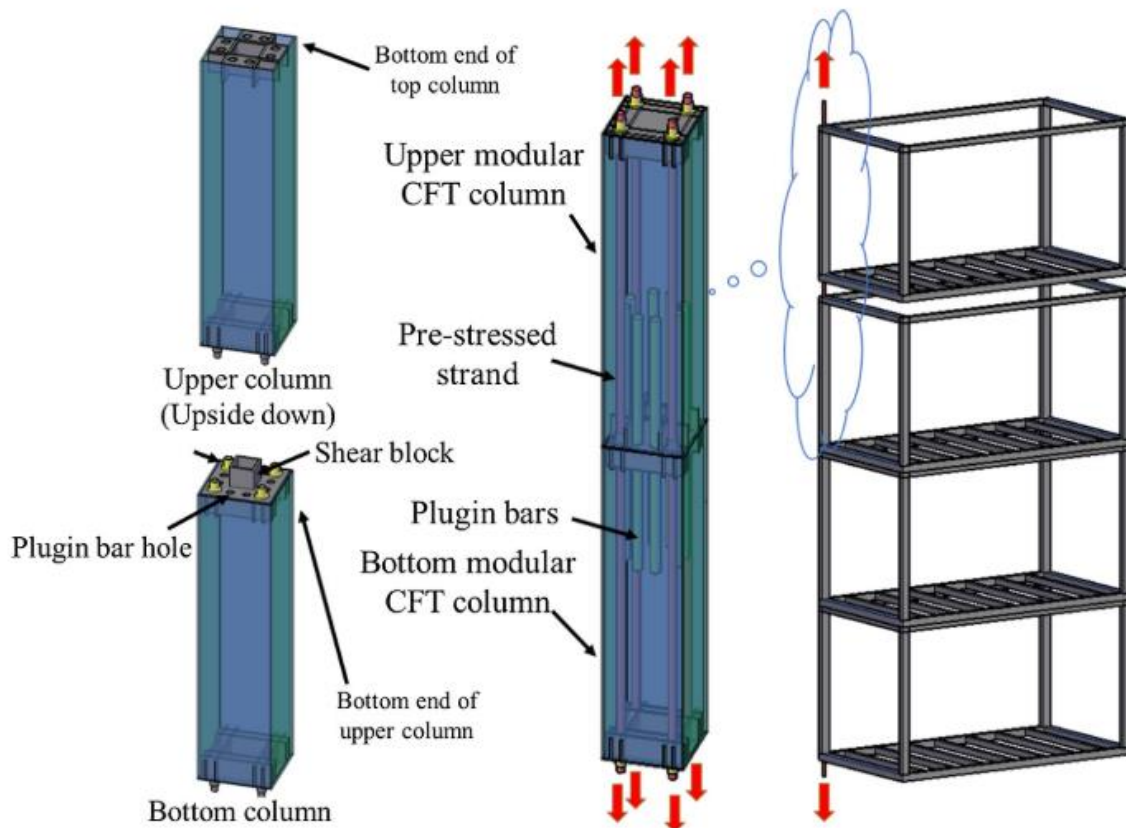


Figure 2-33 Pre-tension connection system introduced by Chen et al. [48]

A connection system as reported in Ref. [49] is illustrated in Figure 2-34. In this connection, the vertical connectivity is obtained using a gusset plate assembly, as shown in Figure 2-34, which connects between the upper and lower modules. [49].

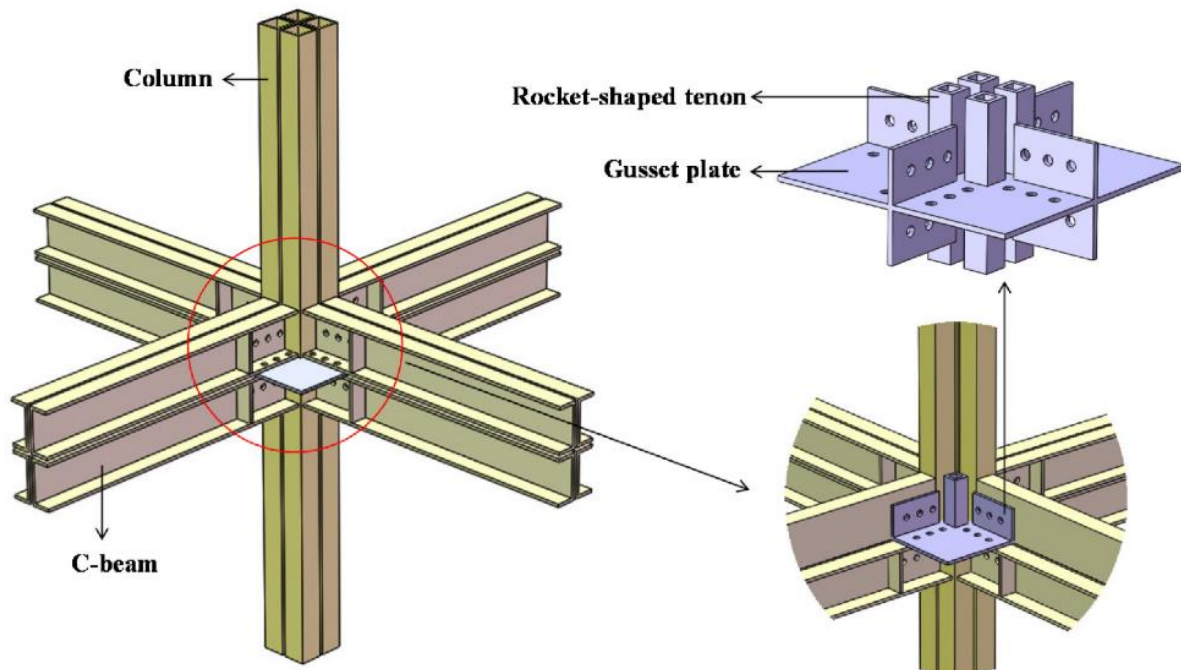


Figure 2-34 Connection system proposed by Deng et al. [49]

Figure 2-35 shows a connection system introduced in one of the latest studies as reported in Refs. [11, 50]. This connection has a bolt assembly taking care of axial and shear connectivity, similar to many connections presented previously. The unique aspect of this connection is that it has a resilient layer placed between the endplates of the columns connecting the top and bottom modules. This resilient layer increases the ductility of the connection to shift failure away from elements such as beams and columns in the building [11, 50].

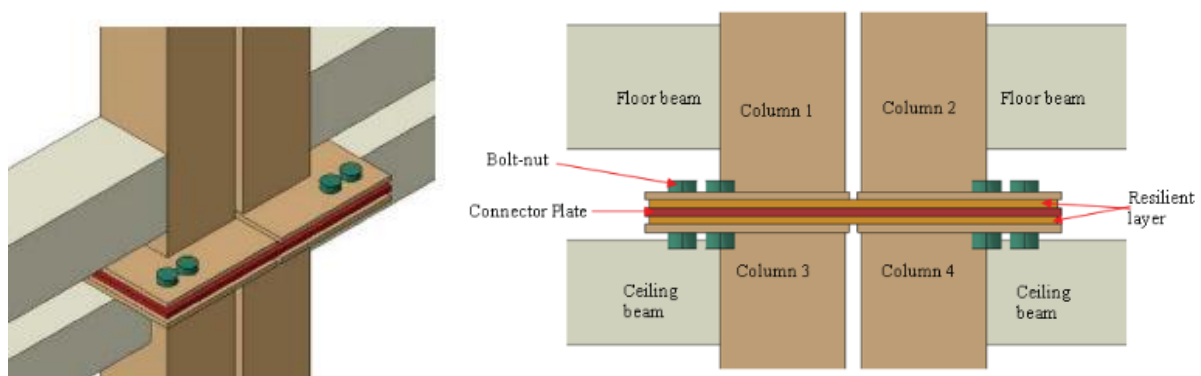
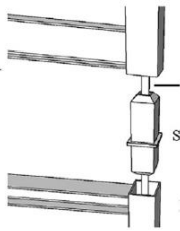
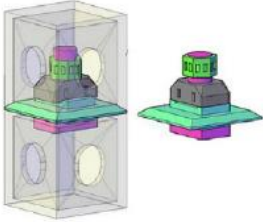
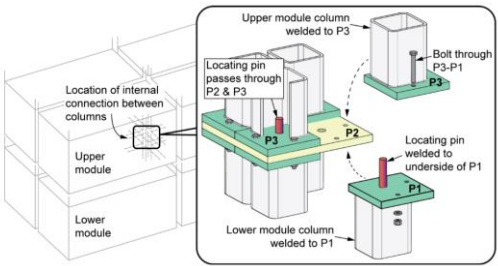
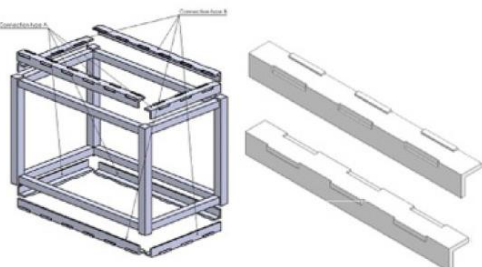
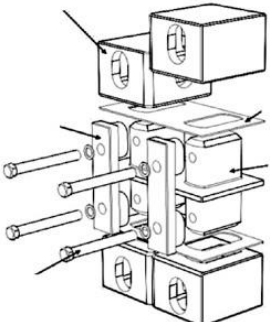


Figure 2-35 Connection system introduced by Sendanayake et al. [11, 50]

Apart from the connections discussed above, many other connections have been developed and are used in current practice, as summarised in Table 2-1.

Table 2-1 Modular building connections

 [51, 52]	Vertical connectivity: uses a pre-tensioned rod passing through the columns of modules Horizontal connectivity: conventional bolted side plate connection
 [53]	Vertical connectivity: through a connector consisting of a key-type rod, nut and plate Horizontal connectivity: A single connection can be formed by welding plate elements of adjacent connectors
 [54]	Vertical connectivity: endplates of columns are bolted, similar to a general bolted connection Horizontal connectivity: bolts used to connect end plates together with a locating pin that resists lateral loading
 [28]	Vertical connectivity: through a tongue and groove arrangement present in the ceiling and floor beams of modules Horizontal connectivity: as for vertical connectivity
 [40]	Vertical connectivity: through a corner casting assembly with a double spigot casting connector and lock-down plates with spigots and bolts Horizontal connectivity: via the transfer plate of a primary double spigot casting connector and additional washer-like or packing plates

Based on the connections presented here, it can be concluded that most connections that are generally used involve bolt action in resisting lateral and gravity loadings. Bolted connections

are widely used in conventional construction because of their flexibility in usage and many other advantages. However, a bolted connection may behave in a dangerous manner when subjected to cyclic loading. More details about bolted connections and their behaviour under cyclic loading are presented under Section 2.6.

2.1.8 Seismic Response of Modular Buildings

Few studies have examined the nonlinear behaviour of modular buildings compared with the number that has focussed on developing and studying the behaviour of different types of connections for modular buildings. The overall response of a modular structure is as important as the behaviour of a single connection under seismic events, especially at the nonlinear stage. Since modular buildings are designed to conventional building design standards, their response to seismic events within design limits is expected to be similar to the response of conventional buildings.

Because of the nature of their construction, modular buildings are more discrete along their height than continuous, like a conventional building. Thus they differ in their response behaviour when undergoing failure in just one connection, which might suddenly increase the damage and cause a catastrophic failure [55]. Hence, it is vital to study the post-elastic behaviour of modular buildings under the influence of increasing seismic events, which exceed the design limit. Even so, most of the studies conducted so far on modular buildings focus mostly on the dynamic response of modular structures within the elastic limit or immediate post-elastic stage. Studies which focussed on the dynamic behaviour of modular structures up to the collapse stage are scarce. Some of the studies which focussed on the seismic performance of modular structures are reviewed below.

2.1.8.1 Work Reported by Annan and Co-workers [24, 33, 38]

Experimental testing and numerical analyses of a modular steel building (MSB) and a traditional concentrically braced steel frame analogous to the MSB in terms of physical dimensions has been reported in Refs. [24, 33, 38]. Observations showed that both the MSB and conventional steel frames behaved in a stable and ductile manner (up to drift ratios around 3.5%). It was further reported that the MSB frame entered nonlinearity because of buckling of columns while nonlinearity of the conventional steel frame was due to out-of-plane buckling of the braces. The research also involved incremental dynamic analysis (IDA) on two-, four- and six-storey two-dimensional (2D) MSB-braced frames. All three types of frames behaved

in a first-mode dominant manner (e.g. the mass participation factors for the first and second modes of the two-, four- and six-storey frames were 94% & 5%, 81% & 15% and 77% & 17% respectively). Inelasticity concentration in the first storey due to the redistribution of internal forces was also reported for these frames [38].

In the study discussed above, the inter-modular connections were more of a fixed nature because of the involvement of concreting and welding in these connections, as discussed above (see Figure 2-15). Thus, the connections were very strong, and it was the columns that were vulnerable in the MSB. However, connections that involve concreting or welding make them permanent and involves a lot of work on-site. This reduces the efficiency of modular construction and are getting out of the practice in modular construction.

The work of Annan et al. served as a preliminary guide for researchers conducting further studies on different configurations of modular buildings and connections. The approach used by Annan et al. (see Figure 2-36) for modelling discrete connections in the modular building has been developed further by other researchers to model inter-modular connections in FE models.

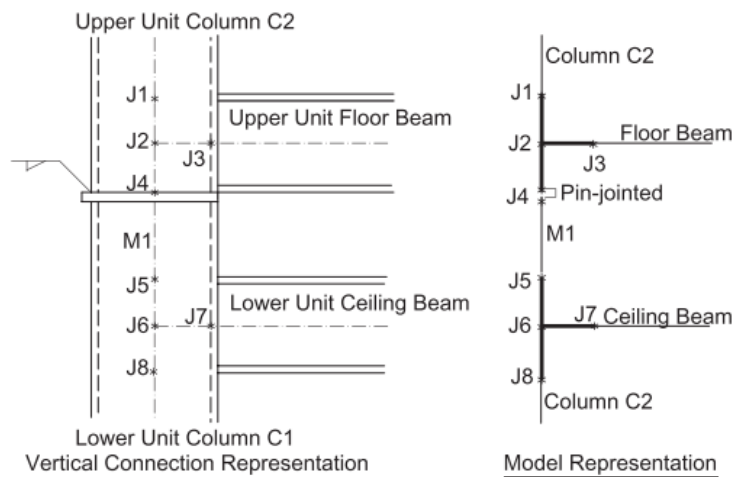


Figure 2-36 Connection model used by Annan et al. [33]

The rigidity of connection areas was captured using rigid end blocks at the end regions of the beam-column joints (represented by bold lines J1–J2, J2–J3, J2–J4, J5–J6, J6–J7 and J6–J8 in Figure 2-36). Similarly, the rigid behaviour experienced by the braces at the gusset plate area was modelled by rigid end blocks. The gap between the upper and lower modules (represented by a short column segment, M1, in Figure 2-36) was modelled using an inelastic beam-column frame element in the FE model. This vertical element was pinned internally into a common

joint with upper unit column J4, such that an independent top and bottom module rotation would develop at this common joint. To ensure this occurred, the short column element representing the gap was pinned internally into a common joint with upper unit column J4 [33].

2.1.8.2 Work Reported by Fathieh and Co-workers [34, 56]

Examination of the behaviour of 2D and 3D four-storey MSB-braced frames using IDA and static pushover analysis has been undertaken [34, 56]. The research aimed to capture interactions between modules, as well as the unique diaphragm action of each floor module; the shear and moment forces in the horizontal connections within the modules; and the displacement and rotations caused by the horizontal connections [34, 56].

As the outcomes of the research, the authors proposed suitable intensity and damage measures that could be employed when conducting IDA for modular buildings. It was also reported that the concentration of inelasticity in these buildings is mainly in the first storey of the building. The research followed a similar approach to that of Annan et al. in terms of modelling the connections in the FE model [34, 56].

2.1.8.3 Work Reported by Gunawardena and Co-workers [12, 57]

As the first part of their research, Gunawardena and colleagues investigated the possibility of having a 20-storey modular building without any external LFRS. The viability of the building in terms of requirements outlined in the design codes for conventional steel buildings in Australia was evaluated through further analysis. Authors conducted static pushover analysis to understand the nonlinear static behaviour and to verify whether the capacity curve from the latter satisfies the demand curves for six chosen ground motions presented in the form of an acceleration displacement response spectrum (ADRS). Moreover, a nonlinear THA was conducted on the selected model to study its behaviour under the six selected ground motions. It was reported that the maximum inter-storey drifts observed were well within specified limits; the natural periods of the building also satisfied code requirements [12]. This aspect of Gunawardena's research is reviewed in detail in Section 2.5.

Once the possibility of a system without an external LFRS had been studied, authors repeated the analysis on a 10-storey model without an external LFRS. The most critical connections and the elements connecting them were monitored during the nonlinear THA to identify possible hinge formations. Of the hypothesised hinge formations, the most likely scenario was reported

to be hinge formation in the main corner column just under the floor beam or above the roof beam. A capacity spectrum analysis conducted on this structure revealed that the structure was capable of performing at ‘life safety’ (LS) levels under the applied ground motions [12].

Although Gunawardena’s research was a useful initiative in regard to the feasibility of pure modular structures and the future of modular construction, the modelling assumptions made in the FE models, especially in modelling the connections, were unrealistic. The researchers made the same assumptions as in previous studies [33, 34, 38, 56]. Accordingly, the real flexibility and strength of the connections were not accounted for, resulting in failure being simulated only in other connecting components.

2.1.8.4 Work Reported by Lacey and Co-workers [35]

A study was conducted on a six-storey modular apartment building located in Port Hedland, Western Australia [35]. The case study building consisted of modules 11.885 m long, 4.95 m wide and 2.95 m high, arranged on either side of a central 2.5-m-wide corridor. The overall size of the numerical model was 30.30×26.29 m, with height at the tallest point of 21.50 m. One unique aspect of this study compared with those discussed in previous sections was its modelling of connections using FE software and use of real stiffness and flexibility values, highlighting the discrete nature of the construction due to the connections between modules. This essential consideration, lacking in previous studies, reflected the real in-built conditions of modular buildings. Instead of using rigid links and pin connections to represent connections (e.g. see Figure 2-36), Authors used link elements, to which the real stiffness properties for each direction were assigned, as shown in Figure 2-37 [35].

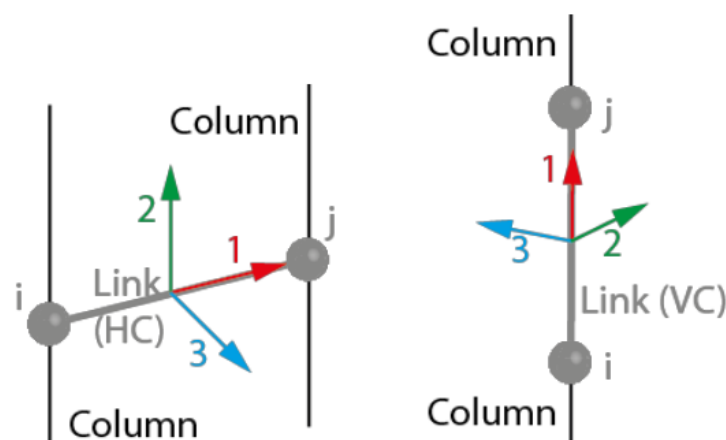


Figure 2-37 FE modelling of Lacey et al.’s connections [35]

Initially, a response spectrum analysis was conducted using the design response spectrum for Australia (Australian Seismic Design Code AS1170.4). Following this analysis, the structure was analysed under 20 earthquake time histories scaled to AS1170.4 based target spectrum. Maximum inter-storey drift ratios were compared between the time history and response spectrum analysis results. The two analyses gave similar results for maximum inter-storey drift in terms of magnitude and location, which was recorded to be the second-storey level. Moreover, variation in the response of the building was investigated for different inter-modular connection stiffnesses [35].

Despite its advanced modelling aspects, which took into account the reality of the connections' stiffness and strength, the study was restricted to the response of the structure only within the design range. This is a severe weakness in most studies conducted to date on modular structures.

2.1.8.5 Work Reported by Sendanayake and Co-workers [11, 50]

In this study, the overall structural response and inelastic behaviour of a medium-rise modular frame (eight storeys) employing the optimised connection system proposed in Ref. [50] (see Figure 2-35) was captured using a 2D nonlinear numerical model developed in ABAQUS. Three ground motions were employed to study the nonlinear behaviour of the building. The researchers reported unexpected variation in lateral inter-storey drifts due to discontinuities in the inter-modular connections. They also replaced the introduced connection with a similar connection but without the resilient layer, to compare the effectiveness of the new connection with that of a conventional connection. They concluded that the change from the conventional to the new modular connection results in observable seismic mitigation of medium-rise steel modular frames [50].

The above review of studies on the seismic behaviour of modular buildings identifies two essential aspects not accounted for in previous work. The first and most critical gap is the inaccurate treatment in FE models of the reality of connection properties. Most studies have only managed to represent inter-modular connections as a hinge or a rigid link, which does not account for the real stiffness and strength of connections. This will lead to inaccurate FE models in which the real behaviour of the structure is not replicated, underestimating the risk that may be posed by the availability of connections between modules. Without proper modelling of its

connection properties, a structure is more likely to be represented as a conventional structure in FE models. However, the most recent research [35] has taken a step forward in accounting for this shortcoming.

Another strong concern with most studies on the seismic response of modular structures is their focus on the seismic performance of structures within design limits. Intuitively, and from the results of the studies reviewed above, it can be concluded that structures perform satisfactorily under design conditions because they were designed to adhere to corresponding code requirements. However, dynamic events might not always be within design limits, and the design codes that have been followed were primarily developed for conventional structures. Hence, adherence to these codes does not necessarily guarantee safe performance of modular structures, especially if they experience a seismic event beyond design limits. Thus, it is vital to conduct analyses of modular buildings under more intense dynamic loadings; importantly, when seismic intensity exceeds design intensity. IDA is one of the more common approaches used in this type of study. Although some studies [34, 56] involved conducting IDA on modular structures, their modelling did not take into account the reality of the connections (as discussed above). Such studies focussed mainly on modular structures with connections of a permanent nature (e.g. welded or concreted), which are more like conventional buildings.

2.2 Structural Systems in Conventional Steel Buildings

Structural systems are the components with the specific task of transferring to the ground all the loadings coming onto the structure, with no damage to any element of the building through excess internal stresses [57]. In this scenario, the loadings include mainly static and dynamic loadings. Static loadings are due to self-weight and live loads in a building (vertical loads), and dynamic loadings are mainly caused by seismic events and wind (lateral loads). Hence, two main structural systems can be identified as vertical load resisting and lateral load resisting structural systems. Among these two main types of structural systems, vertical load-resisting systems are similar in all buildings, irrespective of the height, width, or geographical location of construction. Vertical loadings are taken care of by the floor systems, connections, and other elements of the structure. The components that resist vertical loadings are usually the primary elements that make up the structure; without these elements, the structure would not exist. Hence, these structures and their components are designed to carry the intended vertical loadings on the structure, and the structures do not need any added components to resist gravity loadings.

In contrast, the structural skeleton by itself may not be able to resist lateral loadings in a structure depending on the height, geographical location of construction and many other factors that can change the dynamic loadings (wind and seismic) experienced by the structure. Accordingly, the structure might require additional components installed or lateral load-resisting capability provided to its existing components to withstand lateral loadings. Thus, lateral load-resisting systems play a significant role in the design of structures to perform safely under lateral loadings. Some lateral load-resisting structural systems used in practice are discussed below.

2.2.1 Frame Action

Bare frames with not specifically installed LFRS such as bracings can also be designed to take lateral loadings to some extent. Moment-resisting frames provide the most basic form of lateral force resistance through frame action. Steel moment frames consist of vertical frames made by joining flexible members through rigid joints. The continuity of beams and columns obtained through rigid connections and the bending strength (flexural) of frame elements help resist lateral loads in this type of frames. Rigid connection points transfer the moments from beams to columns. Because of the inherent continuity of monolithic construction in concrete structures, they act as typical moment-resisting frames [58]. However, steel frames require rigid connections between beams and columns to provide moment-resisting frame behaviour. Figure 2-38 illustrates a typical moment connection in a steel frame, and Figure 2-39 shows some moment-resisting frames available in practice.

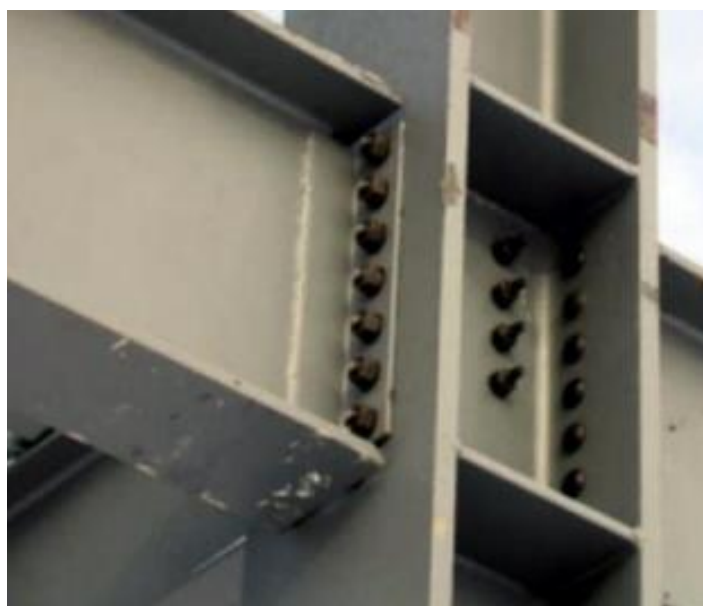


Figure 2-38 Steel moment connection [58]

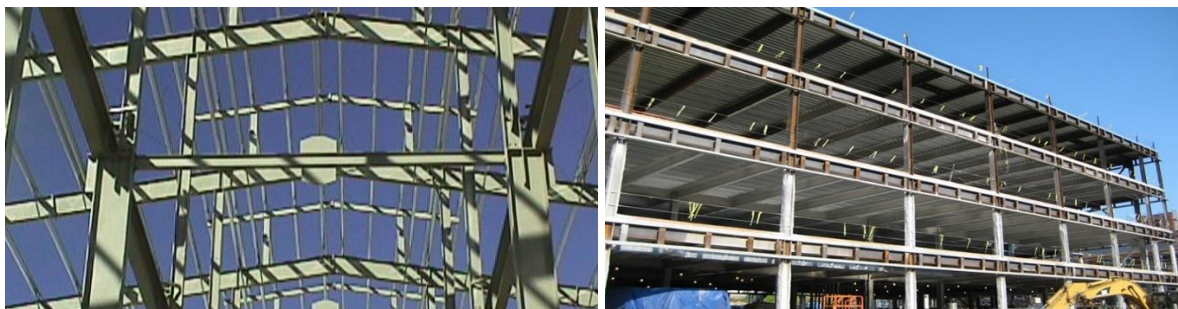


Figure 2-39 Example moment-resisting frames <https://s3da-design.com/> and <https://www.whirlwindsteel.com/>, last accessed August 2020.

2.2.2 Braced Frames

Braced frames provide an extra element in addition to the primary members of the framework. The lateral load resistance in these frames is provided by transferring axial forces in the form of tensile or compressive forces through these extra elements, which are fabricated diagonally inside the frame. The loads coming onto these braces from the floor diaphragm are transferred to the foundations through shear connections [58]. There are two main types of bracings, concentric and eccentric bracings (Figure 2-40), the choice of which depends where on the element the braces are to be connected.

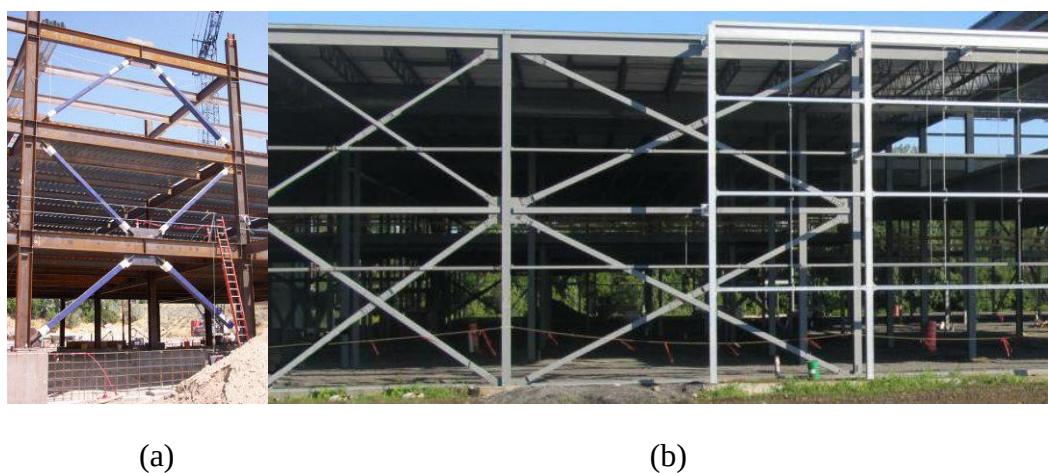


Figure 2-40 (a) Eccentrically and (b) concentrically braced systems <https://ed808.com/>, last accessed August 2020

In concentrically braced systems, the braces are connected to the ends of the elements. A relatively stiff frame is created in the case of lateral loads because of the formation of truss action. Single diagonal bracings, V-bracings and cross bracings are some of the commonly available configurations of concentrically braced frames. When eccentrically braced structures

are used, one or more braces are connected to the supporting frame member eccentrically, rather than being connected at the ends of the members as in concentrically braced frames. The braced frame ductility increases due to this eccentricity. Eccentric braces are prevalent in regions of high seismic activity. According to the height of the structure and the design lateral loadings expected on the structure, designers may choose to have bracings in a building [58].

2.2.3 Shear Walls

Moment-resisting frames and incorporated braces might not always enhance the seismic performance of a structure. In particular, when the structure is mid- or high-rise, or is located in an area of high seismicity, additional lateral load-resisting systems like shear walls may be required (see Figure 2-41) [59]. Shear walls act as vertical diaphragms to withstand lateral loadings through in-plane shear. The walls act more like vertically spanning beams, and their length is greater than their height, making use of the in-plane stiffness [58]. Lateral force resistance of shear walls occurs through combined axial–shear–flexure action [59]. Vertical reinforcement is designed to resist the flexural loading coming onto the wall. For the best performance of shear walls in structures, they should be constructed throughout the full height of the structure [59].

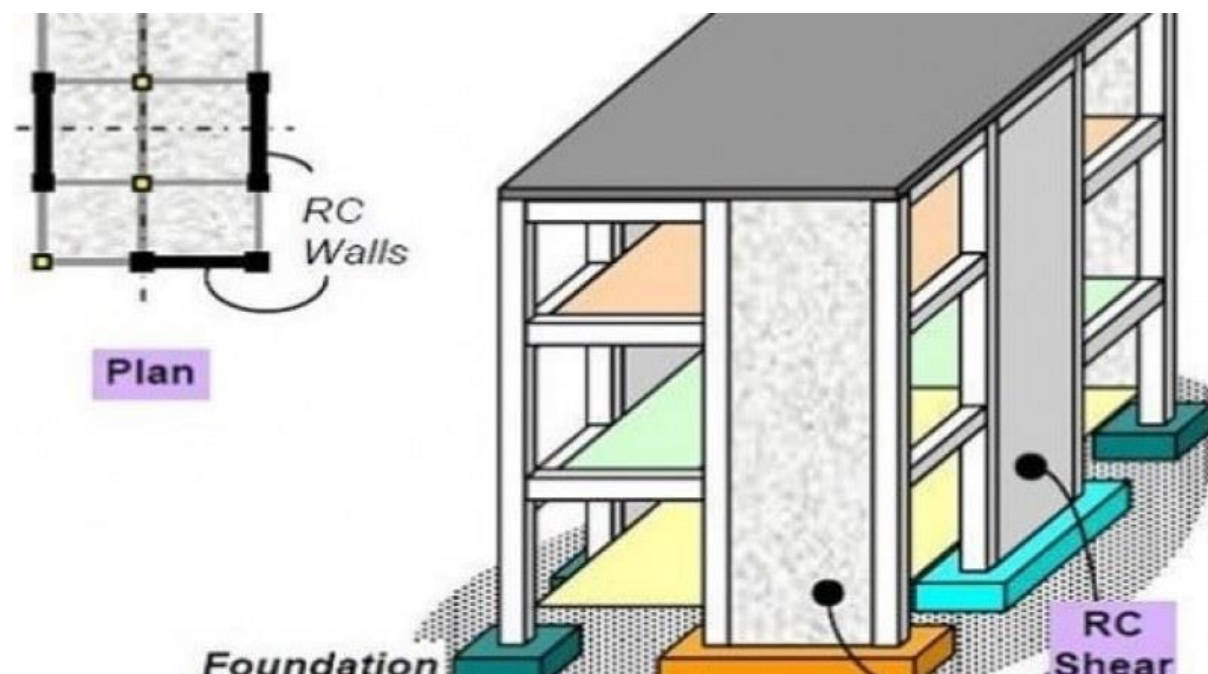


Figure 2-41 Shear wall system in a structure <https://theconstructor.org/>, last accessed August 2020

2.2.4 Combined Shear Wall–Frame Interaction

Shear walls are widely used in combination with frames, rather than alone, to resist lateral loadings. Many researchers have studied the shear wall–frame interaction. In this combined method, the frame usually deforms in shear mode while the shear wall primarily behaves by bending as a cantilever (Figure 2-42) [60].

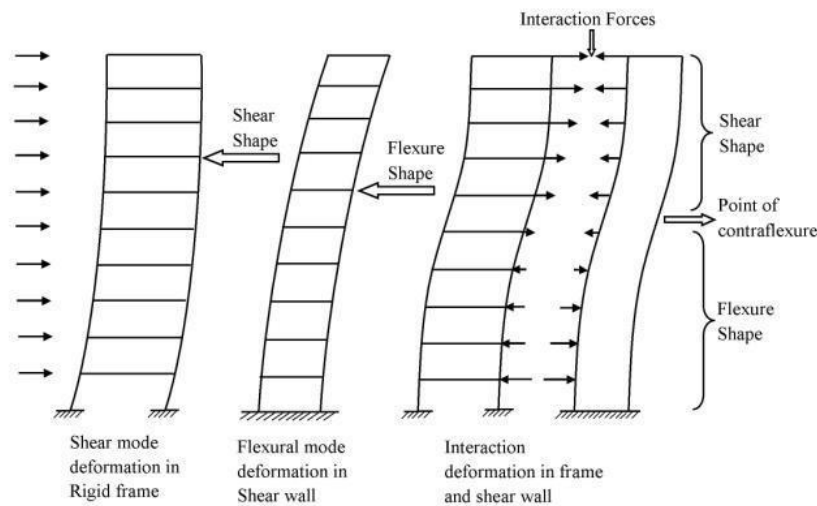


Figure 2-42 Frame–shear wall interaction <https://www.civilax.com/>, last accessed August 2020

2.2.5 Coupled Shear Walls

This type of shear wall is built by interconnecting two or more shear walls using a system of beams or slabs so that the total stiffness of the assembly will be higher than individual shear walls. Figure 2-43 shows the difference between an isolated and coupled shear wall [60].

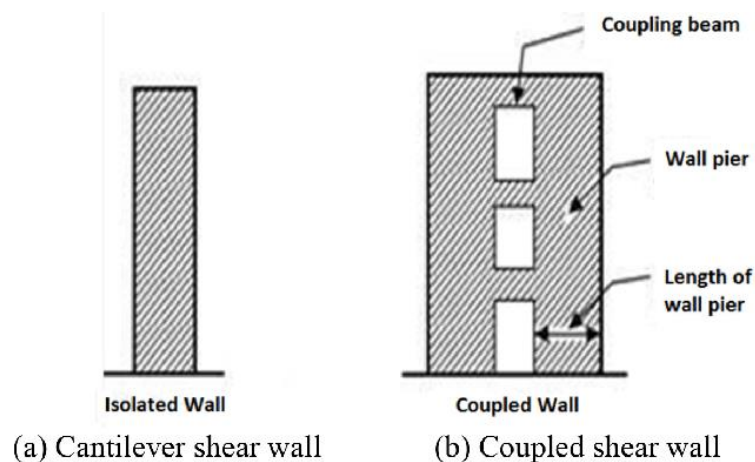


Figure 2-43 Comparison of coupled and isolated shear walls [60]

2.2.6 Tube System

Tube systems have driven a revolution in the design of high-rise structures. Khan pioneered the invention of such systems in the 1960s. Khan defined a tube system as:

A three-dimensional space structure composed of three-four, or possibly more frames, braced frames or shear walls, joined at or near their edges to form a vertical tube-like structural system capable of resisting lateral forces in any direction by cantilevering from the foundation [60, 61].

In tubular structures, the whole structure participates in resisting overturning moments, operating as an inherently stiffened 3D frame system. Tubes can be constructed of shear walls, beams, and columns to make them act as a single unit. A spatial skeleton formed by deep spandrels connecting the closely spaced exterior columns is one of the essential features in tubes that is also advantageous in resisting lateral loads in 3D structural space [60].

Tubes have been used in a range of configurations including framed tubes, tube-in-tube, braced tubes, solid-core wall tubes and bundle tubes. Some of these configurations are briefly discussed below.

A tube-in-tube system is a coupled structural system with an outer tube around the exterior of the building and an inner core structure at the centre or interior of the building. The core and the outer tube are usually connected using the floor diaphragms or outriggers. The combined system shares the lateral loadings imparted on the structure. Petronas Towers in Malaysia is one example of tube-in-tube construction [62]. Figure 2-44 depicts the tube-in-tube type structural system employed in Petronas Towers.

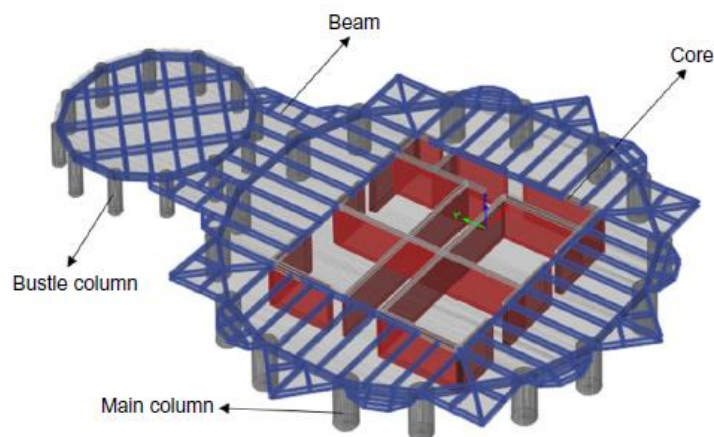


Figure 2-44 Tube-in-tube structural system of Petronas Towers, Malaysia [62]

Frame tubes are one category of commonly used tube systems. This type of system possesses a much stiffer outer tube than in tube-in-tube systems. The stiff exterior tube is achieved by the use of closely positioned columns joined using deep spandrel beams that are strongly connected. The Twin Towers in New York were built using the frame-tube structural system [62].

Braced tube structures, also known as trussed tube structures, are similar to the tube-in-tube structural system but have fewer exterior columns. The strength loss resulting from reduction in the number of columns in the exterior ring is compensated for by having braces between the columns. This is one reason that makes this type cost-effective for builders. However, a disadvantage of employing this structural system is that because of the availability of diagonal bracings throughout the building height exterior, window wall detailing and fixing of facades become complicated [58, 62, 63]. Figure 2-45 shows a building constructed with a braced tube structural system.



Figure 2-45 Braced-tube structural system incorporated building (John Hancock Centre in Chicago, USA) [2]

Bundle tube structural systems, as the name itself implies, involve bundles of many tube systems forming a much stronger single structural system. These are typically used when structure height increase makes a single tube system insufficient for resisting the lateral loadings imparted on the building [63]. The bundle tube system is akin to the cellular structure of bamboo or trees. An important advantage of bundled tube structures is that different tube systems can be incorporated for each tube in the bundle, and they can be ceased at any height level without scarification in structural integrity. This also provides greater flexibility for architects in terms of creating buildings of different shapes [60]. Figure 2-46 shows a building constructed with a bundled tube system.



Figure 2-46 Structure involving a bundle tube system—Sears Tower, Chicago, USA

<http://khan.princeton.edu/>, last accessed August 2020

2.3 Current Provisions for Selecting Structural Systems for Conventional Steel Buildings

Many lateral load-resisting systems used in practice were introduced in the previous section. However, it is imperative to understand the LFRS requirements for different structures. Khan inferred that different structural systems would have to be considered for different heights of structures. This conclusion was based on the fact that, when building height increases, a premium must be paid in the form of lateral forces caused by wind loads, and demands on the structural system dramatically increase [64].

Further, Khan inferred from his studies that when structure height exceeds 10 storeys, the design and stiffness are controlled more by lateral stiffness than strength, and an exponential increase in premium for height was reported with an increase in the number of storeys. Khan concluded that these observations were a result of the magnification of the total lateral deformation due to cantilever action and shear racking caused by the increased lateral loads with height, leading to a dramatic increase in the stiffness demands on the building [64].

Khan identified a hierarchy of structural systems with respect to relative effectiveness in resisting lateral loads for buildings [65], and created categorised charts for steel and concrete structures. Since the focus of this study is steel structures, the chart for steel structures is presented in Figure 2-47.

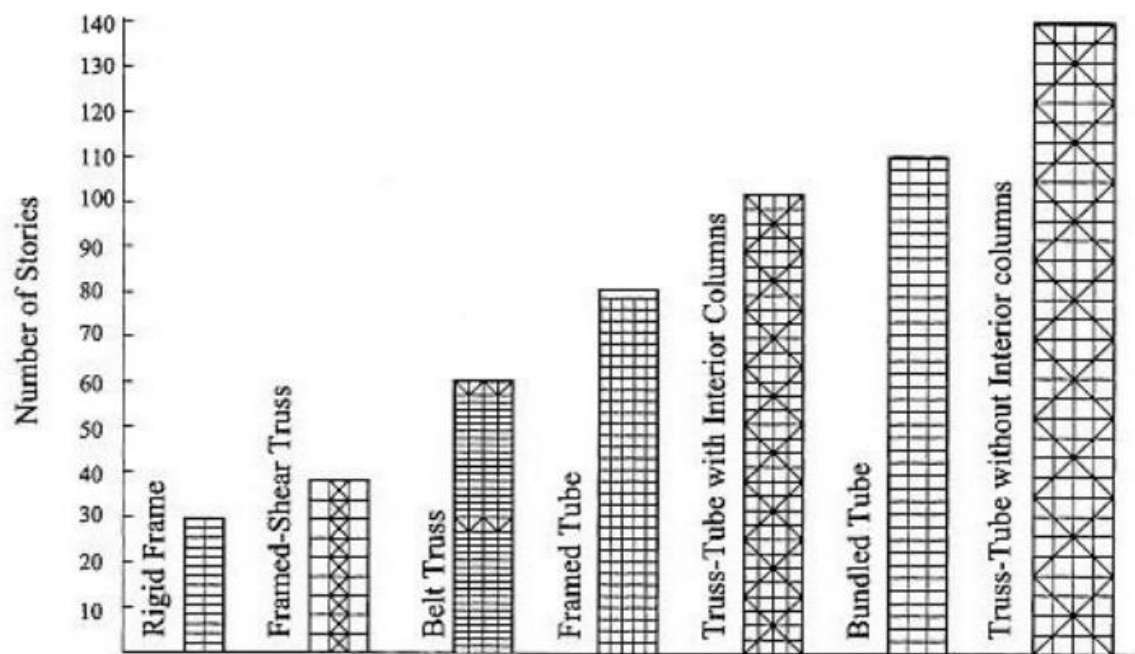


Figure 2-47 Classification of structural systems in steel buildings by Fazlur Khan [64, 65]

Years after the first classification system was produced by Khan, Ali presented a new classification system covering most of the available structures around the world. The classification was conducted based on the lateral force-resisting capacities of each structural system; primarily for two main types of lateral force-resisting system identified by the authors as interior and exterior structures. Categorisation relied on the distribution of the primary force-resisting system throughout the structure. Systems in which a significant component of the lateral force-resisting system is inside the structure are classified as interior structures; if a

significant component of the lateral force-resisting system is located at the building perimeter, it is classified as an exterior structure [66].

Figure 2-48 and Figure 2-49 illustrate the classification proposed for interior and exterior structures, respectively [66]. Further details about each system in each category can be found in Appendix A. In addition to the plots presented in Figure 2-47, 2-48 and Figure 2-49, many researchers have duplicated Khan's experience-based recommendations for the critical number of storeys for the effectiveness of lateral load resisting systems in their own charts. Some of these are presented in Figure 2-50 and Figure 2-51.

All these classifications, which were inspired and led by Khan, are based on substantial research on many buildings constructed in real life and analysed for their structural stability through FE and experimental analysis. *Based on these plots, it can be concluded that low- to mid-rise structures up to around 20–30 storeys do not require an external LFRS such as core or shear walls, which are costly for a project in terms of money and time.*

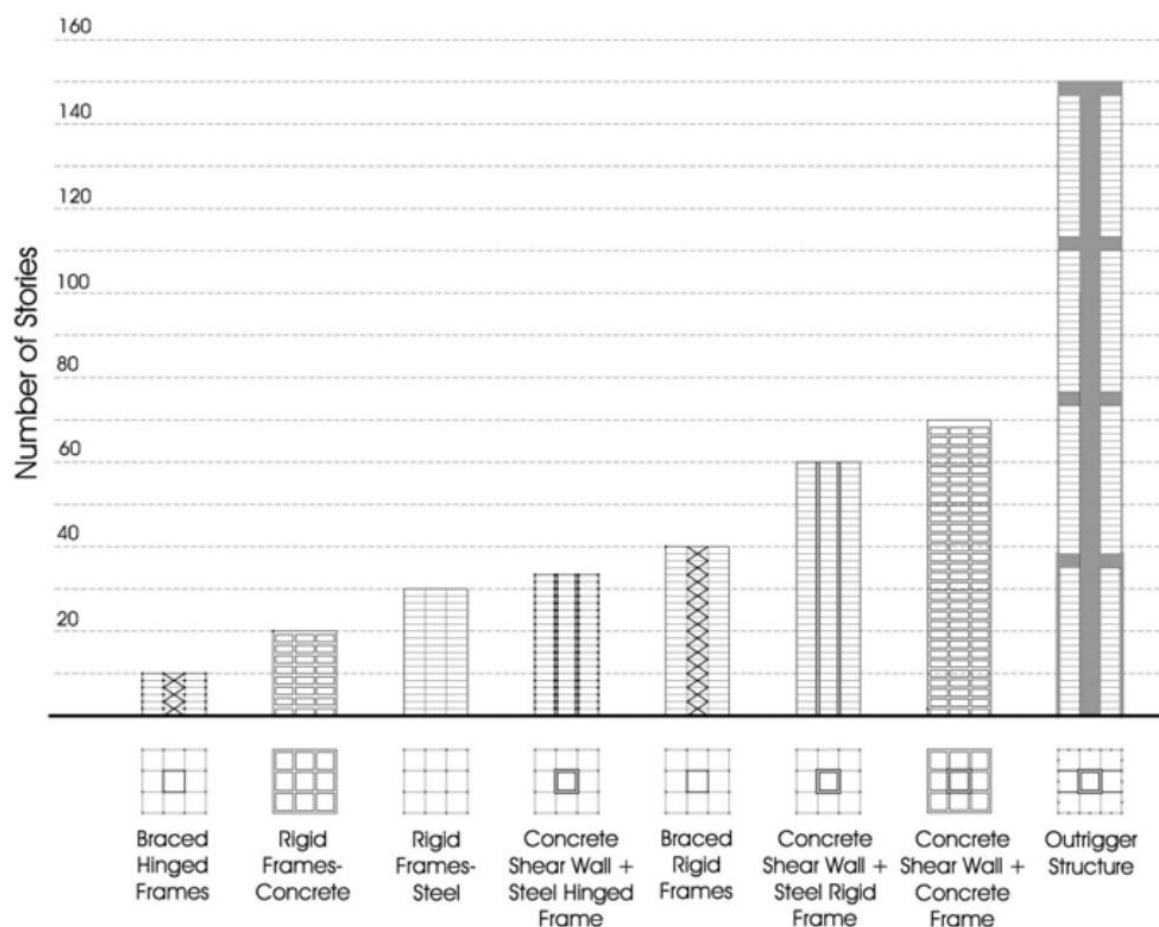


Figure 2-48 Interior structures [66]

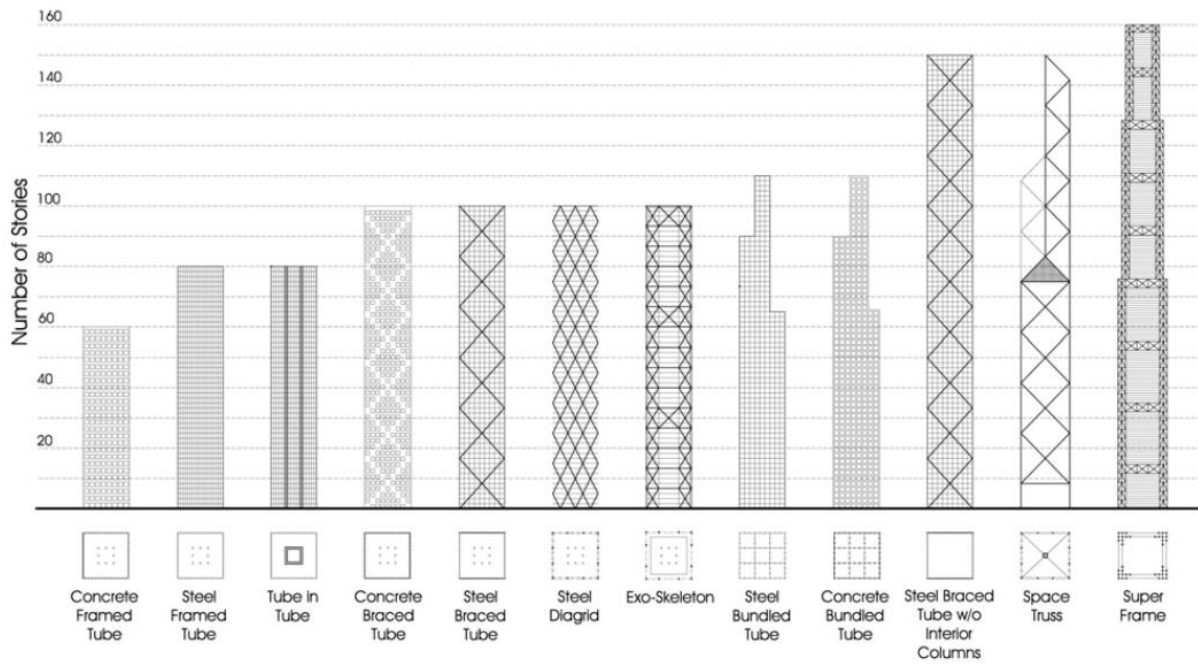


Figure 2-49 Exterior structures [66]

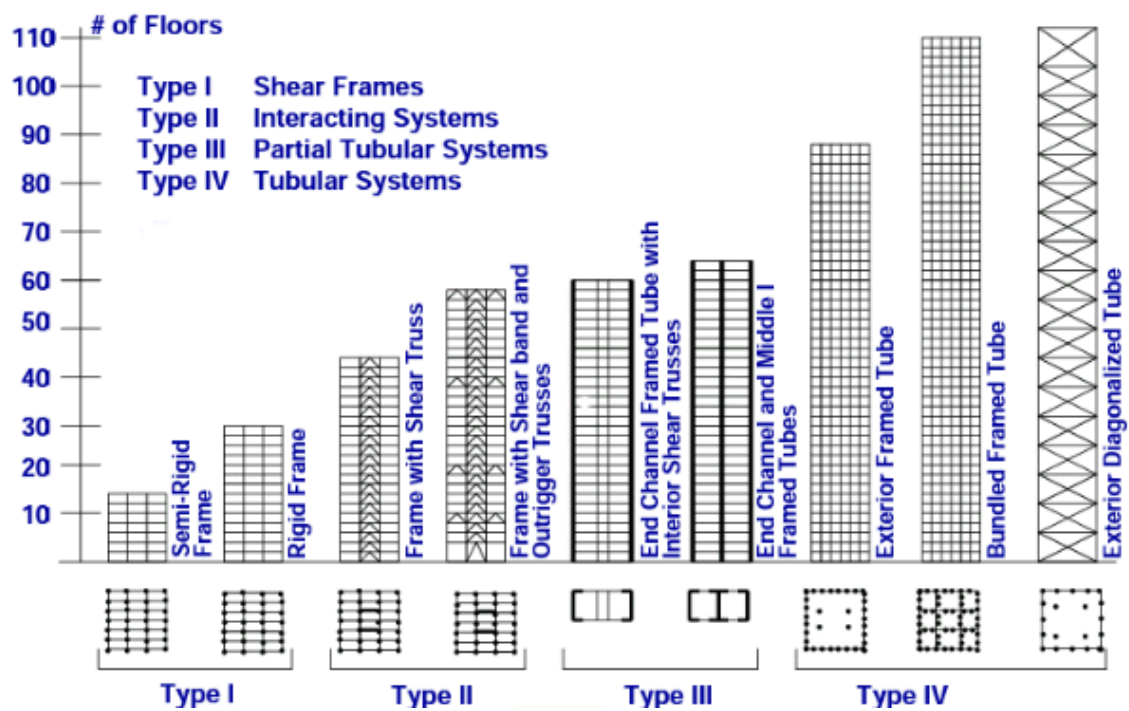


Figure 2-50 Khan's categorisation-CTBUH 1980 <https://en.wikipedia.org/wiki/Skyscraper>, last accessed August 2020

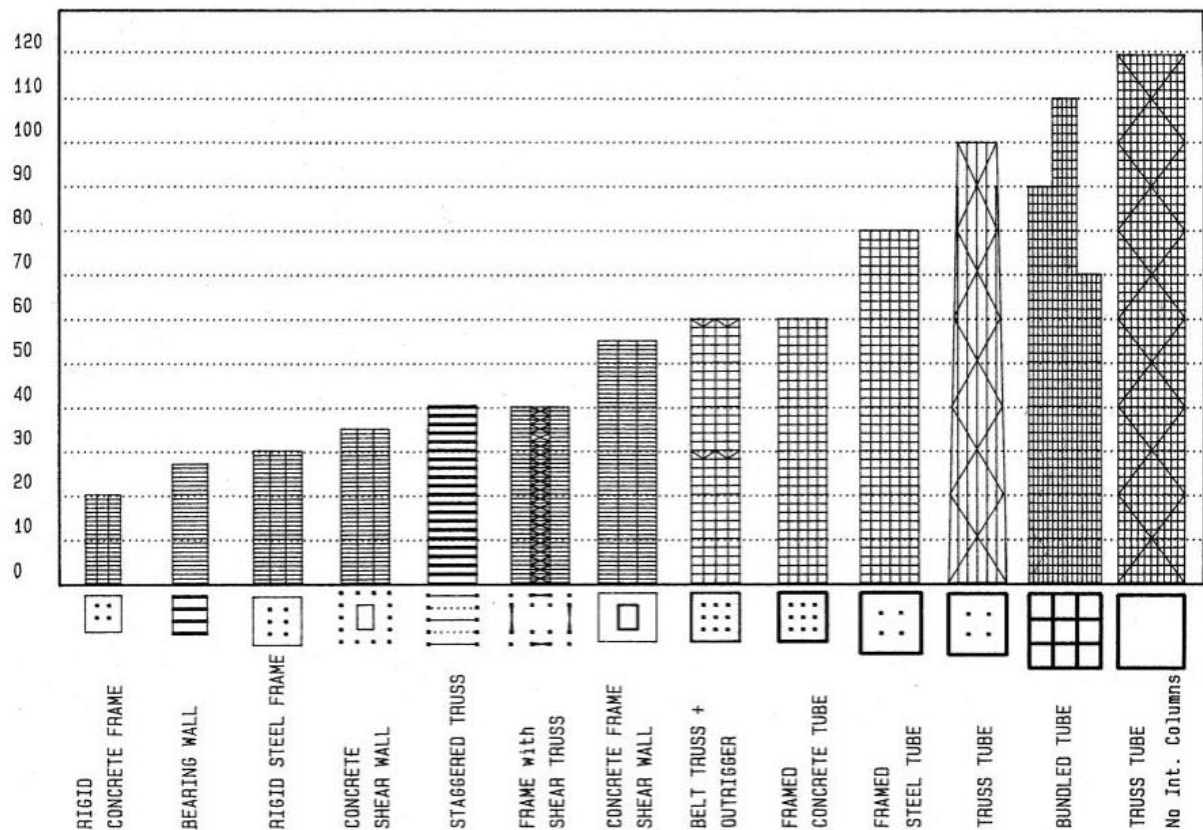


Figure 2-51 Interpretation of Fazlur Khan's structural system v. height relationship

<https://studfile.net/>, last accessed August 2020

2.4 Structural Systems in Modular Buildings

2.4.1 Current Modular Buildings

Although modular construction is an entirely different process to conventional construction in many aspects, such as off-site manufacturing, transport and assembly, the structural systems required to resist lateral loadings in modular buildings have been the same as those in conventional constructions. Many tall- to mid-rise modular structures rely entirely on a conventional LFRS to resist lateral loadings [11].

Figure 2-52 shows a nine-storey building constructed in Melbourne (One9 apartments). This building was erected with off-site manufactured prefabricated modules stacked and tied to a concrete core built on site. The core approach for lateral load resistance followed conventional construction approaches, even though prefabricated modules were used to complete the rest of the structure [12].



Figure 2-52 One9 Apartments, Melbourne, Australia [12]

Figure 2-53 presents a 25-storey modular building constructed in the UK. Like the One9 apartment building in Figure 2-52, this building was built around a concrete core. Prefabricated modules were stacked and attached to the concrete core [2].



Figure 2-53 A 25-storey modular building in Wolverhampton, UK [2]

A 32-storey modular structure constructed in Atlantic Yards in New York, USA consists primarily of a braced structural steel frame in which the modules were placed [2]. This type of modular construction is discussed in Section 2.1.3.1. In this type of modular building, the steel frame with or without braces governs the lateral and gravity load resistance. Figure 2-3 and Figure 2-4 provide some other examples of modular constructions of this nature.

SOHO apartments in Darwin (Figure 2-54) is a 32-storey modular building supported by a concrete core for lateral load transmission [12].



Figure 2-54 SOHO apartments, Darwin, Australia [12]

Podium structures, which are usually constructed of reinforced concrete or monolithic concrete structures on which modular structures are built, have also been used to support lateral and gravity loads coming onto modular buildings and transfer them to the foundations [11]. The SOHO apartment structure was also constructed on a podium and supported by a core at the centre of the building. Figure 2-55 shows two modular structures built on podiums.

Steel braces are also used in modular buildings. Most of the time, to reduce the work required on-site, braces are installed to the modules at the factory premises itself. However, when braces are installed inside modules, they do not intersect at a single working point, creating additional stresses at the vertical connections between the modules [24]. Unlike braced modules manufactured in a factory, as discussed previously, placement of modules inside a steel braced frame has been done in practice. For example, social housing in Tampere, Finland (Figure 2-56) was constructed using a tubular supporting framework with braces [67].

Diaphragm action of module floors plays a significant role in lateral load transmission in modular buildings. The lateral loadings imparted on the structure are transferred to the main lateral force-resisting system in the structure (e.g. core, shear wall, tube) through the inter-module connections [11, 14].

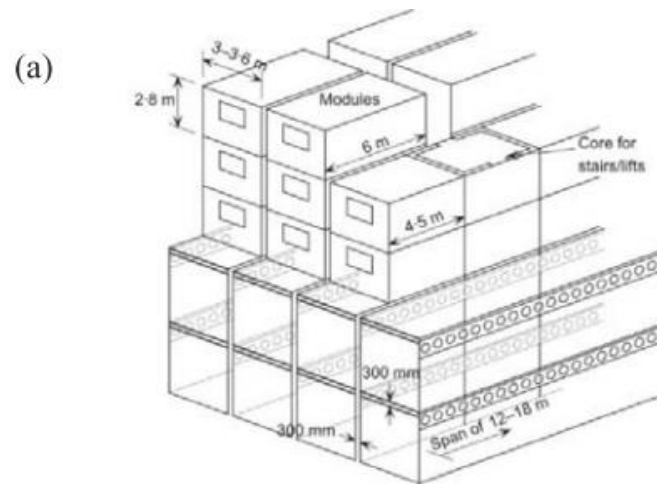


Figure 2-55 (a) A modular structure on a cellular steel beam podium and (b) a commercial-residential building with a podium and 1,400 modules [11]



Figure 2-56 Modular units used in a tubular framework in Tampere, Finland [67]

2.4.2 Fully Modular Buildings (Pure Modular Buildings)

One of the most sought-after advantages of modular construction is the ability to avoid wet construction on site. Fully modular construction will not have any significant on-site work other than the erection and assembly of modules to form the final building. In a fully modular system, the gravity and lateral load transmission take place through the inter-modular connections, and none of the external lateral load-resisting systems discussed in previous sections are required. Inclusion of an external LFRS such as core or shear walls will have a considerable effect on the construction efficiency of the structure [11, 12].

However, fully modular structures have not yet been completely realised in the industry. Most modular buildings (medium–high rise) built to date include some sort of lateral load-resisting mechanism that needs to be constructed in addition to the assembly of modules. However, since the current practice in modular construction is to rely on the conventional building design standards, techniques, and practices, transformation to fully modular is not hindered from any of them. There are no rules or regulations in the current practise requiring external lateral force resisting systems (LFRS) for structures built in low to moderate seismic regions. Hence, if the structure without any external LFRS can still satisfy the code requirements, the structure can exist (in a designer’s perspective). Given the enormous time and cost savings from eliminating any external LFRS, designers might soon opt to move towards pure modular structures while still designing the structure as per conventional design standards. Some researchers have already investigated this transition.

2.5 Research Towards Fully Modular Buildings

The full benefits of modular construction cannot be said to have been achieved until a fully modular structure is developed. A fully modular structure will require no work on-site other than stacking and connecting modules. For example, construction of a core or other external LFRS will not be required, and the modules together with their connections will be capable of resisting both lateral and gravity loadings. This type of structure is not impractical when the number of storeys is as low as two or three. However, for mid- to high-rise structures where lateral loadings have a considerable effect on the stability of the structure, elimination of an external LFRS to achieve a fully modular structure requires careful investigation. Surprisingly, little research has been conducted on the possibility of creating fully modular buildings.

The most accepted approach for investigating the feasibility of any structural system is to check whether the structure adheres to specified code requirements. However, adherence to all code requirements does not necessarily guarantee that a structure will perform safely under any type of loading, especially under dynamic loadings exceeding design limits. However, from the perspective of the economic aspects of construction, codes do not require a structure to be designed for ground motion that might be beyond a certain probability of exceedance. In general, code requirements are specified such that a structure to which code specifications are applied will perform in a safe manner even if that structure experiences loading beyond the design limit.

Modular construction is distinct from conventional building construction in its nature. Nevertheless, because of the relatively recent history of modular construction and the lack of knowledge of the complete behaviour of modular buildings from the elastic phase to collapse, design of modular buildings is still conducted according to conventional steel and concrete design codes. This might not be a concern within design limits, where both conventional and modular buildings might adhere to the code requirements and perform safely. However, if the design limit is exceeded, the code still provides some guarantee of safe behaviour of conventional buildings. But, it is highly doubtful that a modular structure will display safe behaviour in the face of the catastrophic modes of failure progression and energy dissipation that may occur in a modular building as a result of its construction nature. Studying such behaviour is one of the main aims of this thesis.

Gunawardena [12, 68] investigated the possibility of constructing mid-rise modular buildings that fall into the category of fully modular buildings. Their study is briefly reviewed below.

A new structural system called the ‘advanced corner-supported modular system’ was introduced by Gunawardena and colleagues [12, 68]. As far as lateral loads are concerned, this system is considered self-sustaining. Since the main feature of any such structure is going to be the absence of an external LFRS, authors developed modules with stiff wall elements placed around the building strategically to resist lateral loads effectively. In addition to the action of the module, the connections play a significant role in the transmission of lateral loads in this type of structure. Hence, the authors also studied an innovative connection to be used in this system. The elevator core action is provided through prefabricated modules, eliminating the requirement for it to be the central component of the LFRS.

An investigation of the effectiveness of the proposed structural system was carried out using a 20-storey medium-rise modular building, as shown in Figure 2-57.

Two types of modules, Type 1, with stiffer reinforced concrete walls of 100 mm thickness (Figure 2-58 (a)) and Type 2, lightweight wall panels (no reinforced concrete walls) (Figure 2-58 (b)), were used in each storey as shown in plan view in Figure 2-59.

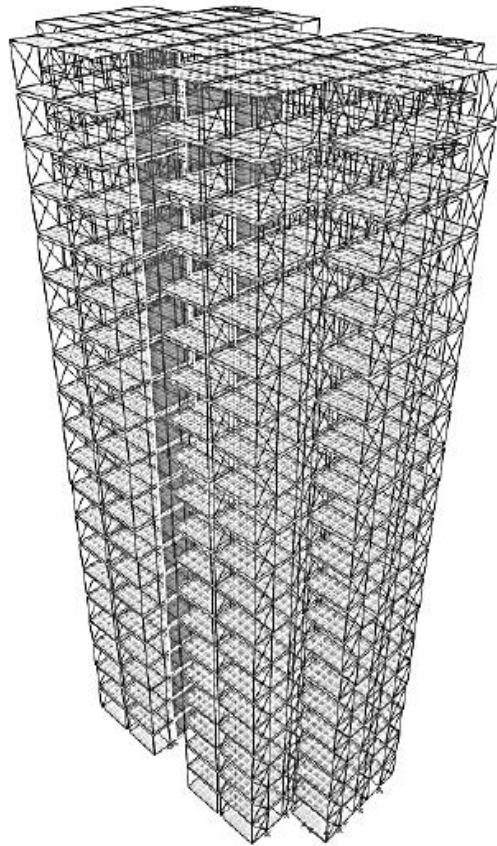


Figure 2-57 FE model of the 20-storey building considered [12]

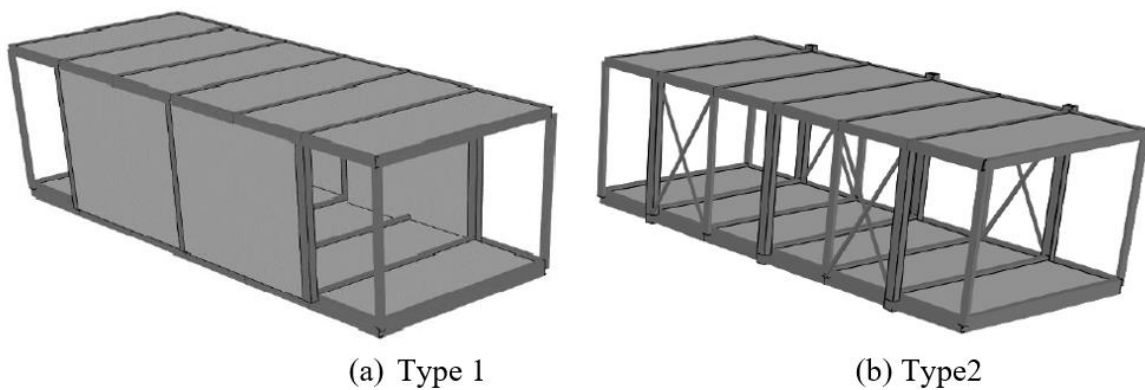


Figure 2-58 (a) Type 1 and (b) Type 2 modules used throughout the building [12]

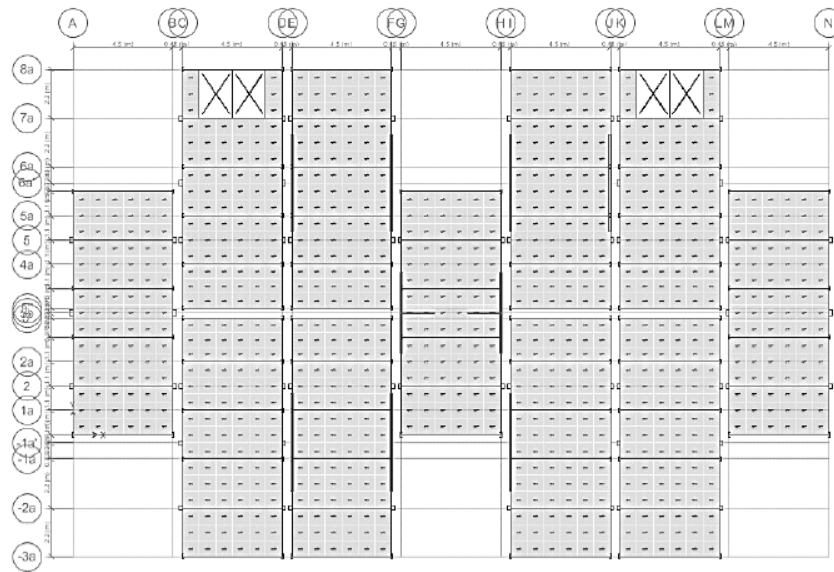


Figure 2-59 Plan view of one storey of the 20-storey building (Type 1 module walls are shown by dark lines) [12]

Initially, the structure was analysed for its modes of vibration, general structural stability, natural period and behaviour under lateral wind and seismic loadings. An earthquake response spectrum analysis was then conducted as per Australian seismic code, AS1170.4 [69], and a wind load was applied as per Australian wind code, AS1170.2 [70].

Further, to understand the behaviour of the structure under seismic events, nonlinear earthquake time history analyses were performed using ETABS under the six earthquakes listed in Table 2-2.

Table 2-2 Applied ground motions [12]

Record	Magnitude (Moment Magnitude Scale)	PGA	Duration
Chichi	7.63	0.278 g	90.0 s
Tabas	7.40	0.852 g	32.8 s
Dunzce	7.14	0.535 g	25.8 s
Kocaeli	7.40	0.312 g	21.1 s
Imperial Valley	6.90	0.519 g	40.0 s
Loma Prieta	7.10	0.371 g	60.0 s

Modal analysis results reported for the research are summarised in Table 2-3. The modal analysis results and lateral loading results were deemed satisfactory in demonstrating a safe performance of the structure adhering to code requirements.

Moreover, as shown in Table 2-4, the storey drifts in static response spectrum and wind analyses were within the 1.5% limit specified by AS1170.4 [69].

Table 2-3 Modal analysis results [12]

Mode	Period (s)	Frequency (Hz)	UX	UY	RZ
1	3.75	0.27	73.2%	0.0%	0.0%
2	2.39	0.42	16.7%	0.2%	67.4%
3	1.79	0.56	6.9%	60.9%	0.1%

UX: Participation of the modal mass to translational movement in X direction
 UY: Participation of the modal mass to translational movement in Y direction
 RZ: Participation of the modal mass to rotational movement about Z (vertical) axis

Table 2-4 Results under lateral loadings [12]

Load Case	Base Shear (kN)	Maximum storey drift
Wind Y (Static)	5494	0.13%
Spec Y (Dynamic)	1232	1.20%

Table 2-5 summarises the results from the nonlinear THA conducted. The maximum drift values observed with and without consideration of geometric nonlinearities were able to satisfy the drift limits specified in AS1170.4[69].

Table 2-5 Storey drift results under nonlinear time-history analyses [12]

Ground motion record	Maximum Drift Without Considering P-Δ Effects	Maximum Drift Considering P-Δ Effects
Chichi	0.62%	1.69%
Tabas	0.63%	1.34%
Dunzce	1.24%	1.55%
Kocaeli	0.85%	0.99%
Imperial Valley	0.75%	1.43%
Loma Prieta	1.15%	1.17%

The advanced corner-supported modular system introduced by Gunawardena and colleagues is an essential step towards pure medium-rise modular buildings. As discussed above, the preliminary analysis demonstrated the structural feasibility of the selected structure.

In addition to the above research outcomes, the facts presented in Section 2.3 regarding classification of an LFRS according to the height of the structure (which will guarantee efficient performance of the LFRS) suggest that low- to mid-rise structures do not require an external LFRS such as core or shear walls. The classification (in Section 2.3), in general, implies that for low- to mid-rise structures, the frame action along with other elements such as braces is sufficient to withstand lateral loadings such as wind and earthquake. This represents an important step towards the construction of mid-rise pure modular buildings, which might eventually be improved to enable building of high-rise pure modular structures.

2.6 Behaviour of Bolted Connections

As discussed in the literature review of the various modular building connections available, such connections are mostly made up of bolts and splices of various sizes. Hence, the failure modes expected do not vary significantly from those of a conventional steel connection, and it is vital to study the behaviour of bolted connections under cyclic and monotonic loadings.

Many authors have reviewed the behaviour of bolted connections [71-74], including some common ways of failure of bolted joints, as follows [75]:

- prying of bolts
- brittle failure
- shear failure/tearing
- bearing
- combination of the above actions.

2.6.1 Prying of Bolts

This process results from an increase in the tensile load on the bolts in a bolted connection due to deformation of the connected parts in the connection (Figure 2-60) [42]. This action may lead the bolt to fail in tension.

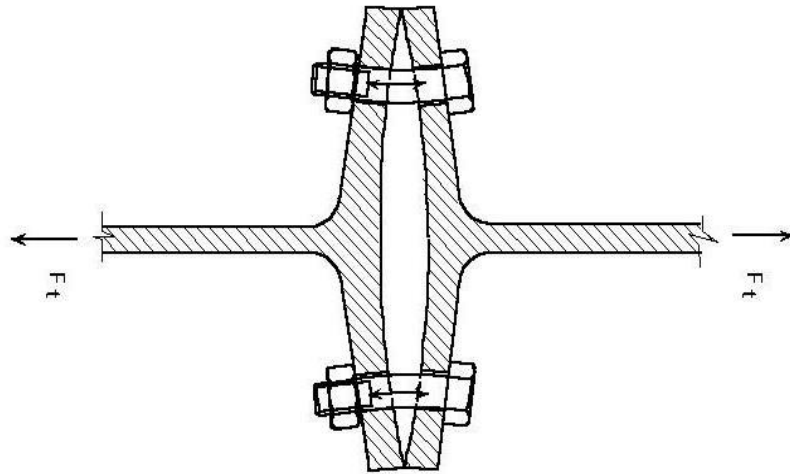


Figure 2-60 Prying action [42]

2.6.2 Bearing Failure

The bearing property concerned with a bolted connection is the force acting on a hole through which the bolt is inserted. Bearing failure is the failure of the connection due to the elongation of this hole under bearing forces (Figure 2-61) [75].

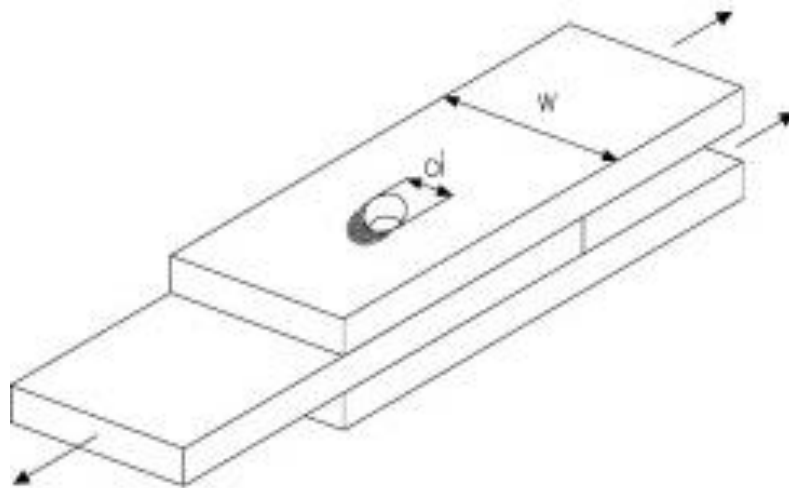


Figure 2-61 Bearing in a bolted connection [75]

2.6.3 Shear Failure

Shear failure in a bolted connection can be expressed in two ways: shear failure of the bolts and shearing of the material around the bolt hole (tearing). If the bolt shank shear strength is less than the applied load, the bolt may fail in shear. However, if the connected plates have a lower shear strength value, they may fail, causing tear out of the plates (Figure 2-62) [75].

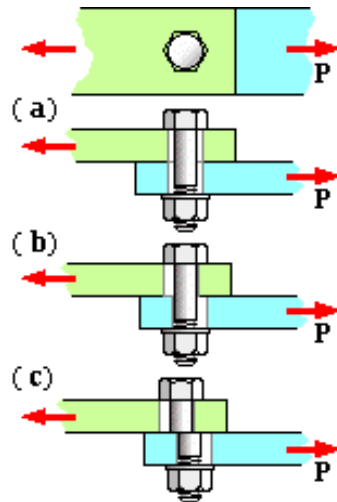


Figure 2-62 Shear failure in bolted connections [75]

2.6.4 Brittle Failure

Brittle failure is a common failure that can be observed in steel connections. In modular buildings, being an assembly of a large number of steel connections, there is a very high chance of these connections undergoing brittle failure. Brittle failure usually occurs due to inelasticity concentration at a connection (Figure 2-63).



Figure 2-63 Brittle failure at a connection [76]

Avoiding inelasticity concentration is one possible precaution that can be taken to prevent brittle failure. Two possible approaches can be used to prevent inelasticity concentration at a connection [76-81]

2.6.4.1 Evenly Distributed Inelasticity

This is the most preferred method of avoiding inelasticity concentration. The inelasticity is distributed throughout the building, and thus no special attention is required to enhance the strength of one element or joint of a building [82].

2.6.4.2 Concentrate Inelasticity at One Location

Although this is not a commonly preferred method, it has several advantages over the previous method. It requires only a particular part of the building set to take the inelasticity concentration. This is the concept behind a structural fuse [83] that will go off during extreme loadings. If a fuse fails, it can be replaced with a new fuse. Some other approaches used to concentrate inelasticity in one specific place are discussed below.

2.6.4.2.1 Concentrate the inelasticity at the connection

A bolted flange plate moment connection is an example of this approach (Figure 2-64), in which the capacity of the connection is increased by setting longer yielding paths. The yielding path will be in the sequence of:

1. yielding of the beam area just after the bolted connection (1)
2. slipping of the bolts (2)
3. secondary yielding of the panel area (3).
4. limited yield of the column flange (4).

This approach will ensure that the capacity of the connection is sufficient to withstand large inelasticity concentrations.

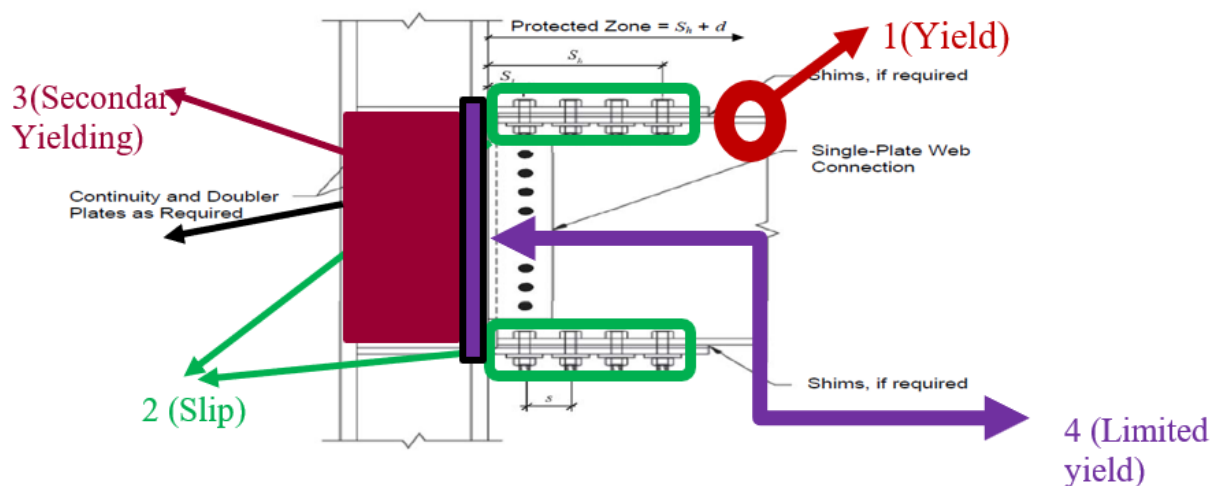


Figure 2-64 Bolted flange plate moment connection [84]

2.6.4.2.2 Concentrate the inelasticity at a location away from the connection [84]

A few examples of this approach are discussed below,

1. Use of a tapered beam section [76]

In this approach, part of the beam is tapered to create a less strong part of the beam (at the taper), allowing inelasticity to be concentrated at that site (Figure 2-65).

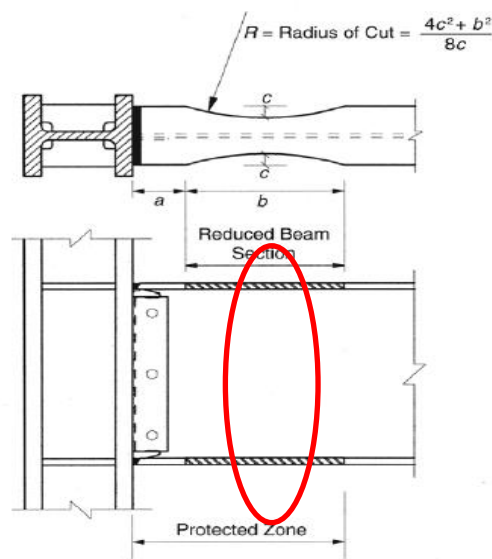


Figure 2-65 Reduced beam section approach [84]

2. High-strength steel bracket connections [79]

In this approach, a high-strength steel bracket is used to substantially strengthen the connection and move the inelasticity concentration away from the connection area (Figure 2-66).

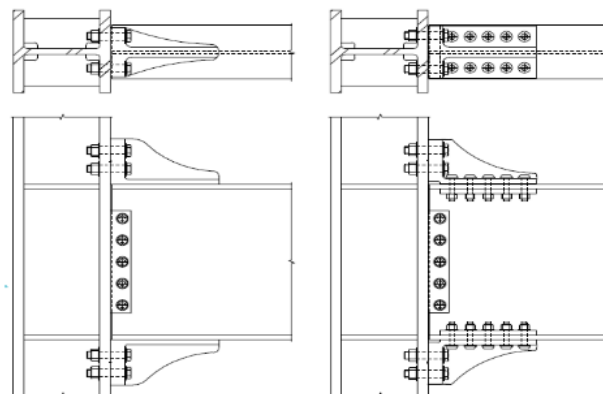


Figure 2-66 Steel bracket connection [84]

2.6.5 Shear and Tension Failure

Pure shear failure is discussed under section 2.6.3 above. However, pure shear failure is not common in a structure [80]. Shear failure usually occurs together with tension and bearing induced in the bolts (Figure 2-67 Typical combined failure).

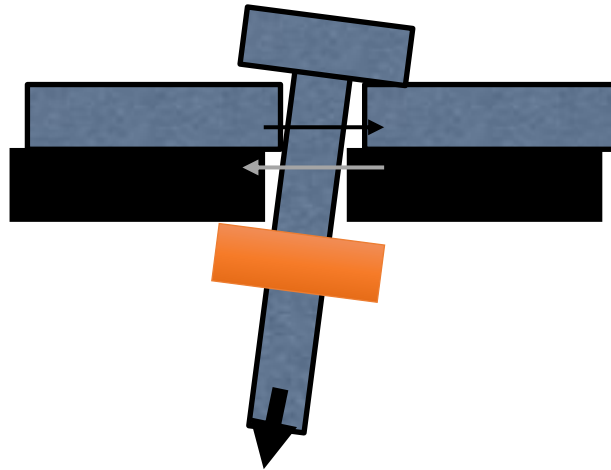


Figure 2-67 Typical combined failure

This mode of failure under combined shear, tension and bearing is highly critical in bolted splice joints. Most of the connections involved in modular buildings are composed of bolted splice joints, which are subjected to significant cyclic loadings during earthquakes and wind loadings. The behaviour of a bolted splice joint under cyclic loading can be explained as follows. After a few cycles, bolts slip and slip failure occurs (Figure 2-68 Bolted splice joint after a few cycles):

1. This slip failure will lead to a loss of pre-tension in the bolted connection.
2. The bolt will no longer be in the clamped condition.
3. The bolt will be subjected to tilting due to the clearance.
4. The tilted bolt will experience a tensile loading borne on the plates.
5. After a few cycles, the bolt will become elongated.
6. Finally, the bolt may fail in tension and shear.

This behaviour of bolted splice joint connections is not widely discussed in the literature. Although there have been many studies on the hysteretic behaviour of bolted connections [73, 79, 81, 85, 86], in-depth research on the real behaviour under combined shear and tension loading, and the danger involved in a connection subjected to such conditions, is lacking in the literature.

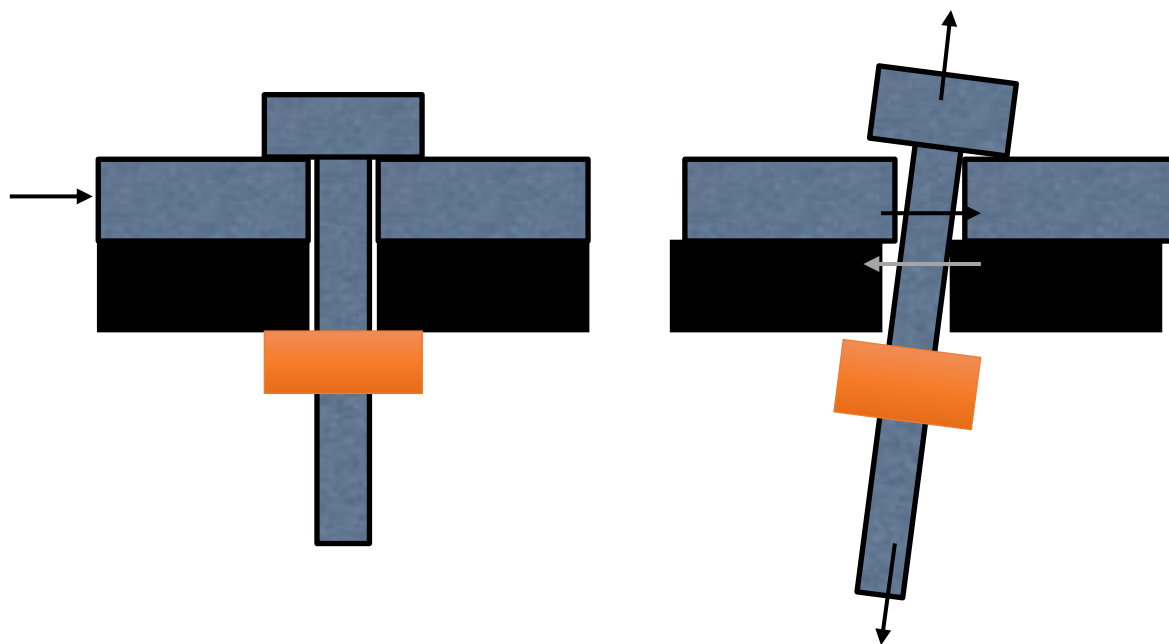


Figure 2-68 Bolted splice joint after a few cycles

Further, due to this sort of response, bolted connections lack sufficient ductility, and tend to act in a brittle manner. Hence, in a modular building, where the connections are the most critical component and one of the hot spots for inelasticity concentration, employment of bolted connections might lead to a catastrophe. If a critical connection in one storey fails, this might immediately put the other connections at risk because of the higher redistributed inelasticity. The brittle nature of failure involved in bolted connections will thus result in all the connections in the critical storey failing and opening up in an unzipping fashion, resulting in brittle failure of the whole structure.

2.7 Seismic Analysis

Seismic analysis can be classified into different methods depending on many factors. One of the means of classifying seismic analysis is based on whether it is static or dynamic. It can also be categorised based on the principles governing the limiting criteria as force-based or displacement-based [12]. The force-based approach sticks to the generally practised limit state analysis common to many design standards around the world. Displacement-based analysis is mainly based on the performance aspect of the structure (target displacements or drifts) under a given earthquake. Thus, performance-based seismic design is another name for displacement-based earthquake design [12].

2.7.1 Static Pushover Analysis and the Capacity Spectrum Method

2.7.1.1 Pushover Analysis

More realistic performance of a structure under seismic events can be obtained using nonlinear models. Even so, nonlinear dynamic analysis requires extensive computational effort. The computational cost can be considerably reduced when the computation is carried out using a nonlinear static analysis together with a technique to calculate the variation in maximum deformation or any other damage quantity of interest under the nonlinear static analysis [85].

Pushover analysis is a technique that determines the deformation of structures under dynamic events with the aid of simplified nonlinear methods [86]. This analysis provides an output in the form of a load-displacement plot, for the structure starting from rest and going until failure of the structure. The structure is pushed from rest until failure under the action of a horizontal force representing the equivalent static load of a specific mode of vibration of the structure. The applied horizontal force is usually considered as the total base shear of the structure. The measured damage is taken as the roof deformation since this produces the worst deformation [87].

Pushover analysis can be performed as either displacement-controlled or force-controlled [88-90]. The force-controlled method relies on increasing the force in pre-defined steps; the resulting displacement is taken as the dependent variable. In the displacement-controlled method, the displacement of the structure is increased in incremental steps, and the resulting force is measured at some point (generally base shear). Most of the time, the displacement-controlled method is preferred over force-controlled methods because it allows the analysis to be undertaken up to the target displacement [12].

2.7.1.2 Capacity Spectrum Method

This method was originally advanced by Freeman et al. [91] as a technique for evaluating the seismic performance of structures. After that, the Applied Technology Council (ATC) 40 [92] recommended this approach for seismic evaluation and design and retrofitting of structures.

The capacity spectrum method assumes that the response of an elastic system with reduced stiffness and increased damping can be used to evaluate the displacement response of a nonlinear system. In this approach, the earthquake demand expressed in the form of a response

spectrum is compared against the capacity of the structure represented in the form of a pushover curve. The capacity curve and the demand curve are then plotted in a single plot that describes the behaviour of spectral acceleration against spectral displacement. The capacity spectrum is obtained by superimposing the earthquake demand curve onto the pushover curve, which is converted to a capacity curve by converting the base shear and top displacement to spectral acceleration and spectral displacement, respectively.

Figure 2-69 shows a diagrammatic representation of the steps involved in converting the earthquake response spectrum and pushover curve to ADRS format to produce demand and capacity diagrams.

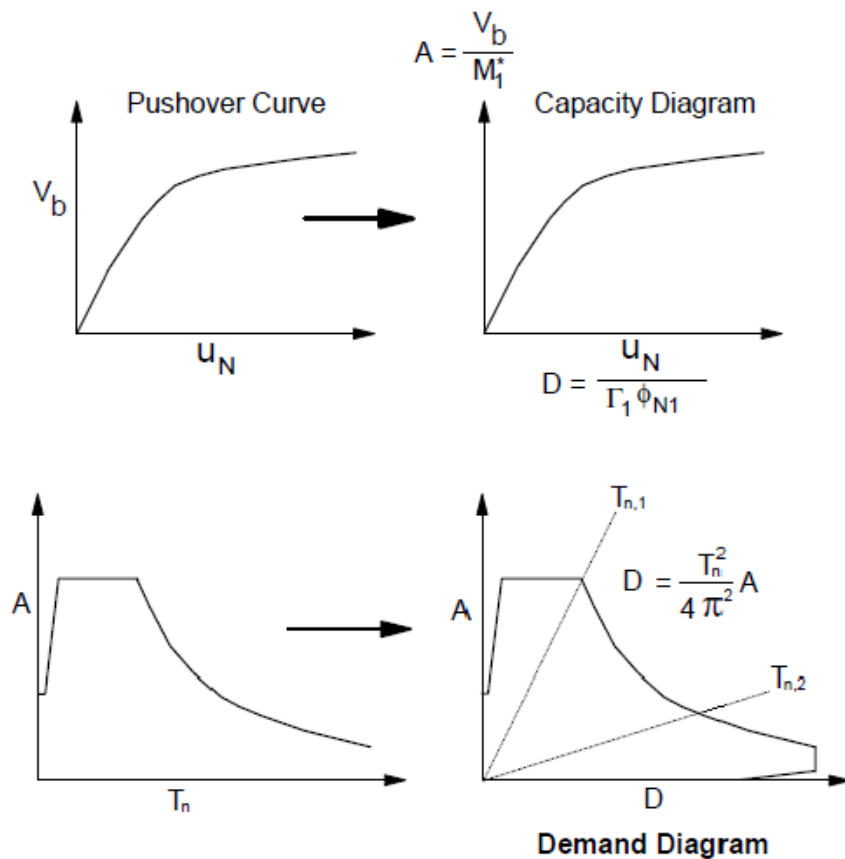


Figure 2-69 Graphical representation of the conversion of a pushover curve to an ADRS diagram [12]

Figure 2-70 illustrates the approach followed in determining the performance point. In terms of interpreting the performance of a structure under a given seismic event, it can be concluded that if the structure performs satisfactorily within the area covered by the ADRS diagram, the structure will be safe in the event of that earthquake or an earthquake of similar intensity.

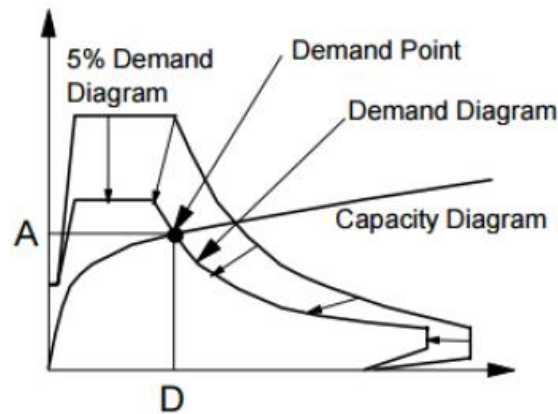


Figure 2-70 Identifying the performance point using the capacity spectrum [12]

However, the pushover analysis and capacity spectrum methods cannot be considered techniques to obtain the detailed performance of a structure under dynamic events. These methods simply serve as preliminary analysis techniques that can be employed to assess the performance of a structure against general code requirements and to obtain a narrow understanding of a structure's expected performance under real dynamic events. Later chapters discuss in detail experimental observations on a modular structure during shaking table experiments. In these experiments, it was observed that the modular building overturned catastrophically. Yet, analyses such as the pushover and capacity spectrum methods are not capable of identifying such a potential failure. Hence, the best approach to studying a structure's performance under dynamic events is to conduct dynamic THA, discussed in detail in the following section.

2.7.2 Dynamic Time History Analysis

2.7.2.1 Background

The dynamic THA is one of the most precise and descriptive approaches for obtaining the performance of a structure under a dynamic event. In this approach, the response of the structure is calculated for every time step of the ground motion. THA can be conducted on nonlinear and linear models [12].

Linear dynamic THA is conducted on linear models that do not take into account any nonlinearity. An example of this type of analysis is the response spectrum analysis. Many mode shapes are taken into consideration in this method. Depending on the modal frequency and modal mass, for each mode, the corresponding response is evaluated from the design spectrum.

The final response of the structure is calculated by combining the response for each mode using modal combination methods [89, 90].

Nonlinear dynamic THA is one of the most accurate methods of analysing the behaviour of a structure under dynamic events and estimating its response under a designed intensity. The ground motions used for analysis are usually obtained from records of past earthquakes. Researchers have also generated synthetic accelerograms using various techniques, which can be used as input ground motions for evaluation of dynamic behaviour of structures [93]. During the analysis of a structure under a design ground motion for a specific region, ground motions chosen from databases of past earthquakes need to be scaled to the design response spectrum corresponding to a particular area for a particular return period. Synthetic accelerograms come handy for this purpose as they can be generated to correspond to a chosen response spectrum for a studied region without the need to scale them [94-96].

Most of the time, dynamic THA is conducted using FE analysis packages like SAP2000, OpenSees and Seismostruct. These applications have numerical integration methods that solve the incorporated equation of motion. Apart from direct use of these FE packages, there are instances where engineers/researchers will have to manually solve the equation of motion with the aid of computer programs such as MATLAB [97, 98]. Numerical integration is employed in the latter part of this research to solve for the response of a rocking and sliding rigid body under a dynamic loading. Thus, a brief introduction to numerical integration methods is provided in the following sections.

2.7.2.2 Numerical Evaluation of Dynamic Response

The most cumbersome aspect of dynamic THA is solving the equation of motion according to whether the analysis considered is linear or nonlinear. Many numerical integration methods are used to solve the equation of motion. If the considered Single Degree Of Freedom model (SDOF) is nonlinear, the analytical solution to the equation of motion under a ground acceleration becomes difficult [99]. These nonlinear equations can be handled with numerical time-stepping methods for differential equations.

Many methods have been developed to solve different types of differential equations arising from these equations of motion under excitation [100]. Some such methods employed to solve differential equations of motion are discussed in Appendix B.

Apart from the methods discussed in Appendix B, many methods, including Newmark's method are widely used in FE software to compute the dynamic response of structures. A complete review of available numerical integration methods is beyond the scope of this study. However, some of the methods discussed above are employed in this study to simulate the dynamic behaviour of rocking structures.

2.8 Incremental Dynamic Analysis

IDA is a solution to an important issue in performance-based earthquake engineering; that is, how to evaluate the performance of a structure under increasing seismic intensities; the mean annual frequency of exceedance of a specific damage index; or a particular limit state capacity [101]. When performing IDA, a suite of ground motions, each scaled to different intensities starting with lower intensities and moving up to intensities causing global dynamic instability, are applied to a structure [102].

Performing an IDA involves several essential steps, of which the first requires an appropriate nonlinear model. A suite of ground motions is then selected, scaling levels are chosen, and the analysis is run. The results are then post-processed [103-106] and usually expressed in the form of charts with the damage measure (DM) and intensity measure (IM) as the dependent and independent variable, respectively.

Since the results are expressed against a DM and IM, the choice of appropriate measures plays a vital role in conveying the IDA results meaningfully. The IM is a measure of the strength of the ground motion recorded on the structure. Many parameters that can be used as ground IM have been proposed. However, an essential property of such a measure is the ability to be scaled. For example, duration, moment magnitude or modified Mercalli Intensity (MMI) are all measures of the intensity of an earthquake. But, as they cannot be scaled to represent variation in the intensity of ground motion, they do not qualify as appropriate intensity representation parameters in an IDA. Some commonly used examples of scalable IMs are peak ground velocity, peak ground acceleration (PGA) and spectral acceleration at the structure's first mode period ($S_{a(T_1, 5\%)}$) [107]. Each of these IMs has its own limitations and criticisms in regard to current practice. Employing of PGA as an IM is not recommended as it is not a direct representation of the seismic demand on the structure [97]. Most studies have used spectral acceleration at the first mode period of the structure as their IM [97].

The choice of an appropriate DM is another crucial step in an effective IDA. In simple terms, a damage parameter is something that can be directly observed or deduced from the observations of the nonlinear dynamic THA. There are ample choices in selecting a DM, such as maximum base shear, peak storey drifts, peak storey ductility, node rotations, maximum roof displacement, global cumulative hysteretic energy, and etc. Nevertheless, selecting an appropriate DM depends on the structure analysed and the purpose of the analysis. For example, if the primary aim of a study is to evaluate the damage to non-structural contents in a structure, an obvious choice would be the peak floor acceleration [107]. Conversely, if one of the aims of the study is to understand the structural damage to the frame, maximum inter-storey drift or maximum roof displacement would be a suitable choice.

The choice of ground motion is an equally important aspect of conducting IDA. The accelerograms employed are generally selected to match with a target elastic response spectrum of interest or to represent earthquake scenarios that control the site hazard [97]. However, real accelerograms that match specific characteristics are not always easy to find. This has forced researchers to move towards manipulation of accelerograms to match the required situation. Scaling is one approach to generating accelerograms with the required characteristics. However, simulation of accelerograms using scaling approaches has proven to misrepresent the structure response, as the intensity of ground motion and frequency content are not independent [108]. Researchers have used synthetic accelerograms generated using various computer software for conducting IDA. Hence, instead of generating many ground motions or choosing a broad suite of ground motions from a ground motion database, computer programs that can generate ground motions are employed to generate an ensemble of ground motions, varying the source–site distance to achieve different intensity values for a specific moment magnitude value [97]. Endurance time excitation functions (ETEFs) use a similar approach. Still, they are unique in that they use just a single pulse that will cover a range of intensities proportional to a specific target spectrum. Further details about ETEFs are provided in Section 2.9.

IDA results are beneficial in terms of understanding the behaviour of a structure from rest through to the onset of collapse [107]. However, conducting IDA can be very time and resource-consuming because of the amount of work and computational effort involved [109]. Moreover, although IDA can be conducted on computer models, it is practically impossible to carry out an IDA on an experimental model because of the repetitive nature of ground motion application involved [109]. Single loading protocols come in handy for purposes that involve

testing the behaviour of a building under increasing seismic intensities. Loading protocols have been commonly used to test components of structures such as connections but not to study the behaviour of a building under increasing seismic intensities. ETEFs are similar to loading protocols and are used to study the behaviour of structures under increasing seismic intensities [110-114]. This concept is discussed in detail in Section 2.9 below.

2.9 Endurance Time History Analysis

2.9.1 Background and Concept

Endurance Time History Analysis (ETHA), as the name implies, is a THA carried out using ETEF to test the endurance of a structure. This is analogous to exercise tests carried out on human beings in medical science. Similarly, ETEFs are loading protocols that include features of real ground motion and increase its intensity gradually over time [112].

The concept of ETHA can be explained with reference to Figure 2-71. Assume that three structures A, B and C are undergoing intensifying dynamic loading on a shaking table.

In the beginning, at low-intensity levels, all structures remain stable and intact. As the intensity of shaking increases, when the structures experience seismic events exceeding their capacity, structural failure develops and structures collapse. Given the failure of structure A and then structure B and finally structure C, as shown in Figure 2-71, it can be concluded that structure A (which had the least endurance) has the worst performance and structure B (which had the highest endurance) has the best performance under the applied loading.

The results could also be interpreted in terms of seismic resilience of the structures: structure B is the best performing in terms of seismic collapse resistance, and structure A is the worst [112, 115].

The results of this hypothetical experiment can be interpreted in terms of response v. intensity plot similar to an IDA curve, as shown in Figure 2-72.

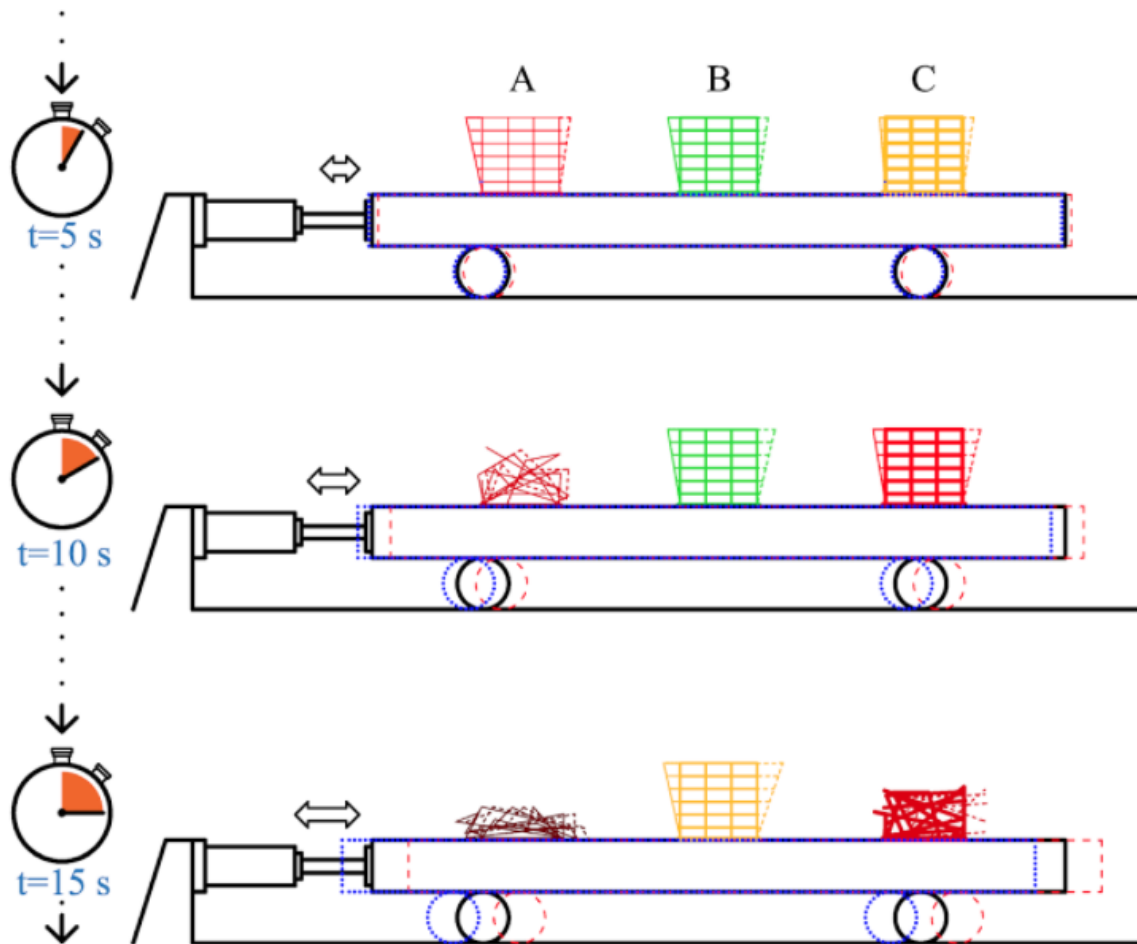


Figure 2-71 A hypothetical shaking table test [115]

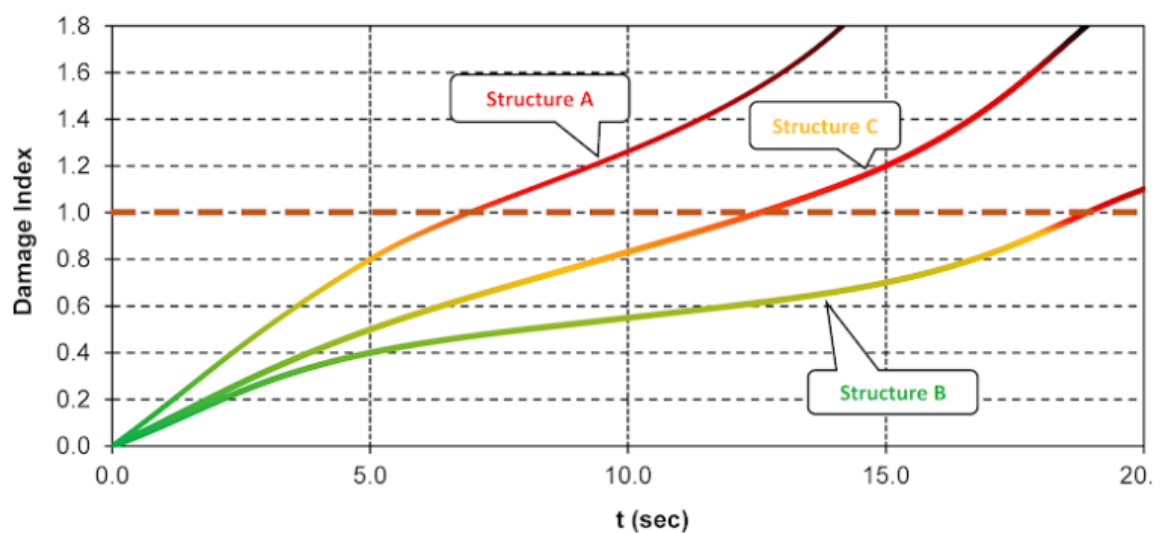


Figure 2-72 Schematic response curve for structures subjected to intensifying dynamic loads [115]

Time is an indication of the intensity of seismic events because intensity increases with time. In practice, for meaningful result interpretation, time can be mapped onto a more convenient IM such as PGA, hazard return period or other IM of interest. The response can be considered an indication of the damage the structure has experienced. The response parameter (damage parameter) can be a relevant simple or complicated structural response, or a function of a structural response such as inter-storey drift ratio, maximum displacement, stress, plastic rotation, damage index or damage costs [109, 115-121]. As shown in Figure 2-72, the results from an ETHA can be expressed in the form of damage v. intensity plot (the time axis can be converted to intensity, which is proportional to time), which is the same method used to express results from an IDA. Employing ETEFs instead of conventional IDA methods reduces the cumbersomeness of the computational processing involved with an IDA. However, the accuracy of the computed results depends on the chosen ETEF; that is, how closely the ETEF represents real seismic conditions [114].

Successful implementation of the ETHA concept is dependent on the characteristics of the applied loading function. Many paths have been followed to produce working intensifying dynamic load functions, among which the concept of the response spectrum has been found to be a very effective strategy for this purpose. A response spectrum strongly reflects the essential characteristics of earthquake time histories; that is, the frequency content and intensity. In most structures, a similar maximum response can be expected under two dynamic excitations with almost identical response spectra [115]. Spectral intensities are used to define seismic load requirements in nearly all current seismic design codes. Preliminary ETEFs have been developed, starting by concentrating on spectral characteristics of excitation functions. The maximum response of a structure under an applied loading is expected to match the response of a particular response spectrum of interest if the response spectrum of the applied loading matches that of the response spectrum of interest. This target spectrum can be in the form of the average spectra from a set of ground motions, design spectra or other spectra of interest.

This has served as a good starting point for researchers to produce useful ETEFs. Initial ETEFs have been generated by considering seismic code-based design spectra as a template and attempting to produce an intensifying excitation function that has the property of producing a response spectrum that tends to remain proportional to this target spectrum at all times [122-125]. One such typical ETEF in the form of ground acceleration is shown in Figure 2-73 and response spectra produced by the ETEF are shown in Figure 2-74.

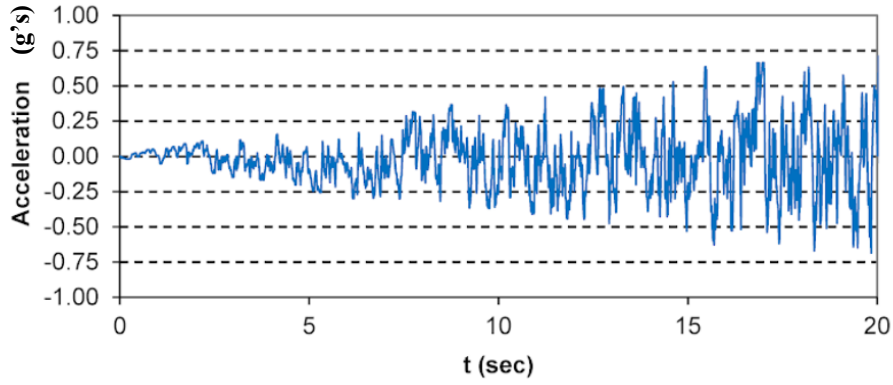


Figure 2-73 A typical ETEF generated [115]

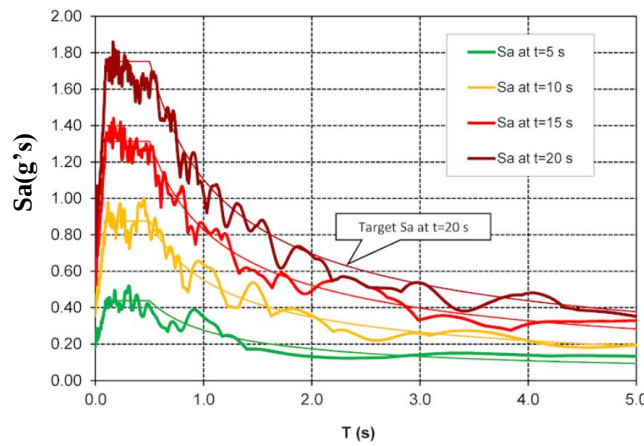


Figure 2-74 Response spectra corresponding to the generated ETEF shown in Figure 2 74 [115]

Optimisation techniques have been employed to generate ETEFs with response spectra that intensify with time, proportional to a chosen target response spectrum (Eq 2-1). This implies that the response spectrum for any time stretch of the ETEF from $t_0 = 0$ to $t_1 = t$ is proportional to that of the chosen target spectrum with a magnification factor proportional to time (t). In some studies conducted before, the problem has been considered as an unconstrained optimisation problem in the time domain, as shown below [112]:

$$\text{Minimise } F(a_g) = \int_0^{T_{\max} t_{\max}} \int_0^2 ([S_a(T, t) - S_{aT}(T, t)]^2 + \alpha [S_u(T, t) - S_{uT}(T, t)]^2) dt dT \quad \text{Eq 2-1}$$

Where a_g is the Endurance Time (ET) acceleration function being sought, $S_{aT}(T, t)$ and $S_{uT}(T, t)$ are the target acceleration response and displacement response at time t , respectively. $S_a(T, t)$ and $S_u(T, t)$ are the acceleration response and displacement response of the acceleration function at time t respectively. α is a weight parameter that is generally set to 1; and T is the period of

vibration. Finding the solution to this optimisation problem with the aid of previous generation ETEFs produced the current ETEFs as generated by Estekanchi et al. [112].

2.9.2 Applications of Endurance Time History Analysis

Analysis of three, three-storey steel moment frames designed to withstand different seismic intensities as per the Iranian national building code has been conducted as reported in Refs. [113, 114]. Frame dimensions and properties of the chosen sections are shown in Figure 2-75 [113, 114]. The frames were designed for different strength conditions as specified under each frame. In designing the frame f2D2A3s1b-Weak, the design lateral load was taken as half of the code-specified base shear, while for frame f2D2A3s1b-Strong, the design lateral load was taken as twice the code-specified base shear. The researchers conducted linear static, nonlinear static and nonlinear dynamic analyses on the structures and compared their behaviour with the behaviour of the building under ETEF [114].

The results for the ETHA in the elastic phase matched the THA results. The forecast for the storey with the highest inter-storey drift was also reported to be consistent among all dynamic analyses [114].

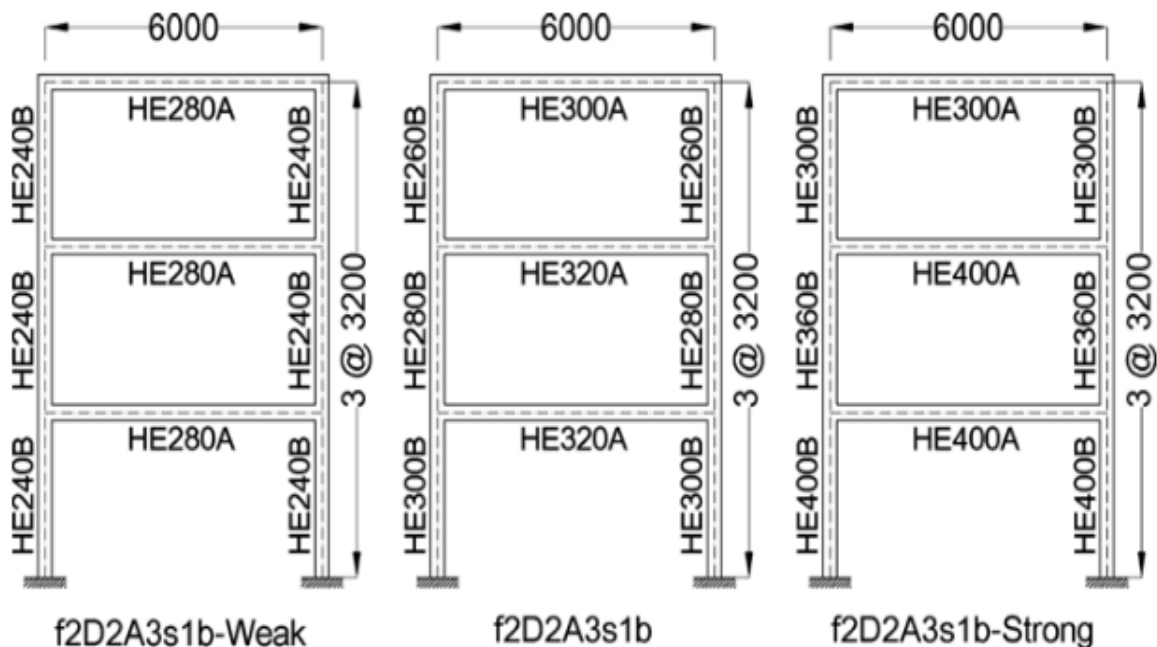


Figure 2-75 Schematics of three frames [114]

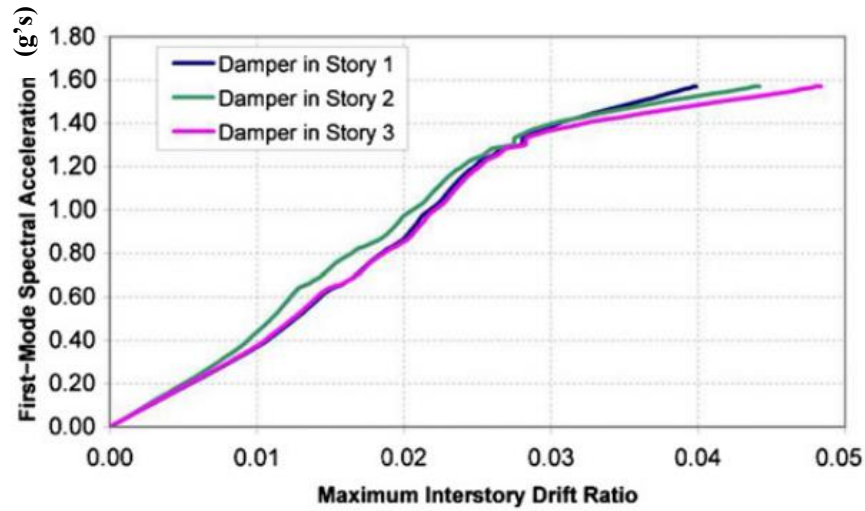


Figure 2-76 ETHA results for the frames, presented as IDA curves [114]

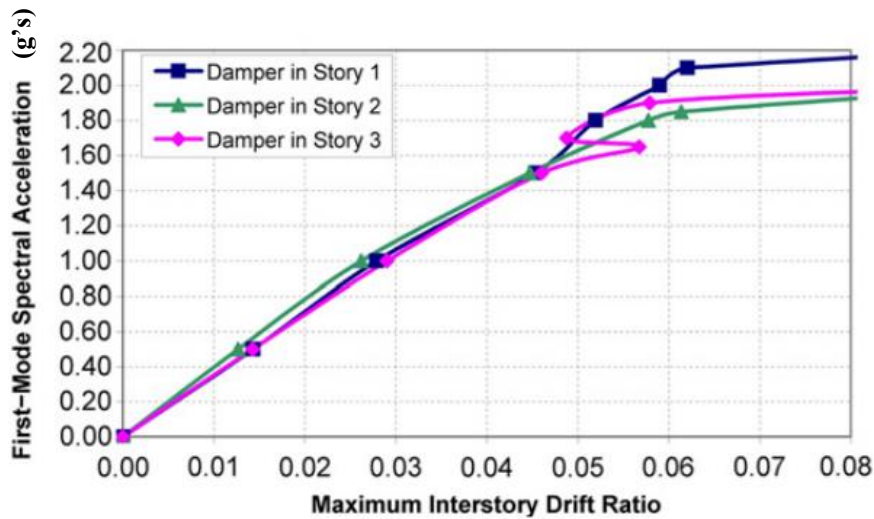


Figure 2-77 IDA curve results from IDA analysis of the frames [114]

The authors also conducted IDA analyses on the selected frames and compared them with the ETHA results, which were expressed in the form of IDA plots (see Figure 2-77). The ETHA estimates for maximum inter-storey drift ratio were reported to be lower than the nonlinear THA findings (see Figure 2-76). For weaker structures that experienced more nonlinearity during a specific earthquake, underestimation by ETHA was reported to be more significant. ETHA results were found to significantly underestimate the response of weaker structures that had a long exposure time to the nonlinear phase of the ground motion. Exceptions were observed sometimes where the ETHA method overestimated the drift ratios of the frames. This is because one acceleration function resulted in significantly higher maximum inter-storey drift values than did other acceleration functions. The base shears from the two methods were considered sufficiently similar. It was concluded that the ETHA and nonlinear THA results

were in good agreement with each other, even for structures with strength and stiffness degradation [114].

A similar study was conducted on the behaviour of four types of steel moment-resisting frames with flexible and stiffened panel zones with a selected ETEF and real ground motions. It was reported that the average damage values for the frames, as well as damage distribution in the frames, could be predicted very well using ETHA [118].

Another study was conducted to develop more reliable hysteretic energy-compatible ETEFs [119]. The generated ETEFs were compared with real earthquakes for their dynamic properties. The ETEFs generated were employed to analyse the behaviour of three concrete moment-resisting frames that were regular in both plan and elevation. It was reported that the generated ETEFs had 92%, 83% and 90% consistency with hysteretic energy, nonlinear displacement, and acceleration spectra of real earthquake events, respectively. Further, the ETEFs generated in the study were similar to real earthquake time histories in terms of their dynamic characteristics. Finally, the analysis conducted on the concrete frames using real ground motions and generated ETEFs revealed that the simulated ETEFs were capable of predicting the response obtained from nonlinear dynamic analysis of real ground motions to a substantial extent [119].

The study also compared the predictions from ETHA with the IDA results, which showed that the ETHA predicted the IDA results with 96% accuracy. As well as being more accurate, the computational time for ETHA was reported to be less than 1% of that for IDA. Finally, the study compared the inter-storey drift and plastic hinge rotation distribution throughout the building height, as obtained from ETHA and conventional THA methods. The ETHA predictions were very close to the results obtained from nonlinear THA [119].

A study focussing on the application of the ETHA method in performance-based design was conducted in which the seismic response of a structure, and different seismic intensities and their threshold values as per code recommendations, were expressed in the form of a ‘performance curve’ and ‘target curve’. The study revealed that, in the expression for performance, replacing time with the probability of exceedance or return period enables more explicit result presentation and increased convenience for the calculated probabilistic cost. The return period of an earthquake, which can be derived from the spectral acceleration corresponding to the effective period of the structure on the response spectrum corresponding

to a particular period, is a better indicator of the seismic hazard. The procedure followed in the study involved examining the coincidence between response spectra obtained from the ETHA accelerogram at different times and those defined for Tehran, Iran at different hazard levels. Substitution of the time axis with the return period increased the usefulness of the results, making the application of the method simple for value-based design [125].

Figure 2-78 and Figure 2-79 illustrate the relationship between the structural period, return period and time in ETHA. This relationship can be employed to derive the hazard corresponding to a specific time in an ETHA. Mirzaee, Estekanchi and Vafai discussed in depth the development of this sort of correlation [125].

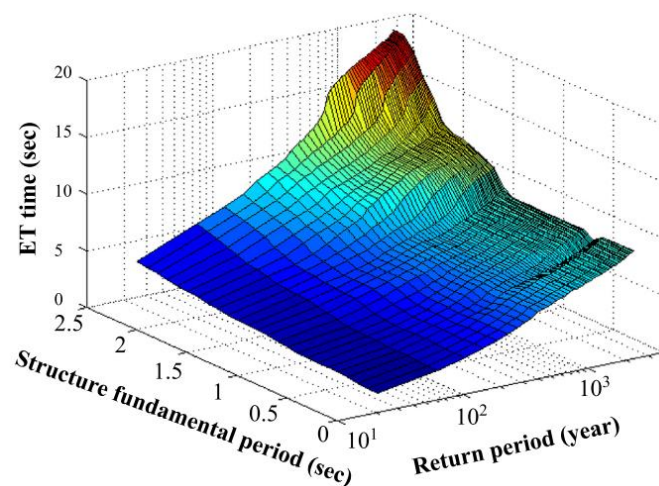


Figure 2-78 Return period v. structural period and ET analysis time [125]

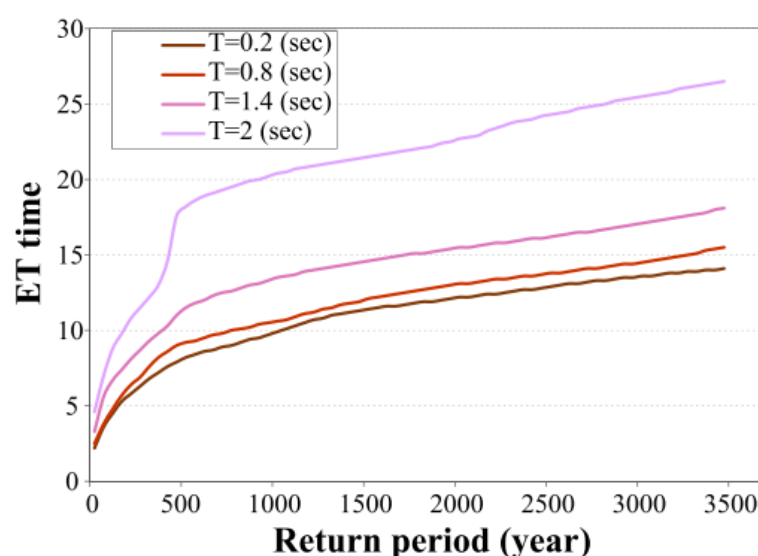


Figure 2-79 Equivalent ET analysis time v. hazard return period for different structural periods [125]

Figure 2-80 plots performance and target curves for a five-storey building. In this plot, the time axis has been replaced with the return period for the convenience of illustration. The plot shows that, even though the structure managed to satisfy collapse prevention criteria (CP in the figure), it did not meet immediate occupancy (IO) and LS criteria. Hence, the frame's performance was deemed unacceptable. Further, the ETHA results for the inter-storey drift envelope were smoothed to take into account the moving average. It is evident from the study that using ETHA simplifies analysis of a structure for its seismic performance, making it easily transformable to the more useful interpretation of results [109, 125].

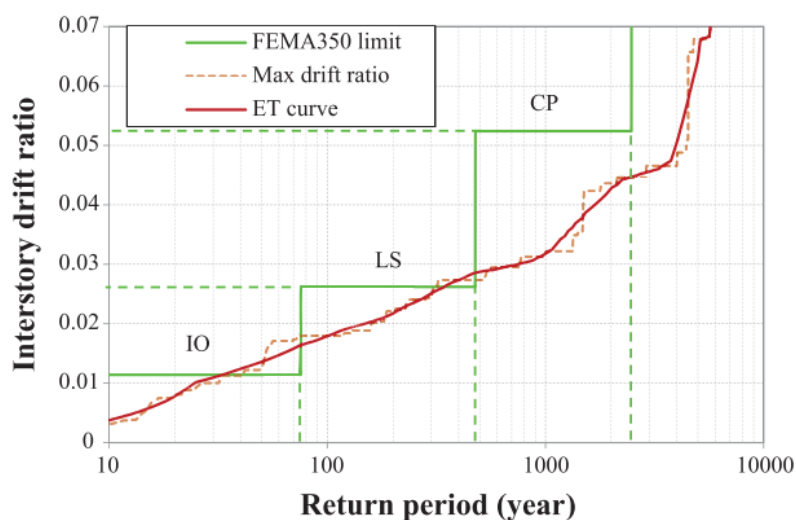


Figure 2-80 A sample performance curve (ET curve) for a steel frame [109]

2.10 Collapse Analyses versus Collapse Simulations

Collapse can be defined as ‘Spread of local damage, from an initiating event, from element to element resulting, eventually, in the collapse of an entire structure or a disproportionately large part of it [130]; also known as disproportionate collapse’ [131]. The phenomenon was first officially reported when the Ronan Point Tower in London, UK, collapsed following a gas explosion in 1968 [132]. The full collapse of the World Trade Centre Twin Towers during the 9/11 terrorist attacks in 2001 made the term more popular (ASCE/FEMA, 2002). Since the occurrence of major events of this nature, collapse studies have been given a very important place due to the requirement to ensure a safer performance of the structure under unforeseen circumstances which would leave the structure collapse.

Accurate representation of collapse response of a structure involves either collapse analysis or a combination of both collapse analysis and collapse simulation.

Collapse analysis deals with the destabilisation of a building resulted from overloading leading to the loss of one or more structural elements (in an earthquake for example), or destabilisation of the structure resulted from severe damage to one or more structural elements (following an explosion for example). A collapse analysis is essentially aimed at assessing the stability of a damaged structure to predict its ability to make use of alternative load paths to remain stable following an extreme event. The structure is deemed to have “collapsed” should the stability has been lost.

Collapse simulation is about predicting the motion behaviour of the structure in the course of ultimate failure. If the structure ends up collapsing, the simulation will also cover the actual collapsing (overturning). Most collapse analysis need not be paralleled by any collapse simulation given that the structure is deemed to have collapsed following the destabilisation. The actual collapsing motion need not be predicted.

In the case of a pure modular building “Collapse Analysis” alone might not give accurate predictions of the life safety outcome of an extreme event such as severe earthquake ground shaking because collapse analysis only goes as far as predicting failure of critical connections which might result in an unzipping of all the connections of the critical storey. Even if the building is deemed to have “collapse” because of the occurrence of brittle failure of the connections, the building might not necessarily actually overturn (collapse) and much to with its second line of defence: the ability of the breakaway part of the tower to experience motions without necessarily collapsing. Hence, it is imperative to conduct collapse analysis followed by collapse simulations to fully predict the collapse response of modular structures which may include the overturning of the free-standing superstructural component formed as a result of unzipping of connections of a critical storey.

2.10.1 Collapse Analysis in Different Types of Structures

The occurrence of collapse can be progressive or disproportionate, depending on the cause and mode of the occurrence of collapse of a structure. Generally, collapse analyses are conducted in two main situations.

The first and most common type is determining the capacity of the structure to redistribute gravity loads when a critical load-bearing element is removed. In this type of collapse analysis, the structure’s response is recorded under the influence of gravity when one or more critical load-bearing elements of the structure is removed. The response of the structure can be

progressive or disproportionate collapse depending on the construction nature and the elements removed. However, analysis of this nature may only provide an idea of the structure's collapse under the loss of the critical element under consideration or one similar to it, under the influence of gravity only. The alternative load path method is one of the most common methods used in analysing collapse of this nature [140]. It involves removing vulnerable members from structures to check the behaviour of the remaining members in the post-damage state after load redistribution. With the gradual increase of failed members, progressive collapse of the structure occurs. This analysis could be carried out using any form of analysis in structural Engineering, such as linear/nonlinear static or linear/nonlinear dynamic analysis. The trigger or initial damage needs to be adequately simulated to analyse progressive collapse using the alternative load path method. A common technique for simulating damage is to remove from the structure the most vulnerable elements or the most probable locations for anomalous sabotage [140]. In performing alternate load path analysis, Murtha considered for removal locations that held the highest loads [141].

The other type of collapse analysis involves evaluating a structure collapse under an abnormal loading. An abnormal loading can be any form of hazard, such as an earthquake, blast loading or crash loading on a structure. Loadings of this nature may have a much more detrimental effect on structure performance. Again, collapse under these scenarios can be progressive or disproportionate depending on many factors including the duration, strength, and nature of the loading; particularly, the construction nature of the structure. If the structure does not have many alternative load paths to redistribute the excess loading as a result of the failure of one or more components, the structure may fail disproportionately.

It is essential to distinguish between collapses analysed using these two methods, mainly because they analyse the structural response in two very different situations. However, a more destructive collapse nature can be expected under abnormal loadings because of the effect of external loadings in addition to the gravity causing the structure to lose stability.

Research on collapse prediction tends to be concerned with analysing progressive collapse behaviour, which has a lot to do with analysing the failure of members through their excessive deformation resulting in gradual loss of redundancy and leading to de-stabilisation of the structure as a whole. Research analysing the collapse behaviour of conventional structures mainly involves modelling the behaviour of structural members and their deformation because failure will be elementwise rather than due to a significant part of the building failing as a

whole. Hence, in predicting the collapse response of conventional structures, merely sticking to collapse analysis which involves predicting the failure progression in the structure is sufficient.

However, in structures like modular buildings where the construction nature is very discrete, analysis of collapse may require an entirely different approach. Modular buildings consist of alternate arrangements of modules and connections. In reality, inter-modular connections are weaker than the modules. Hence modular buildings are more prone to failure because of inelasticity concentration at the connections resulting from the discrete nature of the construction. Figure 2-81 (a) depicts a typical model of a conventional steel building frame, and Figure 2-81 (b) illustrates a modular building frame.

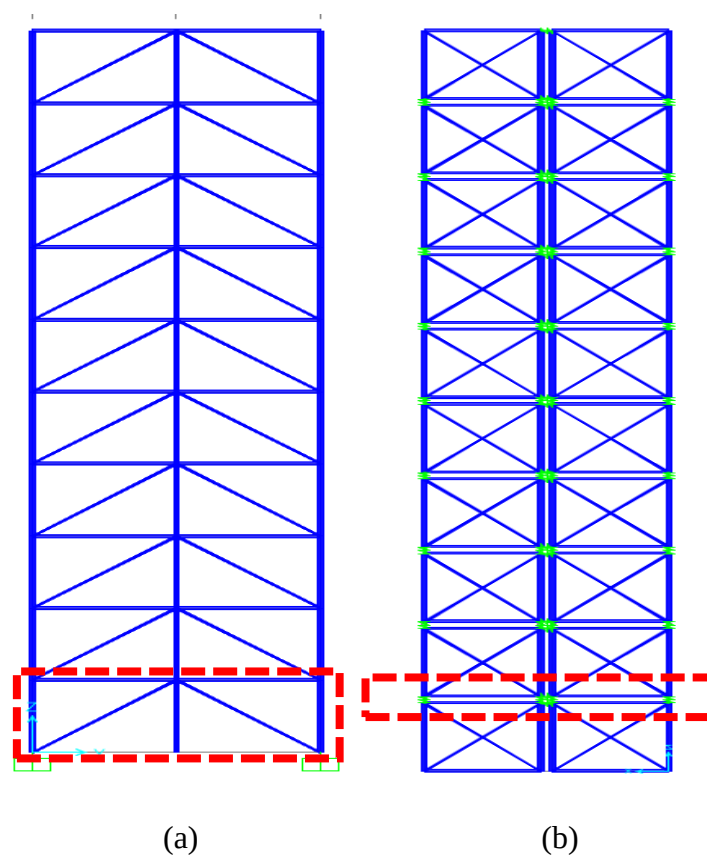


Figure 2-81 Typical model of a (a) conventional and (b) modular building

With reference to Figure 2-81 (a) and considering the storey of interest (as marked using broken red lines), damage to the conventional building will be distributed elementwise as highlighted. Hence, the collapse prediction would require conducting only a *collapse analysis*, which includes analysing each member in the structure under every time step to evaluate their state and simulate the failure of the member once its ultimate limit state is exceeded.

In contrast, the damage to a modular building will be concentrated at the connection points of the storey because of the discrete nature of construction. Under higher seismic intensities, these connections may experience premature failure because of their lower strength compared with the modules. When one connection fails, the alternative load path will be other connections in the same storey. These connections may experience stresses beyond their design limit, causing them to fail. The difference between this failure and that for a conventional building is that it does not progress to the module components. Still, it might progress across the connection storeys, causing a structural unzipping-type failure across connection storeys [55]. As there are few elements other than connections of the same storey that act as the alternative load path, this failure might spread rapidly, unlike the progressive collapse occurring in conventional buildings. Apart from the high rate of damage progression, the unzipping of a connection storey may cause the structure to behave as a huge free-standing block. The free-standing block thus formed may undergo rocking, sliding, and overturning leading to a catastrophe.

Hence, in pure modular structures, a mere *collapse analysis* may not predict the full collapse response of the structure. A collapse analysis to predict the failure of connections and the formation of a free-standing stack of modules, followed by a *collapse simulation* to predict the response of the formed free-standing stack of modules under the rest of the ground motion will be required to predict the complete collapse response accurately. The *collapse simulation* step is an extra step required from the general collapse prediction studies on conventional structures.

These analyses are not commonly required for collapse assessment of conventional buildings, due to the distributed nature of their failure. The rapid and overturning nature of collapse expected in modular buildings might cause significant danger to occupants and the building's surroundings from the collapse of a colossal building mass [55].

It is evident from these discussions that prediction of the collapse of modular structures may require different approaches in multiple stages, unlike collapse prediction for conventional structures. Nevertheless, it is essential to review and understand the approaches followed to *analyse* and *simulate* collapse in conventional structures to choose an appropriate method or a combination of methods to predict the collapse behaviour of modular structures.

2.10.2 Methods of Collapse Analyses and Collapse Simulations

Since substantial nonlinear and mechanical behaviour must be considered in the analysis and simulation of collapse, much work is required when these are to be represented in FE models

of experimental models. Research has focussed on two approaches to numerically simulating and analysing progressive collapse: the discrete element method (DEM) and the FE method [126].

2.10.2.1 Finite Element Method (FEM)

In the finite element method (FEM) approach of studying collapse, FE models of a structure are employed to study its response and collapse behaviour. Collapse studies have mainly been carried out using models developed for three approaches: fine, simplified, and multi-scale modelling.

Fine modelling refers to very intense fine FE models of a structure. Although fine modelling is possible using different elements, based on the behaviour of structural members, because of the complexity of its modelling tasks and the extensive calculations involved, this approach has been mostly used for specimens or substructures [126]. Fine models of steel and concrete frames that also had an exterior mid-side model and corner model have been reported [127]. Static and transient dynamic analyses were conducted to understand the progressive collapse resistance of these structures. Although fine models are more similar to the actual situation of a structure, these methods require substantial time and effort. For this reason, simplified models are preferred when conducting collapse predictions [127].

One study on a beam-column element had lumped plasticity and multi-linearisation to incorporate them into structural models. A damage index was introduced to identify the failure of these structural members through degradation of strength and stiffness of structural members. The stiffness of elements was set to modify and release internal forces in the event of a failure. A simplified FE model developed with these elements and monitored through the said damage index was capable of accurately analysing the collapse [128].

Multi-scale models are a combination of fine and simplified models. In this approach, fine models are used for critical components or structural elements experiencing complicated stress states. Simplified models simulate elements that are in a less critical state or elements that are less influential in the behaviour of the building. Overall, this type of model is computationally less expensive than fine models and produces more accurate results than simplified models [126]. In a study of a high-rise steel building modelled using this approach, the material and geometrical nonlinear properties of the specimens were taken into account using a macro

model. In contrast, materials and locations such as connections and contacts, which are more sensitive and in a complicated stress state, were modelled using a micro model [129].

However, in the overall approach employed in FE modelling of collapse, the full structure is represented using FE models. The relative sensitivity of the location of the structure determines its treatment during FE modelling. Finite element methods are commonly employed in *collapse analysis* due to the detailed treatment required in modelling different elements of a structure, and to evaluate their state at different time steps during the loading and identify failure, and hence collapse.

2.10.2.2 Discrete Element Method

In this method, the structure is represented as an assembly of rigid bodies and springs, to simulate its mechanical behaviour. The constitutive relationships of springs have been used to determine the internal forces between the rigid bodies. Newton's laws of motion have been used to calculate the displacement of rigid bodies.

The DEM, which simply depends on the equations for equilibrium, constitutive relationships and boundary conditions, is more suitable for analysing discontinuous structures and large deformations, than is the FEM. Compared with the FEM, the DEM is best suited for analysing of nonlinear and large displacement problems, because it avoids the formation of a stiffness matrix and complicated matrix operations with the aid of explicit formulations in a dynamic analysis. However, this is not an indication of any superiority of the DEM over the FEM for solving discontinuous problems [130].

The DEM has proved very useful, especially when a study involves rigid body movement and contact [126]. The first reported use of the DEM was in the simulation of the movement of a rock slope, and it later proved very popular in mining projects and soil mechanics [130].

Apart from employing just the DEM, studies have been successful in using a combination of finite-discrete element methods to model progressive collapse [131]. Comparisons between the outcomes from experimental and numerical methods involving these approaches have demonstrated high accuracy of predicted results and simplification of the process involved [126, 131].

2.10.2.3 Applied Element Method

The applied element method (AEM) is a hybrid of the DEM and FEM. This has benefits in terms of cost of computation and accuracy of simulating the collapse response, being a mix of the DEM, which involves much lower computing costs and the FEM, which can be used to represent the most critical areas in a building accurately [132]. A comparison can be made between FEM and AEM as shown in Figure 2-82.

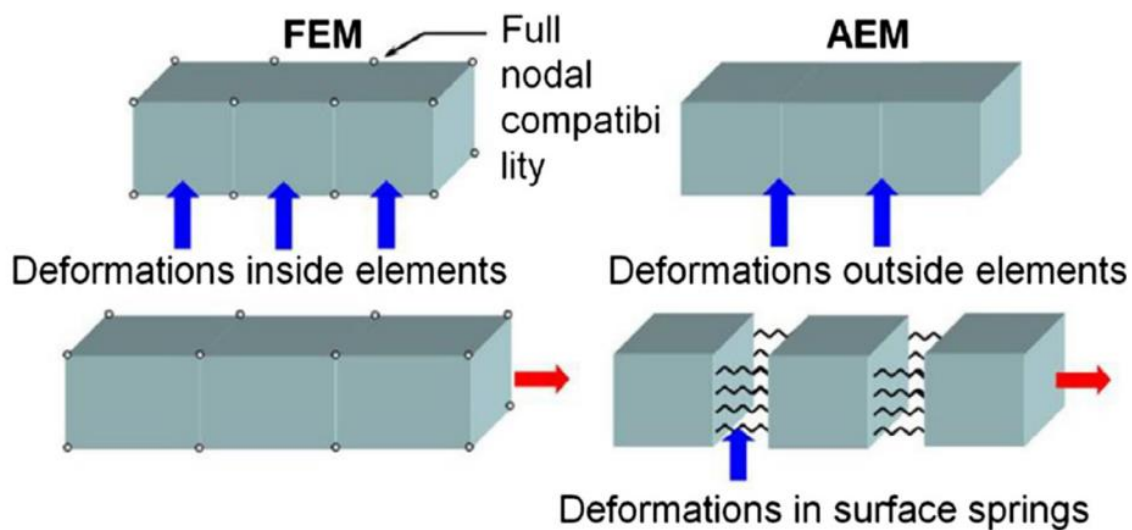


Figure 2-82 Comparison of the FEM and AEM method [132]

It is essential to choose an appropriate method or a combination of different methods from these discussed methods to model the progressive collapse behaviour of a structure while maintaining an appropriate balance between the accuracy of results and computational time. Further, the relative advantages and disadvantages of each of these methods can only be realised by exploring previous studies on progressive collapse, which is presented under section 2.10.3 below.

2.10.3 Previous Studies on Progressive Collapse

In a study on the consequence of corner column loss on the collapse behaviour of a steel frame, the collapse mechanism of the frame due to loss of a corner column was identified via critical ductility curves developed using an analytical method [133].

Tavakoli and Alashti conducted an analysis to study the effect on seismic performance of a multi-storey, moment-resisting frame with damaged columns [134]. The study reported that removal of a middle column resulted in more robust behaviour of the structure, but this was

less robust following the removal of a corner column. Moreover, the study showed that the capacity of the structure to resist progressive collapse was enhanced with increasing numbers of storeys and bays because more elements were participating in resisting progressive collapse [134].

A study involving steel moment-resisting frame structures showed that the failure of an edge column on the ground floor could cause a considerable increase in axial force to other columns in adjacent bays, leading to failures in adjacent bays [135].

Another study analyzed the behaviour of steel moment-resisting frames designed to adhere to the Iranian national building code [136]. For this study, a 10-storey steel moment-resisting frame designed for high-level seismic zones in Iran was employed. Its exterior frames were examined under four situations for collapse progression after losing one main column; that is, the corner column on the ground floor, fifth floor, eighth floor and the floor below the roof. The study revealed using nonlinear THA that the building collapsed progressively upon removal of columns at each stage [136].

Researchers have studied the significance of stability considerations within a geometric and material nonlinearity analysis framework, as the most important approach for correctly predicting collapse mechanisms and the corresponding collapse loads [137, 138]. The most commonly observed collapse patterns in structures have been identified as plasticisation of beam edges above the location of the lost column (yielding-type beam failure; ductile) [139]; stability loss resulting from the removal of adjacent (buckling-type column failure; brittle), beam-column connections and shear failure [140]; and, less commonly, the failure of the global structural system through loss of stability, which is more widely seen in tall and slender buildings [141].

A numerical investigation on a steel modular building to evaluate its structural robustness was undertaken; in the first stage of the study, a five-storey building made of six modules per storey and connected with rigid inter-modular connections was studied in the face of the loss of one base corner module [142]. This model was considered a reference model for further parametric studies involving analysis of the effect of the number of modules per floor; size and quantity of supporting posts; capacity of inter-module connections; and effect of rotational stiffness and longitudinal wall braces on the structural robustness of a steel modular building. The study also focussed on the response of the structure to different element removal scenarios. Four probable

ultimate states were reported for the structural configurations: ‘collapse’, ‘collapse arrested without member buckling’, ‘collapse arrested with member buckling’ and ‘partial’ collapse, which occurs as a result of the inadequate capacity of the inter-module connections [142].

The five-storey, six-bay structure made of pinned inter-modular connections and the five-storey, four-bay structure assembled with rigid connections, which reported collapse, experienced initial load distribution, buckling of members, crushing of modules (floor by floor) and building motion stabilisation. The authors reported that the progressive collapse was due to the excessive buckling deformation-induced rapid motion of the building. The kinetic energy gained from the building motion was reported to be dissipated through the crushing of the modules. Another five-storey, six-bay structure connected using rigid connections, which automatically forced the members of the module to be more vulnerable through buckling, was also studied. The typical process of progressive collapse in this type of building began with load redistribution, member buckling and stabilisation of the full structure. It was observed that during this type of failure, the building motion during member buckling was controlled because of redundant load paths and sufficient member capacity, and the building was protected from overall instability [142].

The study was extended to a modular building consisting of eight bays (eight modules per storey) and five storeys, and the section size of members was greater than in the structures discussed above. Because of the increased collapse resistance resulting from increased section sizes and the availability of more alternative load paths, the structure was reported to have stabilised immediately after initial load redistribution without any progressive collapse being observed [142].

The study was focussed on the progressive collapse nature of modular buildings designed with either rigid or pin connections without accounting for the flexibility of such connections in reality. Due to this limitation, the progressive collapse behaviour observed in the study was not significantly different from that for conventional structures as discussed earlier under this section. This research represents the first type of collapse analysis presented in Section 2.10.1, which focusses on the collapse induced in a structure under the action of gravity alone.

It can be concluded, based on the findings of most of the above studies, that the loss of elements leads to a progressive collapse in conventional structures. Moreover, failure is distributed throughout the building with and without alternative load paths throughout the building height.

The progressive collapse nature of modular buildings, which are assumed to have rigid or pinned type connections, has been discussed under this section. Even though these assumptions are not very realistic in practice, they represent a useful initiative for simulating collapse in modular buildings.

2.10.4 Collapse Analysis and Simulation in Pure Modular Structures

Collapse can be of paramount concern, mostly during dynamic loadings. Collapse predicted under dynamic loadings provides more precise information about the collapse behaviour of a structure. Nonlinear pushover analysis can be useful when the nature of the collapse of a structure needs to be understood with minimum computational and time investment. However, a pushover analysis might not give the full picture of a collapse. Compared with static methods, dynamic analysis inherently involves a dynamic influence, damping and inertia, which makes predictions more accurate [126, 142, 143].

In conducting collapse analysis for conventional structures, it is crucial to precisely detail and model the nonlinearity of each component of the structure. Collapse analysis involves taking into account the nonlinearity of the structural elements resulting from plastic hinges or fibre elements and tracking their behaviour under dynamic loadings [132, 143].

One of the trickiest phases in collapse analysis is the choice of the collapse analysis method. FEM may be the most accurate method for modelling the complete nonlinearity of a structure and understanding its collapse behaviour. However, this may not be economical in terms of labour and computational time. Conversely, DEM might not suit all types of structures or situations.

When buildings are considered, especially conventional structures, the FEM using a simplified or multi-scale model alone is sufficient to analyse collapse behaviour. As highlighted at the beginning of Section 2.10.1, research analysing the collapse behaviour of conventional structures is mainly about modelling the behaviour of structural members and their deformation because the failure will be elementwise rather than due to a significant part of the building failing as a whole. Hence, there is no necessity of conducting collapse simulations which involves simulating the mechanics of a component separated from the building.

However, the situation may be entirely different in the case of a modular structure. As explained in Section 2.10.1, failure involved in a modular building may require the analysis of

a large block of modules, or of single modules falling apart from the modular building. This is not generally required in the case of conventional structures. Hence, collapse response prediction for modular structures may require a combination of collapse analysis and collapse simulation.

Based on an understanding of the nature of construction and the relative strengths of the connections and modules, it is hypothesised in this study that failure will be more concentrated on the connections. Further, it is hypothesised that the failure of one of the most critical connections may lead to other connections of the same storey experiencing a sudden increase in inelasticity concentration and hence the failure of all other connections in that storey. This is expected to leave behind a large block of superstructure component consisting of many modules, free-standing on top of the failure plane (critical storey). This superstructure component may undergo rocking, sliding, and overturning under the influence of subsequent ground motion.

Based on the above hypotheses, two stages of required collapse prediction can be identified for a modular structure under a seismic event. The first stage of the collapse response prediction would require conducting *collapse analysis* to study the failure of connections and the formation of a free-standing stack of modules. Even though it is hypothesised that the failure will be limited to the connections, the FEM can be utilised to develop a fine model of the full building, and then study failure progression in the structure to verify the hypothesis; that is, show that failure of connections is the concern, and the modules are safe. The Chapter 3 of this study is based on the collapse analysis on fine finite element models of a prototype modular and conventional steel building. The model of modular building employed under this chapter was aimed at accurately representing the failure progression in the connections up to the formation of a free-standing stack of modules.

The second stage of the collapse response prediction will require conducting *collapse simulations* (as explained under section 2.10.1) to model the response of the free-standing block of modules under the rest of the ground motion. To simulate the collapse response at this stage, DEM can be considered as the best suitable approach, as it is more suitable for the behaviour of a stack of modules or single modules that can be assumed to be rigid bodies.

Chapter 4 and Chapter 5 of this study is aimed at making further simplifications of the considered prototype modular structure, where, modules are modelled as rigid bodies which

never fail. Hence, the need to model the entire structure using a fine finite element model is avoided, and the connections which are the most critical elements can be modelled using a fine finite element approach. This type of model was classified as simplified or multi-scaled models as per section 2.10.2.1.

The *collapse simulation* component of the modular building collapse response is entirely simulated using the DEM method (discussed under 2.10.2.2). Since the response of the free-standing superstructure under this section is expected to be rocking and sliding as a full block, FEM is not required. Chapter 6 of this study is aimed at studying this phase of the *collapse simulation*.

2.11 Dynamic Similitudes and Scaling

Studying the behaviour of large complex structures can be cumbersome when it involves physical testing at real scales. Physical scale modelling has been used in many studies to experimentally examine the behaviour of large complex structures under different conditions, such as seismic events [144].

Scaled models are designed to have geometric, kinematic, or dynamic similarity to the prototype. Geometric similarity implies that the model and full-scale structure have the same shape [145]. This will lead to the same scale ratio for all linear dimensions between the prototype and the model. Kinematic similarity implies that the prototype and the model have analogous particles at a comparable point at a similar time (similarity in velocity). The dynamic similarity is deemed to be satisfied when analogous parts of the prototype and model undergo analogous resultant force. When a model reflects dynamic and geometric similarity, it automatically reflects kinematic similarity too [145, 146].

Moncarz and Krawinkler categorised the possible levels of accuracy between scaled and prototype models as ‘true’, ‘adequate’ or ‘distorted’. A true model satisfies every similitude condition. An adequate model accurately scales the most important features of the prototype; while there is no significant effect on the prediction equation, the secondary influences are allowed to deviate. A model is referred to as distorted when it has deviated at all from the similitude requirements, which creates a distortion in the prediction equation, or when the prediction equation requires the addition of compensating distortions in other dimensionless products to preserve itself [147].

Two types of similitude are commonly used in scaled models involving structures. One is the Cauchy similitude, which is expressed as the Cauchy number, $C_N = \rho v^2 / E$, where C_N is the Cauchy number; ρ is the density; v is the velocity, and E is the elastic modulus. According to the Cauchy similitude, the Cauchy number should be the same in the model and the prototype. The Cauchy similitude criterion is usually sufficient for cases in which restitution forces are essentially elastic [145, 148].

The second type of similitude is the Froude similitude, which is expressed as the Froude number, $C_F = v^2 / Lg$, where L is the length, g is the acceleration of gravity, and other notations are as mentioned above. The Froude similitude criterion is adequate when gravity actions play a significant role [145, 148].

Similitude relationships for major parameters of interest in any study, expressed in the form of Cauchy and Froude similitudes, are shown in Table 2-6. Here the subscript M refers to the model, and subscript P refers to the prototype.

Table 2-6 Similitude relationships [145, 148]

Quantity	Symbol	Cauchy similitude	Froude similitude
Length	L	$L_P = \lambda L_M$	$L_P = \lambda L_M$
Modulus of elasticity	e	$E_P = e E_M$	$E_P = e E_M$
Specific mass	ρ	$\rho_P = \rho \rho_M$	$\rho_P = \rho \rho_M$
Area	A	$A_P = \lambda^2 A_M$	$A_P = \lambda^2 A_M$
Volume	V	$V_P = \lambda^3 V_M$	$V_P = \lambda^3 V_M$
Mass	m	$m_P = \rho \lambda^3 m_M$	$m_P = \rho \lambda^3 m_M$
Velocity	v	$V_P = e^{1/2} \rho^{-1/2} v_M$	$v_P = \lambda^{1/2} v_M$
Acceleration	a	$a_P = e \rho^{-1} \lambda^{-1} a_M$	$a_P = a_M$
Force	F	$F_P = e \lambda^2 F_M$	$F_P = \rho \lambda^3 F_M$
Moment	M	$M_P = e \lambda^3 M_M$	$M_P = \rho \lambda^4 M_M$
Stress	σ	$\sigma_P = e \sigma_M$	$\sigma_P = \lambda \rho \sigma_M$
Strain	ε	$\varepsilon_P = \varepsilon_M$	$\varepsilon_P = \lambda e^{-1} \rho \varepsilon_M$
Time	t	$t_P = \lambda e^{-1/2} \rho^{1/2} t_M$	$t_P = \lambda^{1/2} t_M$
Frequency	f	$f_P = \lambda^{-1} e^{1/2} \rho^{-1/2} f_M$	$f_P = \lambda^{-1/2} f_M$

Chapter 3: Development and Analysis of Prototype Models

3.1 Prototype Buildings Considered in This Study

3.1.1 Prototype Model of the Modular Building

It is shown with the support of the literature in Section 2.2-2.5 that low- to mid-rise structures can exist without the requirement for any external LFRS. Specifically, Section 2.5 reports on an investigation of the possibility of a 20-storey pure modular building [68]. A stack of modules as low as five storeys may not have a significant effect during a seismic event. Hence, this study focussed on the collapse behaviour of a structure for which the possibility of existence has already been established, and that might suffer impacts from lateral loadings such as earthquakes.

Considering these aspects, and to avoid any external effects such as higher-mode effects, which might hinder clear understanding of what is happening in the structure during the collapse, a 10-storey modular building was considered in this study. Considering land constraints in metropolitan areas, structures are more likely to be built taller than to expand horizontally. Hence, an aspect ratio of 1:5, which equates to two bays of modules per storey, was employed. Intuitively, such a slender structure presents the worst-case scenario, so the results of this study can be easily expanded to form general conclusions on the performance of most of the modular structures. The final dimensions of the structure were chosen to be 32.5 m in height, 6.3 m in width and 9 m in length. The size of the modules chosen mostly controlled the final dimensions of the structure.

3.1.1.1 Module

The module size was chosen to be 3 m wide, 3 m high and 9 m long to generally represent the modules currently used in practice [1, 2, 11, 12, 14, 18]. The modules were assumed to be manufactured with in-built braces consisting of $50 \times 50 \times 3$ SHS to resist lateral loadings, rather than stiff wall elements as reported in Refs. [2, 12]. The ceiling elements of the modules were assumed to be made of 75-mm-thick concrete panels, and the floor was considered to be made of 100-mm-thick concrete panels [2, 12]. Diaphragms were assigned to each module floor and ceilings separately to represent the diaphragm action provided by the slab and ceiling elements.

The module used, together with a summary of all components in the module, is shown in Figure 3-1.

The mass of wall elements and other dead loads on the structure were lumped at the nodes. In addition to the dead loads, a live load corresponding to a general loading category of 3 KPa was added as per AS/NZS 1170.1 [149]. Permanent loads (G) resulting from the weight of the module and its components, and imposed loads (Q) were combined ($1.2G+1.5Q$) using combination factors as per AS/NZS 1170.0 [150] to calculate the net loading on modules.

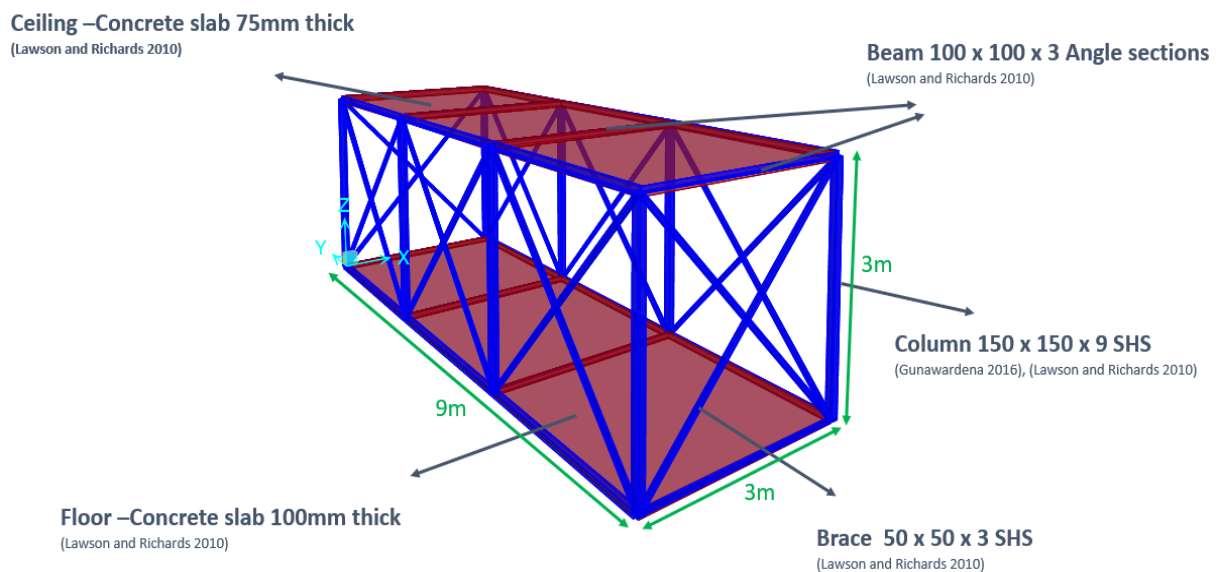


Figure 3-1 Illustration of a module used in this study

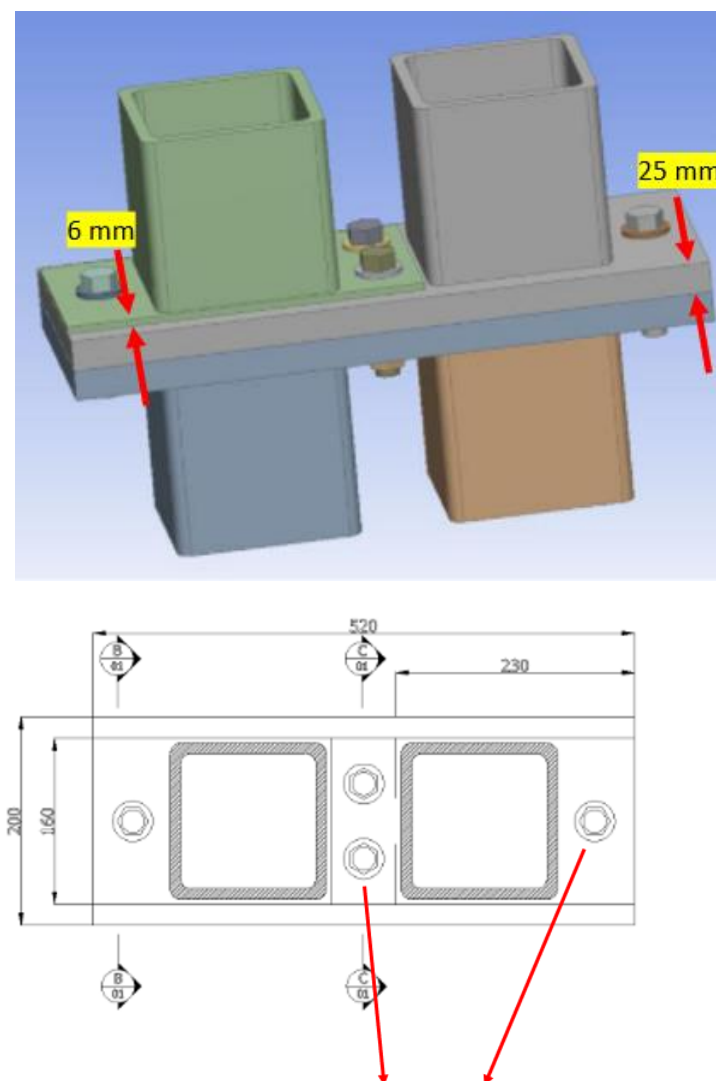
3.1.1.2 Inter-modular Connection

The literature review emphasised that most connections invented by many researchers involve bolts and splice action to act as vertical and horizontal connections. The connection system proposed by Gunawardena et al. is an excellent general representation of many connection systems that have been used in modular construction. This connection, shown in Figure 3-2, was adopted in this study as the inter-module connection.

This study was conducted to align with the actual design practices. As per the design practices, for short to mid-rise structures (less than around 15 storeys in height), it is the general practice to use uniform structural component designs throughout the entire structure height. In this context, columns, beams, slab sections, connections and any other component of the structure are referred to as structural components. Furthermore, the structural model consideration in this study can be correlated to the study conducted by Gunawardena's study [12]. In

Gunawardena's study, the author conducted analysis on a 20-storey modular building. In this structure, a uniform joint design was adopted throughout the structure height. Moreover, the joint design considered in this study was the same joint design that was proposed by Gunawardena. Hence, this study also aimed at highlighting the risk involved in this sort of uniform design considerations throughout the structure height, especially for modular buildings. Accordingly, a uniform connection design was considered along the structure height.

The connection properties were required to be fed into the FE models to simulate the real-life response of the structure in the FE model. The main properties of the connection in two directions were required: under push-pull action and shearing action, as shown in Figure 3-3.



- 12mm *friction grip* bolts with 14mm bolt hole diameter
- Grade 8.8 with a yield strength of 660 MPa

Figure 3-2 Inter-modular connection used [12]

3.1.1.2.1 Behaviour of the Connection Under Shear Action

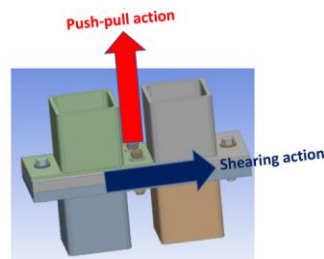
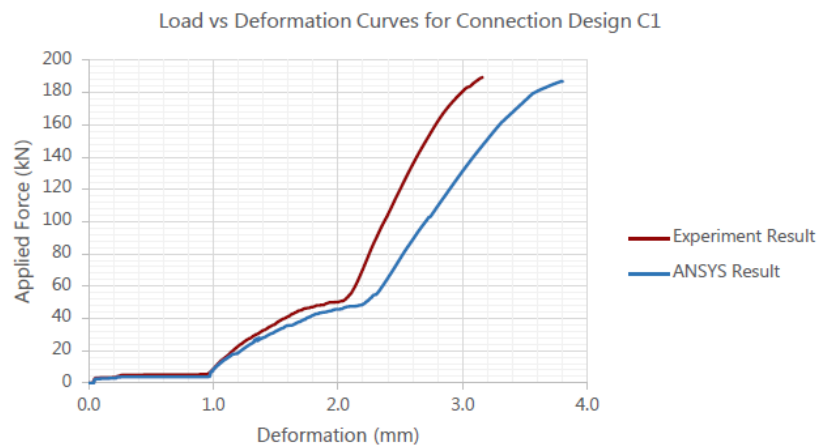
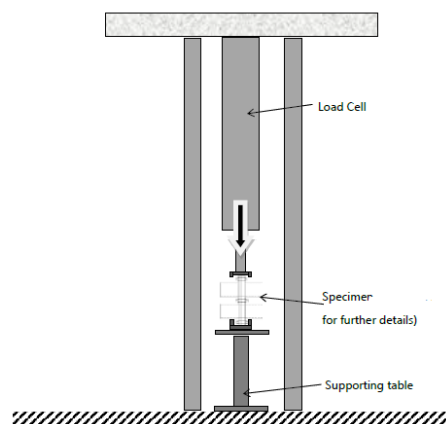


Figure 3-3 Main deformation types experienced by the connection [12]

The deformation plateau of the connection under shear action, obtained using experimental and numerical approaches as reported in Ref. [12] was employed in this study. Figure 3-4 (a) illustrates the load v. deformation curve obtained in this manner.



(a)



(b)

Figure 3-4 (a) Load v. deformation curve under shear action and (b) test set-up [12]

As shown in Figure 3-5, three distinct stages were identified in the load-deformation plateau for the connection. The first was the *slipping stage* (OP in the figure) in which the splices slip because of the loss of tensile grip from the bolts; the second was the *bearing stage* (PQ), which occurred because of the slipped bolt bearing on the splices; and the last was the *failure stage* (QR) where the connection was no longer functional. This idealisation is very useful when connection details need to be fed into FE models [15].

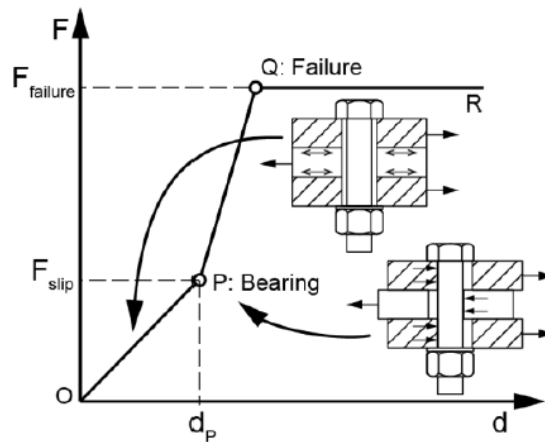


Figure 3-5 Different stages of bolted splice connection behaviour [15]

The shear force v. deformation plateau depicted in Figure 3-4(a) was idealised following the approach presented in Figure 3-5. The idealized shear force v. deformation plateau is depicted in Figure 3-6 below. The values corresponding to each parameter marked in Figure 3-6 are summarized in Table 3-1.

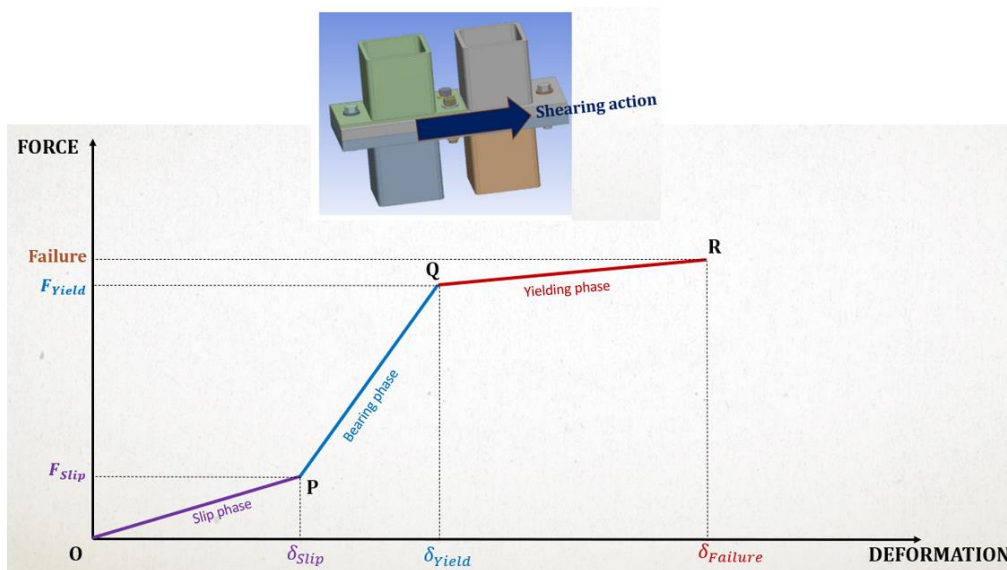


Figure 3-6 Idealised curve for connection shear deformation

Table 3-1 Computed parameters corresponding to the idealised curve (shear deformation) in Figure 3-6

K_{OP} -Stiffness at slip	12,700 KN/m
K_{PQ} -Stiffness at bearing	98,000 KN/m
K_{QR} -Stiffness after yielding	$0.001 \times K_{PQ}$
δ_{Slip} -Shear deformation at slip	2 mm
δ_{Yield} -Shear deformation at yield	4 mm
$\delta_{Failure}$ -shear deformation at failure	12 mm

Values obtained from experimental and numerical computations undertaken as per Figure 3-4 (a) were used for the stiffness at slip (K_{OP}), stiffness at bearing (K_{PQ}), stiffness after yielding (K_{QR}), shear deformation at slip (δ_{Slip}) and shear deformation at yield (δ_{Yield}). In this study, the QR branch of the plot was assumed to be for a line with a very low gradient (very low stiffness), rather than assuming it to be a flat line, to prevent convergence issues during FE analysis.

Moreover, this study aimed to simulate the collapse of the building, which required simulating the failure of connections. For this purpose, the QR line in the plateau was assumed to end at a finite displacement value calculated using the ductility value for the connection. General ductility values for these types of bolted splice connections were used in determining the maximum deformation of the connection before failure [42, 71].

3.1.1.2.2 Behaviour of the Connection Under Tensile Action

Once the shear deformation plateau was obtained, it was essential to obtain the axial force deformation plateau for the connection under push-pull action. No experimental results were reported for the axial force-deformation plateau for this connection. Hence, to obtain the axial deformation plateau for the connection, theoretical calculations were performed. According to Wileman et al. [151], the stiffness of clamped material can be calculated using the following equation:

$$K_m = AEde^{B(\frac{d}{L})} \quad \text{Eq 3-1}$$

Where,

E = Modulus of Elasticity of the material

d = Diameter of the bolt hole

$\frac{d}{L}$ = Aspect ratio of the joint, where ‘L’ is the grip length of the bolt

A = Numerical dimensionless constant depending on the material and carries a value of 0.78715 for steel

B = Numerical dimensionless constant depending on the material and carries a value of 0.62873 for steel

This equation was thus employed to arrive at a calculated tensile stiffness for the connection, as $K_m = 540,000$ KN/m. The strength of the connection was computed, taking into account the lowest of the strength of the bolts in tension and the strength of the plates against bearing. Finally, as for the shear deformation plateau, a finite value was used as the maximum axial deformation of the connection at failure based on the ductility values. The resultant force deformation behaviour of the connection in the axial direction is shown in Figure 3-7, and the corresponding values are presented in Table 3-2.

Table 3-2 Important parameters in the axial force-deformation plateau

K_{OP}	540,000 KN/m
K_{PQ}	$0.001 \times K_{OP}$
δ_{Yield}	0.5 mm
F_{Yield}	224 KN
$\delta_{Failure}$	1.5 mm

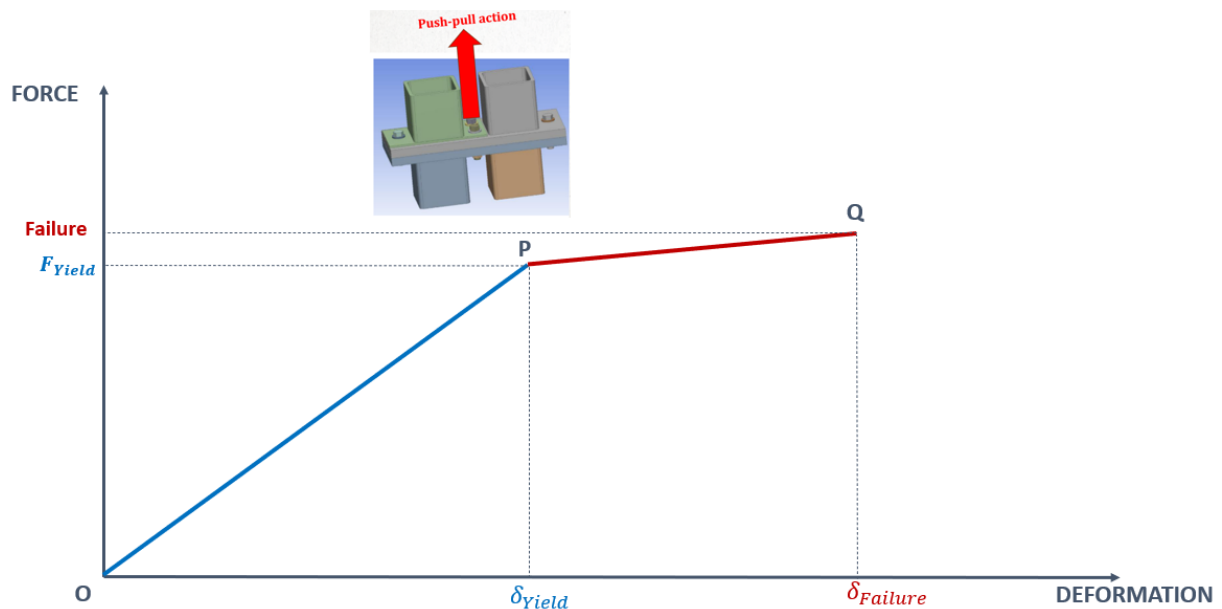


Figure 3-7 Force–deformation plateau in the axial direction

3.1.1.3 Finite Element Modelling of the Connections and the Building

Inter-modular connections play a highly significant role in the overall performance of modular buildings. Hence, it is crucial to model these connections accurately to capture all essential states that the connection may undergo. Unlike in conventional buildings, modular building connections form a small gap between adjacent modules in both the horizontal and vertical direction. This gap may be one reason for connections being one of the most critical elements in the building. When there is a small gap creating a less strong element between two strong modules, it is highly likely that the connection will be a spot of inelasticity concentration. Moreover, modelling of the connection should be able to capture the relative strength and stiffness of each connected component.

One of the major drawbacks of many previous studies conducted on modular buildings is that their FE modelling did not account for the real stiffness and strength of connections. Many authors who led initial research in modular construction did not account for the real behaviour of inter-modular connections. The connections were just considered as pinned, assuming only rotations between modules but not allowing for the flexibility of connections that would exist in reality [24, 33, 34, 38, 56].

Later, with more research on modular buildings specifically emphasising the overall structure's behaviour, which is strongly governed by the inter-modular connections, researchers began to focus on representing the actual connection behaviour rather than representing the connection as a hinge [15, 35, 152]. A two-node link element has been used in SAP2000 and ETABS to represent the connection [68, 152]. This proved beneficial by making it possible to assign the real behavioural plateau of the connection in each direction and to specify a length for the connection so that the gap between the modules in real life could be simulated.

In this study, connectivity between modules is represented using a two-node link element with stiffness and strength properties calculated as above assigned in each direction.

Special consideration was given to assigning stiffness for the axial direction of vertical connections (vertical push-pull action). A relatively high stiffness was assigned to the vertical (axial) compression direction only, to simulate the higher stiffness a connection may possess under compression (when modules sit on connections). FE modelling was carried out primarily in OpenSees and subsequently verified by modelling in SAP2000.

In modelling vertical connectivity in SAP2000, two link elements had to be used to represent the vertical connection. One link element took care of primary connection properties such as shear stiffness and axial tensile stiffness; the other link element, assigned as a gap element placed parallel to the main link element, took care of the higher stiffness in axial compression only. The combined action of these two links placed in parallel represented the actual behaviour of the connection in all primary directions. The approach to modelling of a single vertical connection in SAP2000 is summarised in Figure 3-8.

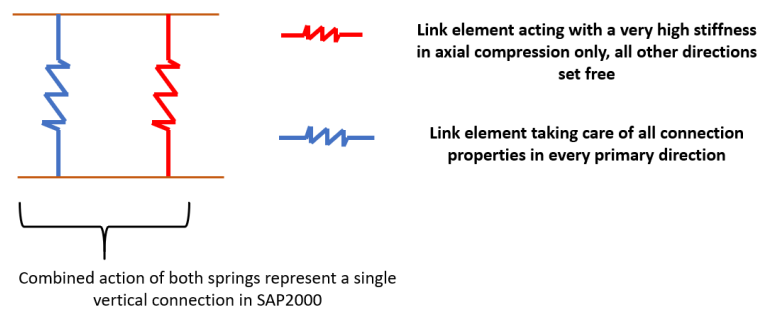


Figure 3-8 Representation of a single vertical connection in SAP2000

This approach of two link elements placed in parallel had to be followed in SAP2000 because of limitations in terms of the availability of material plateaus in SAP2000.

However, when modelling the structure in OpenSees, the vertical connection was represented using a single link element, which was fed a force–deformation plateau that could account for the different force v. deformation relationships in axial tension and axial compression. Accordingly, hysteretic material in OpenSees was employed with a higher stiffness in the axial compression direction and with the real properties of the connection in other directions (Figure 3-9).

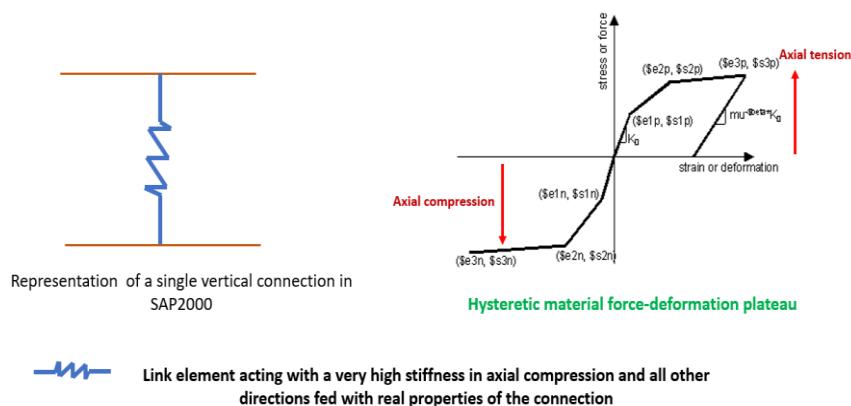


Figure 3-9 representation of a single vertical connection in OpenSees

Figure 3-10 illustrates the prototype modular building with all principal dimensions, and Figure 3-11 presents an extruded view of a single storey connected to upper and lower modules.

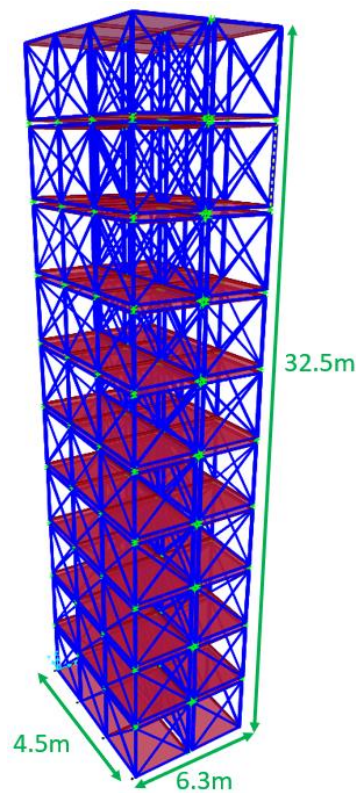


Figure 3-10 FE model of the prototype modular building

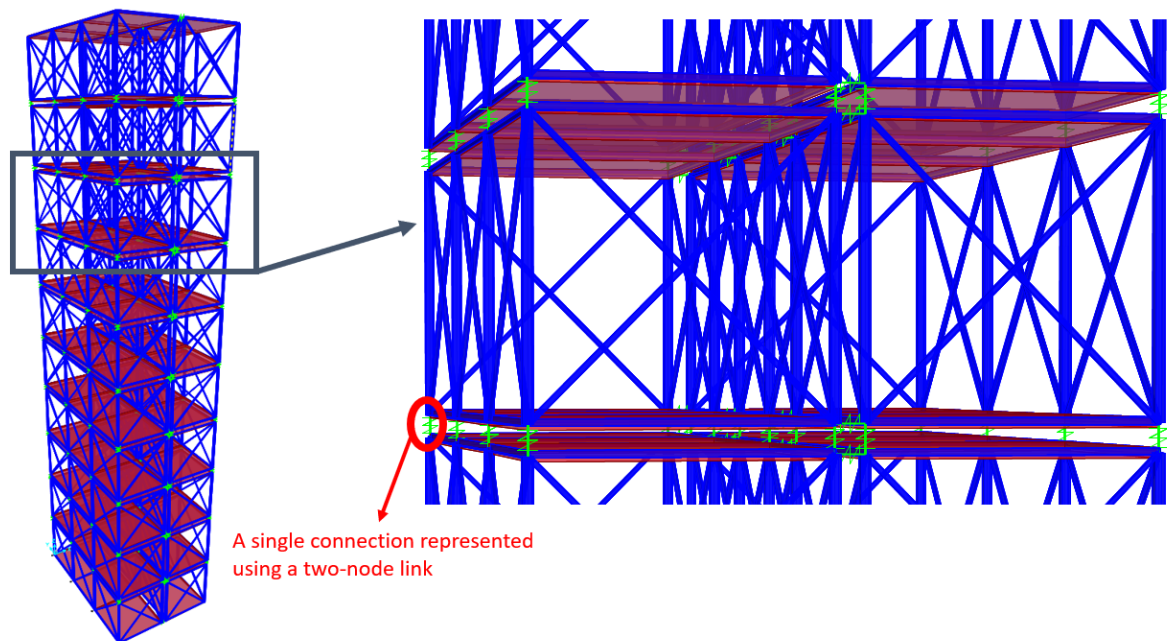


Figure 3-11 Extruded view of a storey showing connectivity between the modules

All frame elements in the modules were modelled with displacement-based beam-column elements, and section fibres were added to capture nonlinearity along the element.

3.1.2 Prototype Model of Conventional Steel Building (Control Model)

To add value to the results from the modular building analysis, a control model that represents a conventional steel building with the same dimensions and similar features as the modular building was developed (physically analogous). It was intended to serve as a benchmark building with which the modular building results could be compared to draw conclusions and provide recommendations.

The chosen structure was assumed to be entirely constructed with steel sections and comprised of concrete slabs as floor elements. I-sections with an outside height of 162 mm, top flange width of 154 mm and web thickness of 8 mm were chosen as the column sections. Beam sections were made of I-sections with an outside height of 150 mm, top and bottom flange width of 75 mm and web thickness of 5 mm. Concentric braces made of SHS of outside depth and height 50 mm and thickness 3 mm were fitted in all the storeys.

Since the building is ten storeys only, the same section properties were chosen for all storeys. Concrete slabs elements of depth 150 mm were employed as floor elements. Slab element masses were lumped at the nodes, and rigid diaphragms were assigned at storey levels to simulate the diaphragm action provided by slab elements. The mass of wall elements and other dead loads on the structure were also lumped at the nodes. In addition to the dead loads, a live load corresponding to general loading category of 3 KPa was added as per AS/NZS 1170.1[149]. Permanent (G) and imposed (Q) loads were combined ($1.2G+1.5Q$) using combination factors as per AS/NZS 1170.0 [150] to calculate the net loading on storeys.

The developed control model of the conventional structure was mainly intended to dissipate energy through yielding of braces. The frame was designed to fall into the class ‘ductile’ as per the ductility classification under Australian standards.

As can be seen from the FE models of both the conventional and modular structures, the main difference between them is the connections between modules, which cause a discontinuity in the modular structure (Figure 3-12).

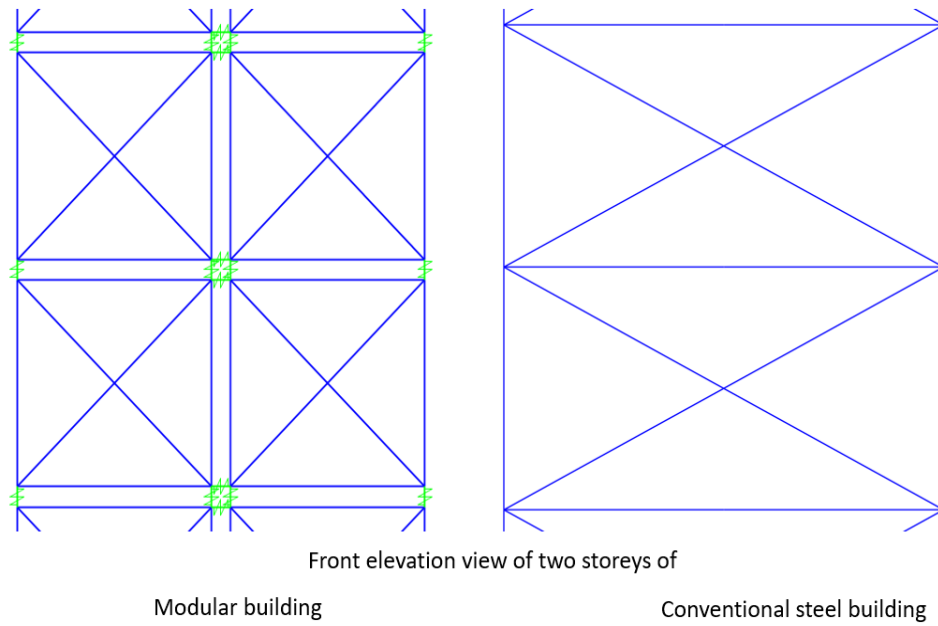


Figure 3-12 Comparison of the front view of two storeys of the modular and conventional steel buildings

Figure 3-13 illustrates the conventional steel building (control model) used in this study.

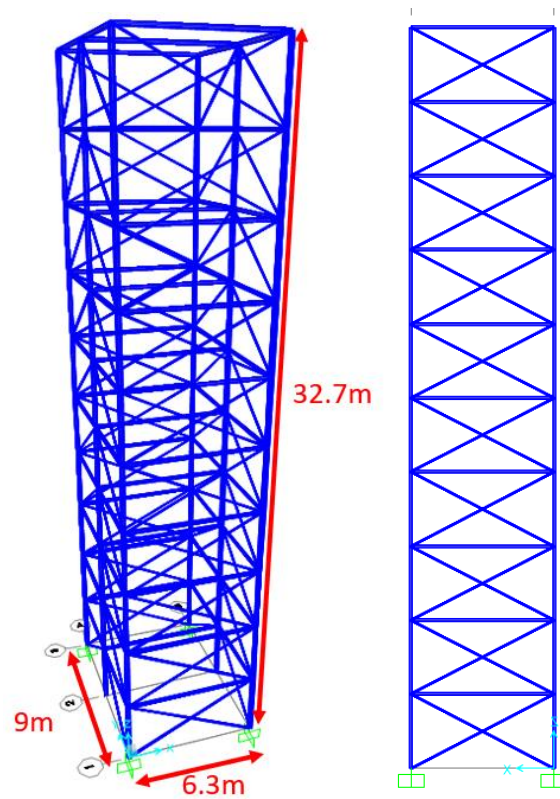


Figure 3-13 FE model of the prototype conventional steel building

3.1.3 Control Model of a Free-standing Superstructural Component of a Modular Building

It was revealed during this study that the connections of the first storey failed in an unzipping manner leading to a separation of the superstructure of the modular building above the first storey. With further increases in ground motion intensity, this superstructure component, which composed of a stack of modules forming nine storeys (all storeys above the first storey), behaved as a rigid body that rocked, slid, and overturned.

A control model made of a stack of nine storeys, exactly corresponding to the superstructure component of the modular building that overturned was developed to study and understand the reason behind the free-standing body overturning. The main aim was to compare the response of the free-standing control model, starting from rest, with that of the superstructure component of the modular building that collapsed, starting as a free-standing body with initial conditions obtained from the failure of the connections, under the same ground motion stretch. However, analyses were conducted on a scaled-down model of the prototype structure introduced in section 3.1.1. (see Figure 3-14). This analysis and the results obtained are discussed in detail in Section 4.4 and Section 6.4.1 .

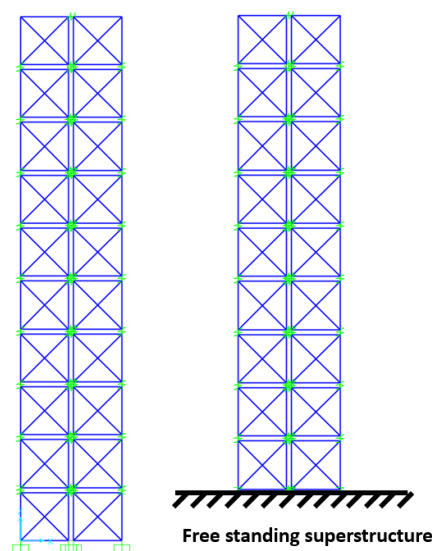


Figure 3-14 Representation of the free-standing superstructure component

3.1.4 Progressive Collapse Analysis in the Finite Element Models

The main aim of the study was to examine and compare the performance of a modular and a conventional steel building under increasing seismic intensities. Hence, analysing progressive

collapse was an essential component of the study. Many methods for progressive collapse analysis and simulation are discussed in Section 2.10. However, the approach explained in Section 2.10.4 was followed to analyse collapse in the developed FE models.

In this study, the progressive collapse was studied through FE modelling in OpenSees (Open Software for Earthquake Engineering Simulation) [153]. The primary approach employed was to monitor the stress or strain in all the elements and to remove a particular member from the structure once it reached the ultimate state. For both the modular and conventional steel buildings, all frame elements were modelled with fibre sections to incorporate nonlinear behaviour into the elements. All members were monitored at each time step during dynamic or static analysis to check whether any member had reached specified critical stress or strain values. If any member was found to have reached a specified critical value, that member was removed to simulate the real-life loss of the member through failure.

In contrast to frame elements, in which fibre sections provided the opportunity to track their nonlinearity, the connections in the modular buildings, which were modelled as link elements, were not equipped with a direct approach for tracking nonlinearity. Connections were capable of axial (push-pull action) or shear deformation, or a combination of these. Hence, it was vital to take into account the net effect of push-pull action and shear deformation in monitoring for the failure of connections.

There are many failure identification criteria available based on material properties. The Tsai-Hill yield criterion has been explicitly used for anisotropic materials [154] and was applied in this study to identify the failure of connections. This can be expressed as in Eq 3-2 [154]:

$$\left(\frac{\delta_{Axial}}{\delta_{Axial\ ultimate}}\right)^2 + \left(\frac{\delta_{Shear}}{\delta_{Shear\ ultimate}}\right)^2 - \frac{\delta_{Axial} \times \delta_{Shear}}{\delta_{Axial\ ultimate} \times \delta_{Shear\ ultimate}} \leq 1 \quad \text{Eq 3-2}$$

Here δ denotes deformation or any damage parameter of interest. In the FE model, the condition in Eq 3-2 was checked for every connection, at every time step of the dynamic time history. Any connection not satisfying the condition was assumed to have failed and was no longer functional. If a connection was indicated to have reached the critical value, the connection was removed from the structure to simulate the failure of the connection before proceeding to the next time step. This approach falls into the category of FE models predicting collapse on a fine model, as discussed in Section 2.10.2. However, modelling progressive collapse using this approach on the fine model of the modular and conventional building only analysed stage 1 of

the collapse, as discussed in Section 2.10.4. Simulation of stage 2 of the collapse is discussed in detail in Chapter 6.

3.2 Finite Element Analysis of the Prototype Model of Modular Building

The initial phase of this study focussed on studying the structural feasibility of the modular building in terms of code requirements. The approach followed previously [12, 68] to evaluate the feasibility of a pure modular system was used as a guide for conducting the necessary analysis as per Australian standards.

3.2.1 Modal Analysis

Analysing the structures began with the most basic form of studying the compliance of a structure, Modal analysis. Every design code provides specifications for the modal period requirements of structures of different heights and different types. Modal analysis results for the modular structure are summarised below (see Figure 3-15).

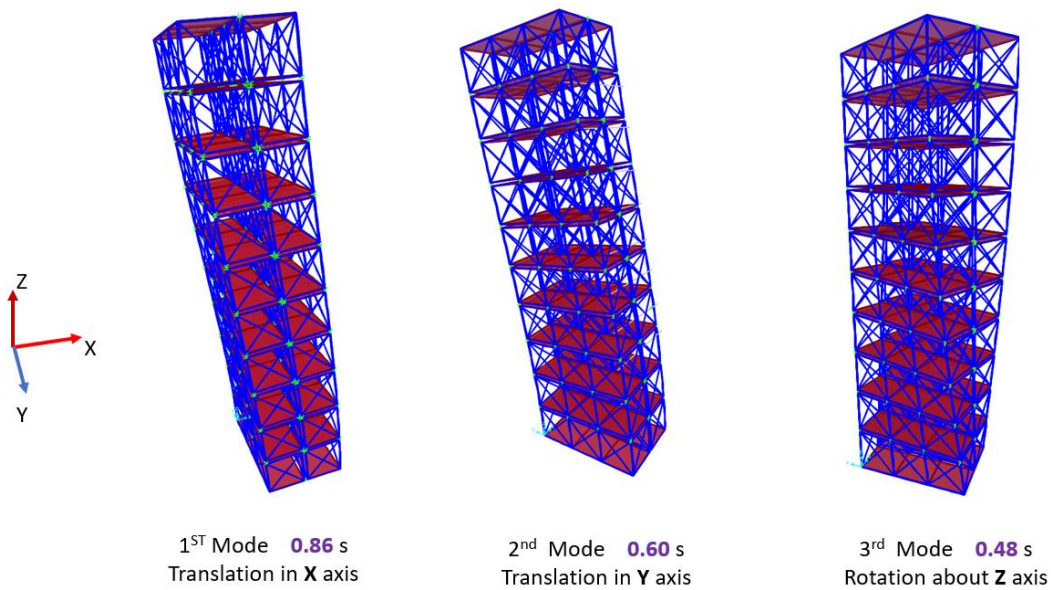


Figure 3-15 Modal analysis results of the prototype modular structure

The first mode period value obtained was checked against the AS1170.4 [69] fundamental equation as follows:

$$T_1 = 1.25k_th_n^{0.75} \quad \text{Eq 3-3}$$

Here T_1 , k_t and h_n refers to the fundamental period, a factor depending on the structure considered, and the height of the structure, respectively. Since this structure does not fall into

the general steel or concrete building category, $k_t = 0.05$ can be assumed considering as ‘other structures’. This will result in a first mode period value of $T_1 = 0.85$. This is in perfect agreement with the FE model result for the modal period value. Moreover, the modal shapes obtained for these structures resemble those of conventionally available general structures.

3.2.2 Pushover Analysis

As the next step of the study, a pushover analysis was conducted to understand the dynamic behaviour of the developed structure and a single module of the type used in the study, prior to any rigorous nonlinear analysis. All analysis was conducted in OpenSees software.

3.2.2.1 Pushover Analysis on a Single Module Fixed to the Ground

It is highlighted throughout this thesis that the nature of the construction of modular buildings is entirely different from that of conventional constructions as it involves an alternate arrangement of modules and connections. Thus, it is important to be aware of significant properties such as the stiffness and strength of each of the major components when drawing important conclusions about a structure’s performance. A pushover analysis was conducted on a single module to understand its strength and stiffness characteristics.

As expected, and intended, with increased roof displacement relative to the base, the in-built LFRS in the module (i.e., braces) failed first. With further increases in roof deformation, the module columns began yielding until failure. The pushover curve obtained is shown in Figure 3-16. This can be employed to estimate the stiffness and strength of the module.

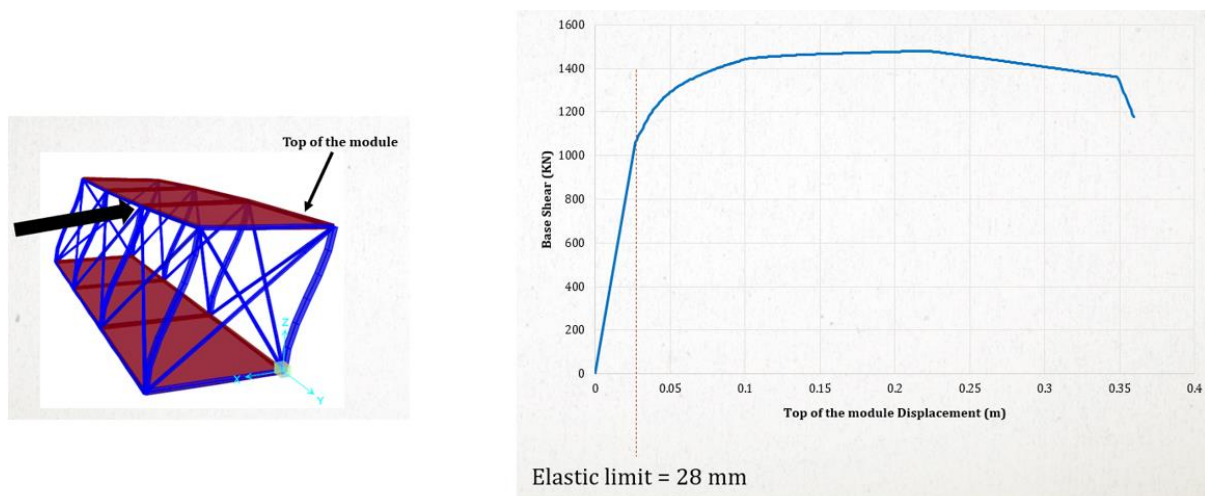


Figure 3-16 Pushover analysis of a single module

3.2.2.2 Pushover Analysis of the Full Building

The modular structure was subjected to a displacement-controlled pushover analysis in OpenSees. The roof displacement was increased until global instability occurred in the structure, and the program terminated.

Figure 3-17 illustrates the pushover curve obtained. It was observed during the analysis that the deformation of the building was mainly governed by the inter-modular connections, especially in the first storey. Despite yielding of 2-4 bracings in the first two storeys, significant damage or deformation was not observed in the modules. The failure initiated from the first-storey edge connections, resulting in higher inelasticity concentration at the other connections of the same storey. This resulted in subsequent connection failures in the same storey, causing an unzipping of the structure across that storey.

This observation raises a warning regarding the performance of the structure under dynamic events, which can be catastrophic due to failure that revolves around connections rather than spreading throughout the building. The analyses in subsequent chapters aimed to verify this observed hazardous collapse nature through experimental and numerical studies under seismic events.

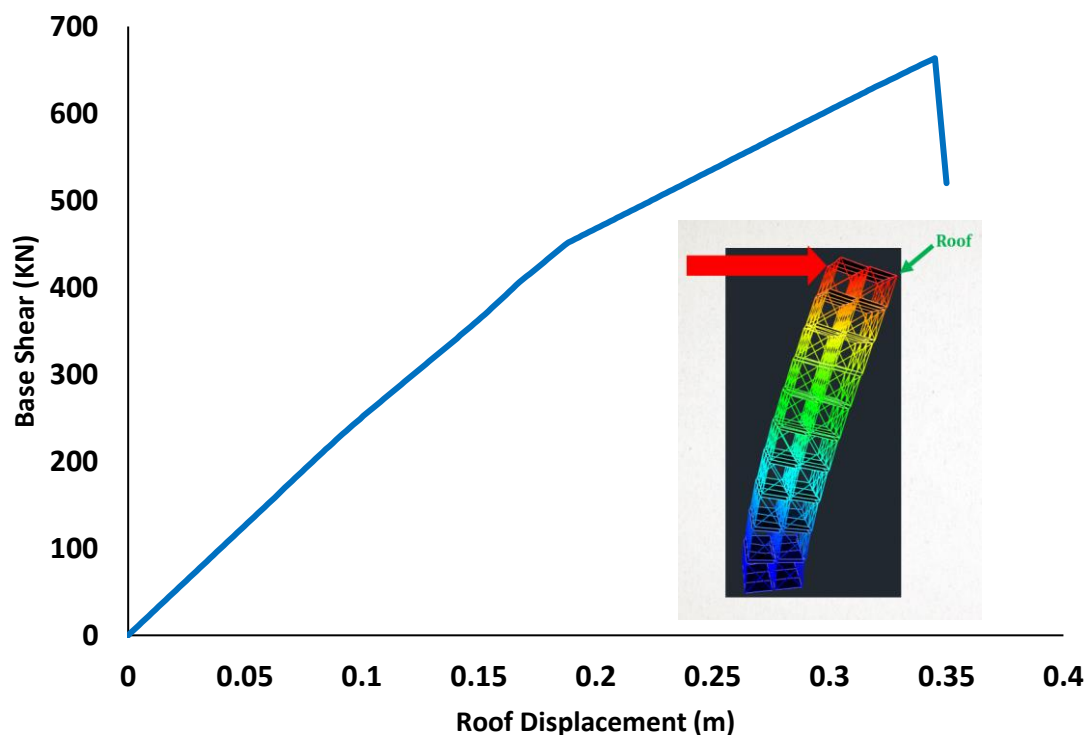


Figure 3-17 Pushover analysis on the prototype modular building

3.2.3 Capacity Spectrum Analysis

The capacity spectrum method is a useful method for checking the performance of a structure under a particular design condition. This method and the approach used are discussed in detail in Section 2.7.1.2. The capacity curve obtained from the pushover curve generated in Section 3.2.2.2 was compared against three demand curves generated from three ground motions scaled to the Melbourne-based target spectrum for soil class C, and a hazard return period of 500 years. The chosen ground motions are listed in Table 3-3. The response spectrum of selected ground motions scaled to the target spectrum is shown in Figure 3-18.

Table 3-3 Ground Motion Records Selected

Ground motion name	Moment magnitude	PGA
Imperial valley-06	6.5	0.29
Chi-Chi	6.2	0.07
El Mayor	7.2	0.39

It can be inferred from Figure 3-19 that the capacity of the structure at the yield point was always above demand under the three chosen ground motions (the first yield in the structure is out of the demand curve). This implies that the structure can perform safely under these three ground motions when scaled to Melbourne ground conditions.

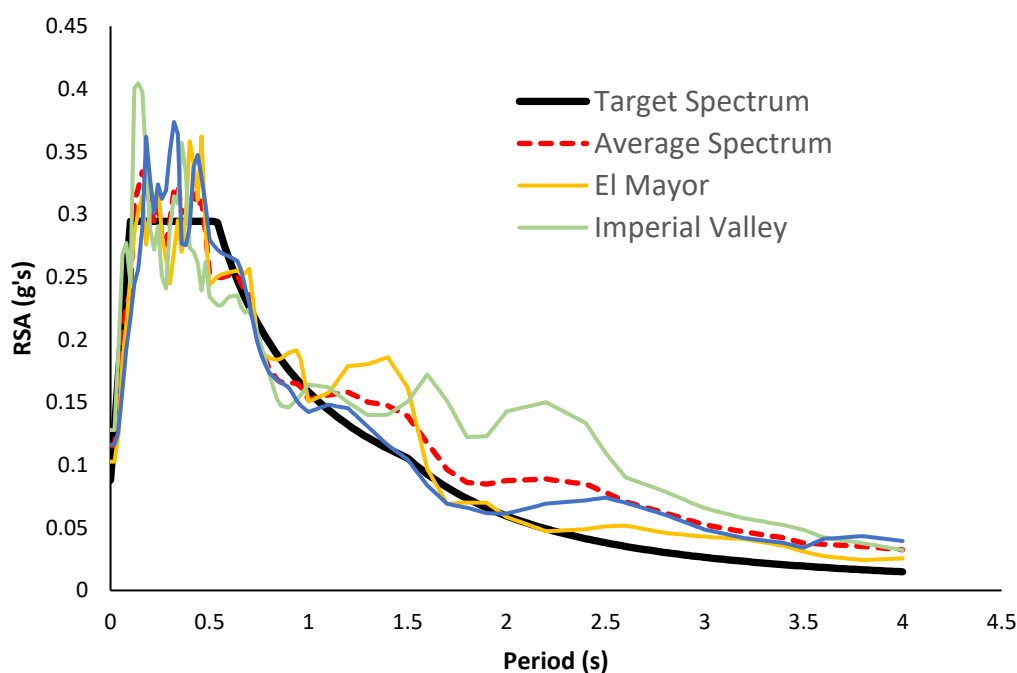


Figure 3-18 Ground motions scaled to the target spectrum

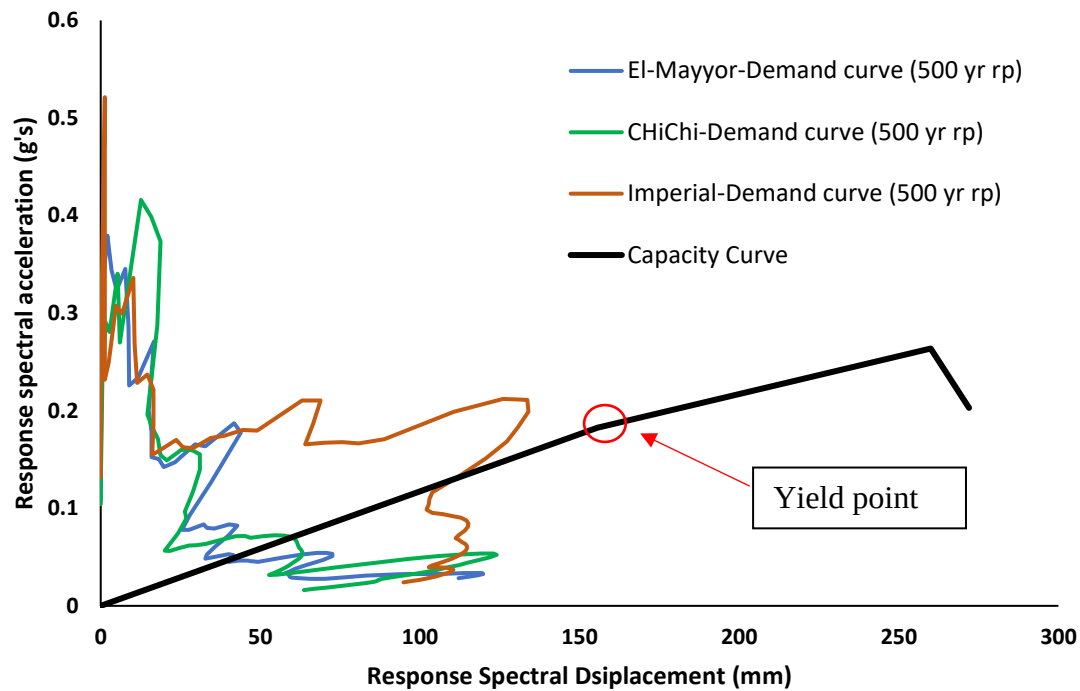


Figure 3-19 ADRS diagram for the prototype modular building

3.2.4 Dynamic Analysis

In addition to the analysis conducted above, a dynamic THA was also conducted to achieve a more accurate interpretation of the performance of the modular building under dynamic conditions. As discussed under section 2.9, an ETHA was conducted rather than an IDA to study the performance behaviour of the structure from rest to collapse.

3.2.4.1 Dynamic Time History Function (Endurance Time Excitation Function)

The ETEF used in this study was extracted from the ETEF database created by Estekanchi [116]. Figure 3-20 illustrates the variation in acceleration with time for this excitation function.

This record was generated by optimising to fit the average acceleration spectra, average displacement spectra and average cumulative absolute velocity (CAV) of the first component of the Federal Emergency Management Agency (FEMA) P695 far-field record set. This ETEF follows an exponentially intensifying profile that is consistent with CAV when compared with conventional ground motion scaling to match the desired intensity. The ground motion was 40 s in duration [155]. Further details on the development of this ETEF can be found in Ref. [155]

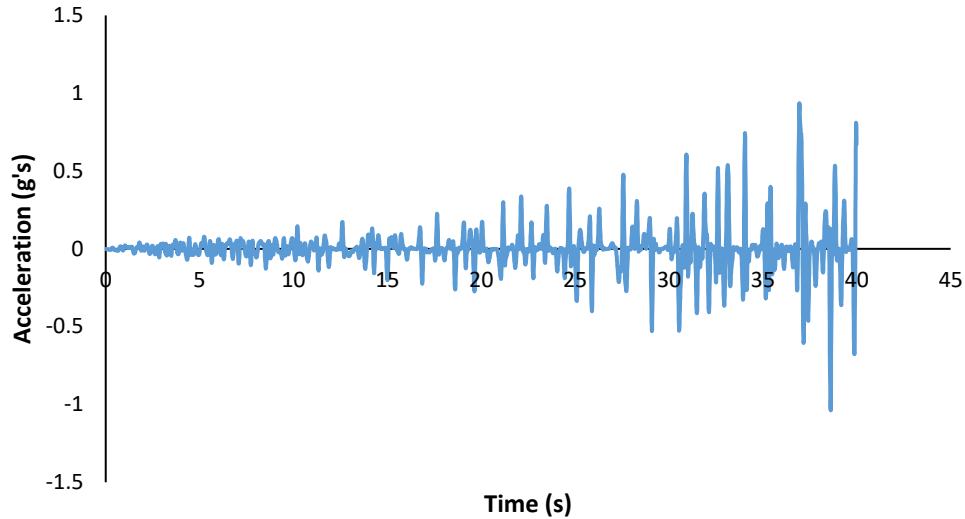


Figure 3-20 The ETEF used in this study [116]

3.2.4.2 Behaviour of the Prototype Modular Building Under the ETEF

3.2.4.2.1 Observations During the ETHA of the Modular Building

The main observations on the behaviour of the structure under ETEF are summarised in Figure 3-21. In addition, a video of the behaviour of the structure from rest until collapse can be accessed at <https://youtu.be/clBrX40jUY0>.

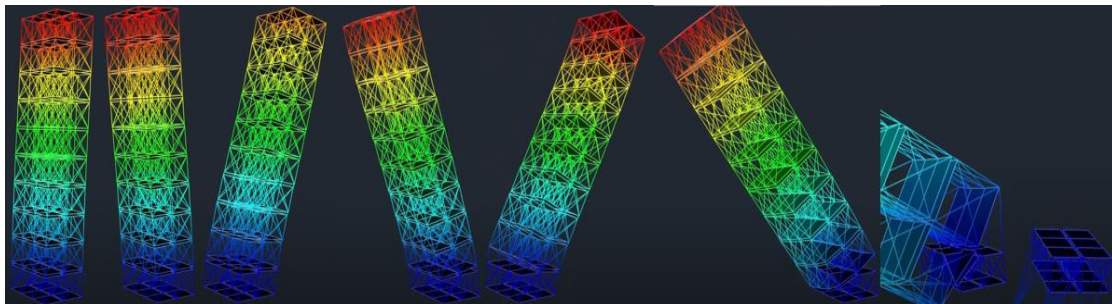


Figure 3-21 Behaviour of the structure at different time steps

The OpenSees program developed was set to monitor every element in the structure, which includes the beams, columns, braces and inter-modular connections, and to detect and display the failure of any of these elements, and the time at which failure occurred. The observations under the ETEF were aligning very well with those seen during the pushover analysis. Failure initiated from the edge connections, which resulted in other connections of the same storey being more vulnerable to failure. After the failure of first-storey edge connections, failure progressed to other connections of the same storey, which became critical and resulted in an

unzipping of the structure around the first storey. The lateral force-resisting braces built into the modules were on the safe side, with the exception of four braces of the first- and second-storey modules that yielded but did not result in any significant damage to the module.

3.2.4.2.2 Variation in Damage Under the Endurance Time Excitation Function

The main aim of this study was focussed on the variation in global damage in the structure over time. The primary concern when analysing the global collapse of a structure is structural damage to the frame. Hence, roof displacement of the structure was selected as the primary damage parameter to be monitored during the analysis. The variation in roof displacement with time is presented in Figure 3-22.

As highlighted in Chapter 2, time in endurance time history functions is proportional to seismic intensity. Thus, it is meaningful to convert the time axis to the seismic intensity and obtain variation in displacement with seismic intensity. The plot thus obtained is similar to the curve obtained through rigorous IDA by applying a ground motion scaled to different intensities.

Many studies have focussed on converting time histories to IDA curves using different approaches. Some are discussed in Section 2.9.2 [109, 114, 120, 123].

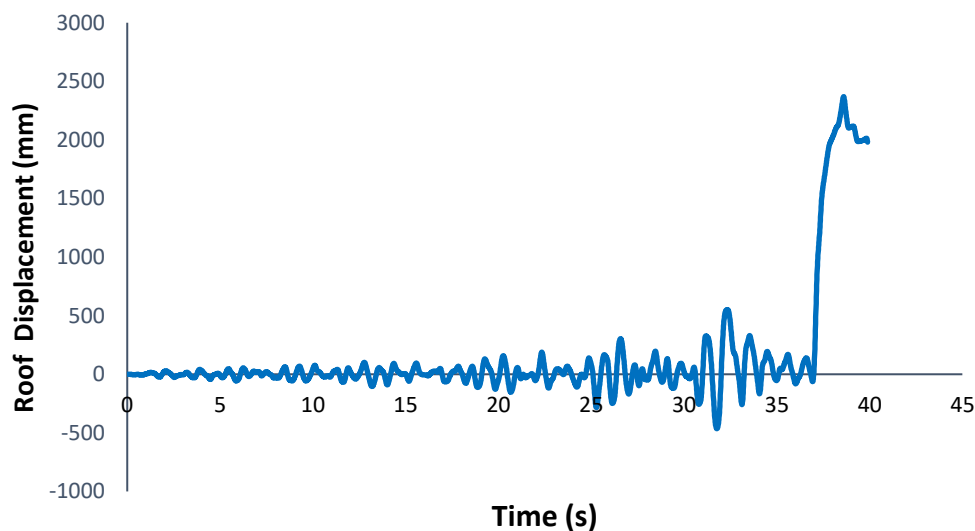


Figure 3-22 Roof displacement time history under ETEF

In this study, the spectral acceleration at the first mode period of the structure was chosen as the IM. In an IDA curve, the maximum value of the damage parameter corresponding to a specific seismic intensity is plotted against seismic intensity.

The roof displacement time history obtained from ETHA is used to identify peaks (as shown in Figure 3-23) in the time history; the seismic intensities corresponding to these peaks are then picked to plot the corresponding IDA curve.

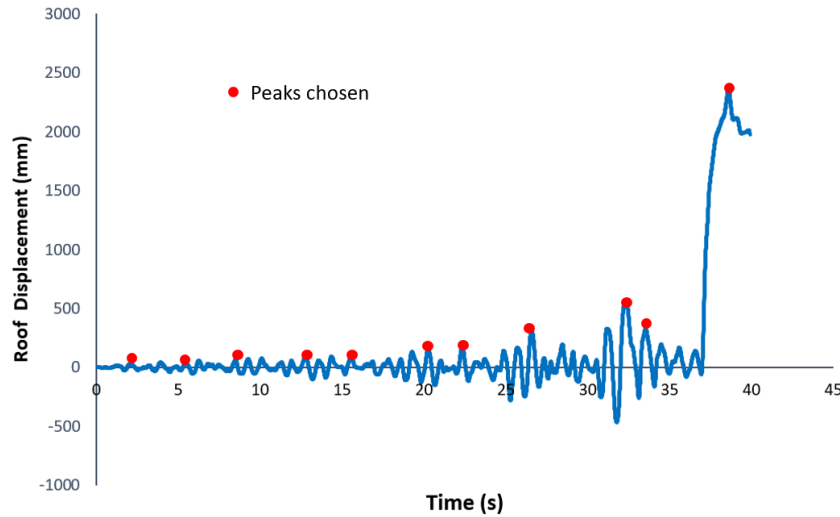


Figure 3-23 Chosen peaks along with the displacement time history

To calculate the seismic intensity corresponding to different time values that cause the peaks, the response spectrum corresponding to the time range ending from the time that requires the corresponding seismic intensity is first generated. Once the response spectrum has been generated, the spectral acceleration value corresponding to the first mode period of the structure is obtained as the seismic intensity corresponding to that time (e.g. consider requiring finding the seismic intensity corresponding to the 20th second of the time history; the steps are shown in Figure 3-24). Once these points are obtained, the IDA curve can be obtained by plotting the spectral acceleration at the first mode period (seismic intensity) against roof displacement (damage parameter) as shown in Figure 3-25.

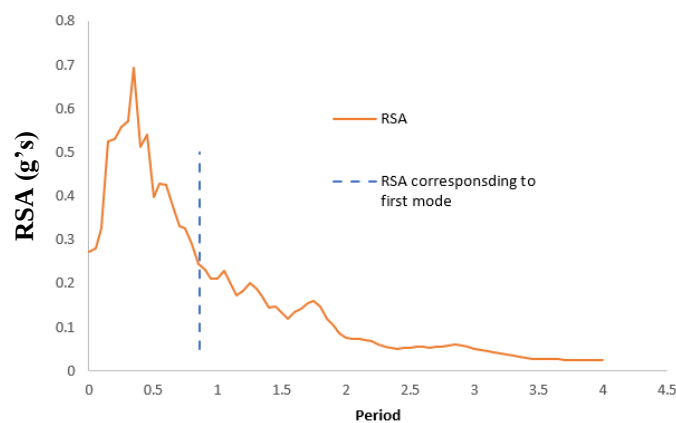


Figure 3-24 Response spectrum corresponding to the time history up to 20 s

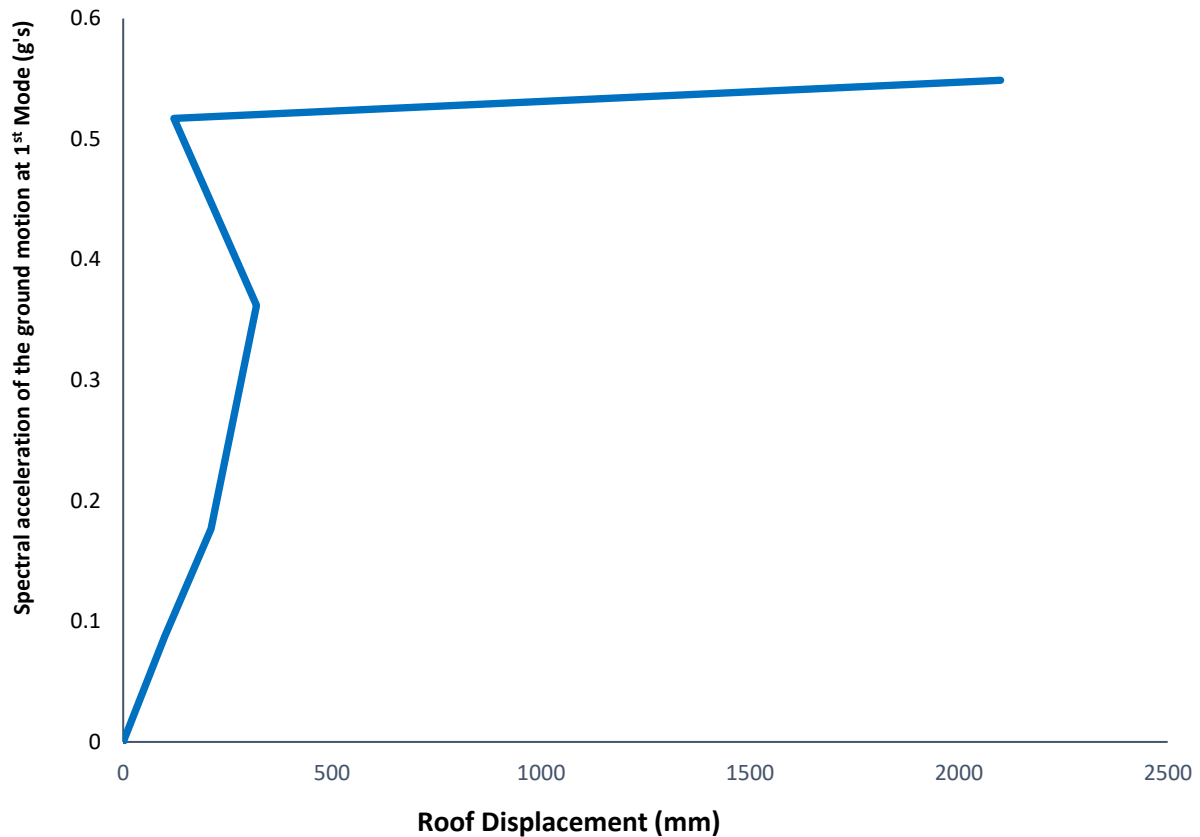


Figure 3-25 IDA curve for the prototype building under ETEF

It can be seen in Figure 3-25 that there is a sudden increase in damage in the modular building at some point during ETEF application. The sudden increase in damage occurs around a seismic intensity, spectral acceleration value of 0.5 g, which is around a hazard return period of 2500-years. Moreover, the structure was safe and performed perfectly normally under a hazard return period of 500-years, which corresponds to a spectral acceleration value of around 0.25 g.

Figure 3-26 depicts the roof displacement time history with critical time values at which the first and last connection failures occurred in the structure. A very short time is apparent between the first and last connection failure, indicating that the failure of the first connection has a significant effect on the failure of other connections in the same storey.

Figure 3-27 (b) represents the roof displacement time history within the elastic range only. It can be estimated from the maximum elastic displacement shown in the plot that the maximum elastic drift is around 1%, which is an acceptable value for most structures. This maximum elastic drift can be theoretically verified via hand calculation, as demonstrated in the next section.

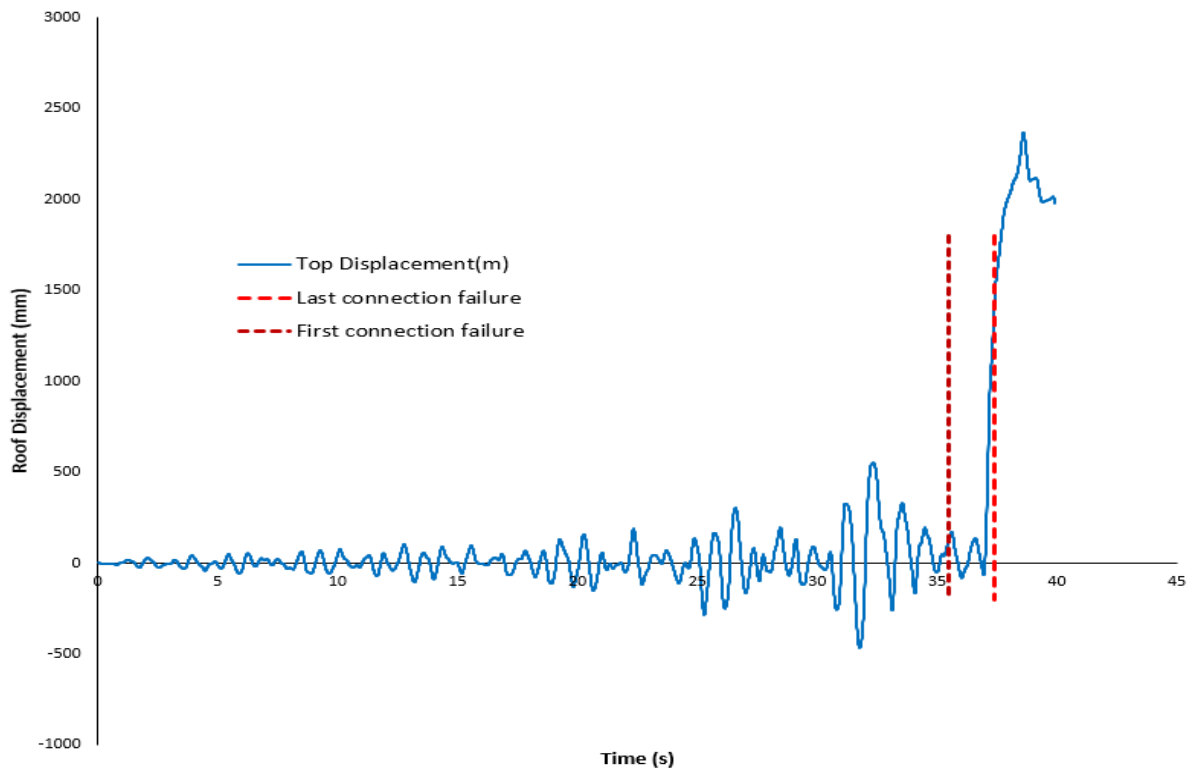


Figure 3-26 Damage time history denoting first and last connection failures

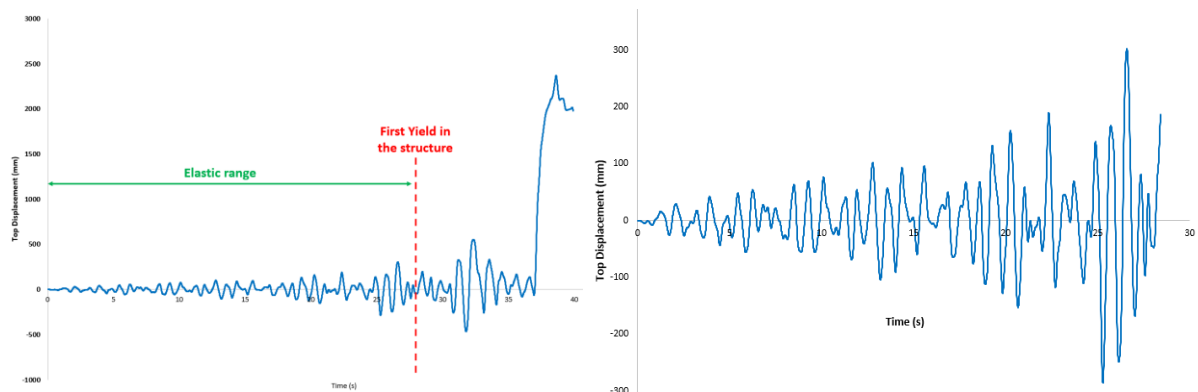


Figure 3-27 (a) Two distinguished ranges in the displacement time history and (b) elastic range only of the displacement time history

3.2.4.2.3 Theoretical Calculation of Maximum Possible Elastic Drift in the Modular Building

The total drift occurring in the modular structure can be decomposed into three primary sources of deformation: shear deformation of connections, racking deformation of modules, and horizontal displacement due to axial deformation of connections, as shown in Figure 3-28.

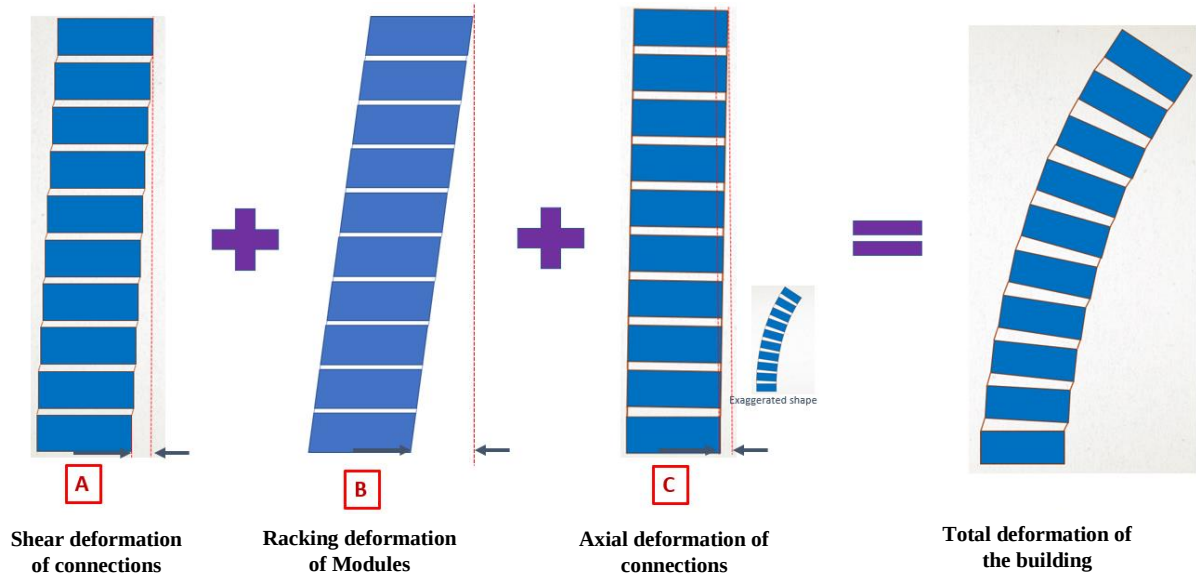


Figure 3-28 Composition of total lateral deformation of the building

It can be deduced from Figure 3-6 and Table 3-1 that the horizontal deformation component per storey due to shear deformation of connections is 4 mm (elastic). The contribution from racking deformation of modules per storey can be estimated as 28 mm from the linear part of the pushover analysis curve for a single module shown in Figure 3-16. Finally, the contribution to the lateral deformation of a storey from axial deformation of connections can be computed using the following approach, as summarised in Figure 3-29:

$$\Delta_{Axial_max} = H_m \times \frac{\alpha L_c}{B} \quad \text{Eq 3-4}$$

The axial strain in the connection, α , can be calculated using the maximum value of axial deformation per connection, which is calculated and tabulated in Table 3-2. Calculation using Eq 3-4 shows that the maximum horizontal deformation per storey due to the axial deformation of one connection is 0.5 mm.

The final theoretical maximum elastic drift can be computed by summing the three components contributing to the drifts, as shown below (initially deformations are computed and then converted to drift values. Multiplication by a factor of $\cos(45)$ was carried out to account for the fact that maximum axial and shear deformation does not occur at the same time):

$$\Delta_{Shear} + \Delta_{Racking\ of\ modules} + \Delta_{Axial} = \Delta_{total} \quad \text{Eq 3-5}$$

$$\Delta_{Racking\ of\ modules} \times \text{No of stories} + \cos(45) \times (\Delta_{Shear} + \Delta_{Axial})_{max} \times (\text{No of stories} - 1) = \Delta_{total}$$

The maximum elastic roof displacement, $\Delta_{total} = 309$ mm and maximum elastic roof drift = 1%. Hence, the maximum elastic drift obtained using numerical analysis is in excellent agreement with that obtained through theoretical calculations.

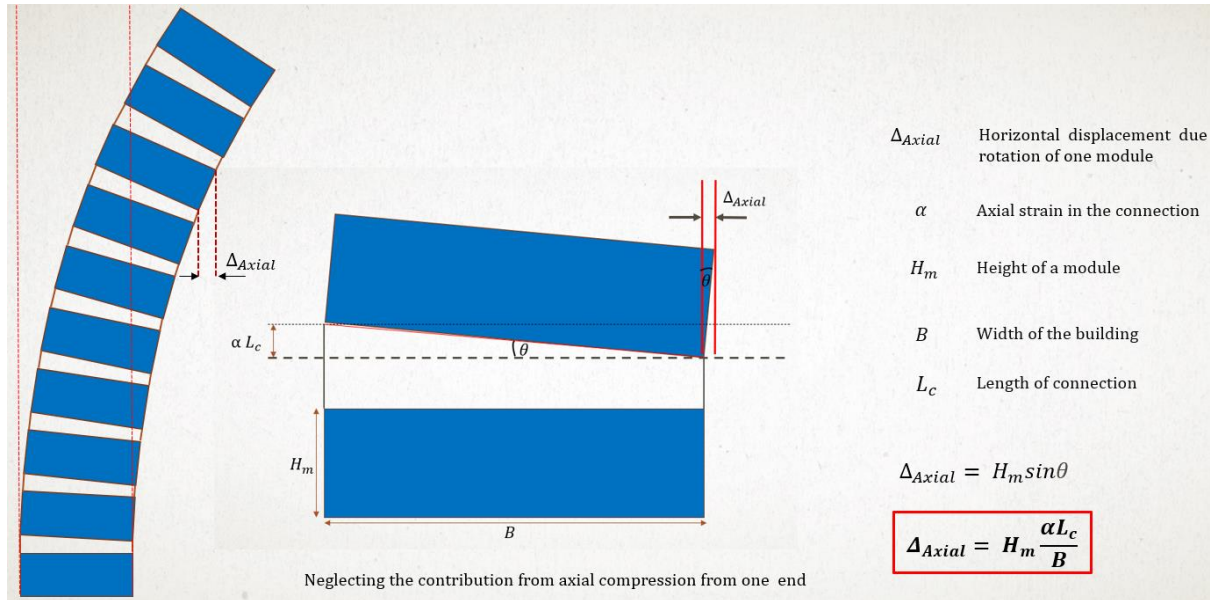


Figure 3-29 Calculation of horizontal deformation per storey due to axial deformation of connections

3.2.5 Feasibility of the Pure Modular Building Considered

In this chapter, a prototype pure modular building of 10 storeys was introduced and investigated for structural feasibility in regard to code requirements. The structure provided highly satisfactory in terms of modal periods, mode shapes, capacity curve and dynamic performance. These positive results are promising for the chosen prototype building to be a feasible pure modular system similar to the advanced corner-supported system studied in Ref. [68].

Despite these very positive outcomes within code provisions, an undesired outcome was observed in the performance of the structure, when the seismic intensity exceeded the design limit (considering a design hazard return period of 500 years). At higher seismic intensities, the structure collapsed in a dangerous manner, raising concerns about whether designing such a structure to meet conventional code requirements is sufficient to ensure its safety under extreme dynamic conditions.

To elaborate on the risk seen with the modular building when a seismic event exceeding the design limit occurs, the study was repeated on the prototype model of the conventional structure

developed in Section 3.1.2. All analyses conducted on the prototype model of the modular building were repeated for the prototype model of the conventional building.

3.3 FE Analysis of the Prototype Conventional Steel Building

Since structural configurations and design patterns are very well established for conventional structures, there was no requirement to investigate the feasibility of the structural system as a whole.

However, to verify the modelling assumptions and other structural choices made, this structure was also subjected to all analyses undertaken on the prototype modular structure.

3.3.1 Modal Analysis

The fundamental modal periods and mode shapes are summarised in Figure 3-30 below. The first mode period was compared against AS1170.4 [69] based Eq 3-3 ($T_1 = 1.25k_th_n^{0.75}$) specifically for braced frames ($k_t = 0.06$). the period was computed to be 1.0s based on the equation suggested in the code. The numerically obtained period (see Figure 3-30) was in fair agreement to the code-specified value. Moreover, the mode shapes indicated that the structure was similar to other conventional structures.

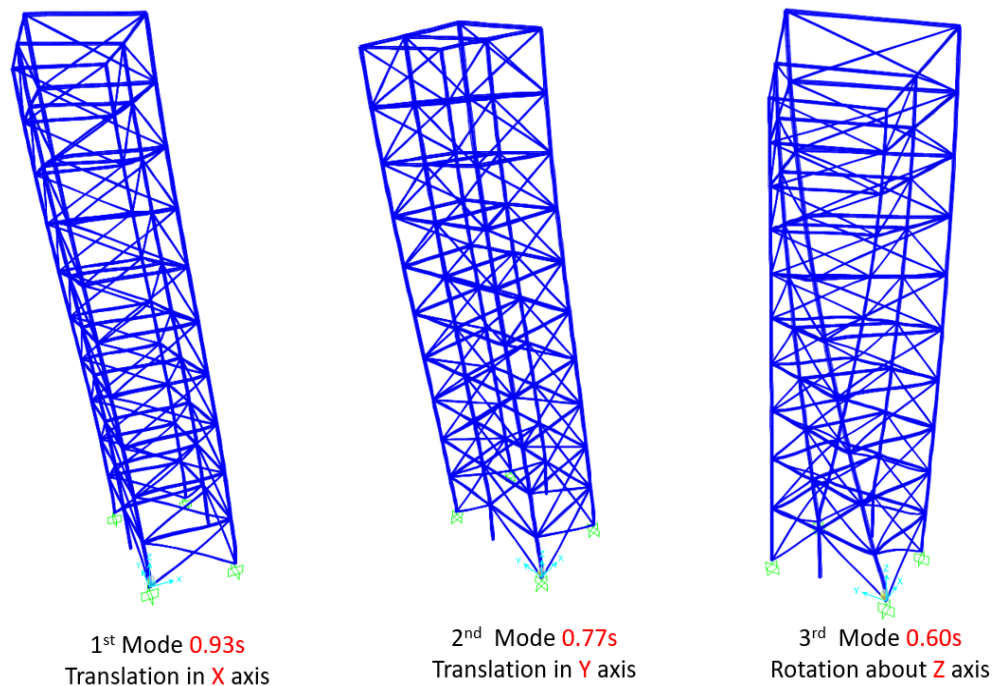


Figure 3-30 Primary modal periods and mode shapes of the conventional steel structure

3.3.2 Pushover Analysis

A displacement-based pushover analysis was conducted in OpenSees on the prototype model of the conventional steel frame. Figure 3-31 presents the pushover curve obtained for the conventional steel frame.

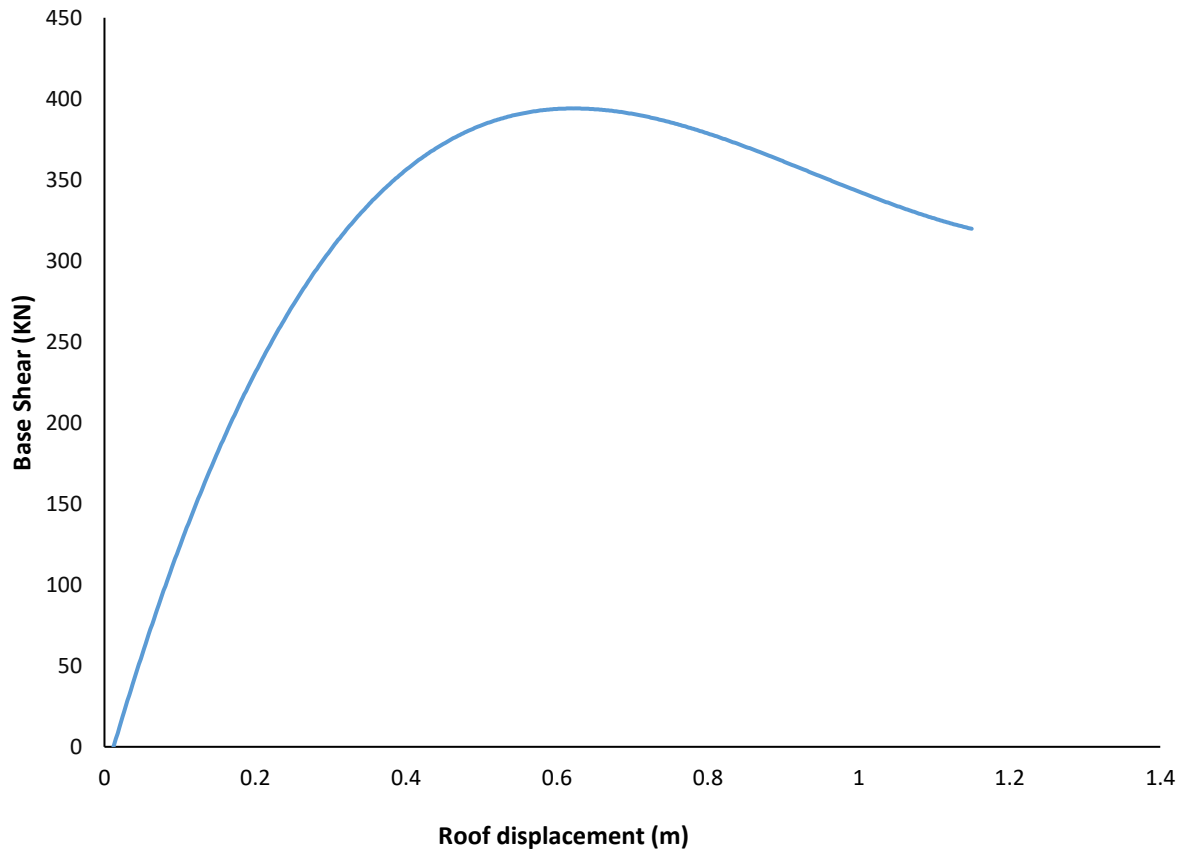


Figure 3-31 Pushover curve of the conventional steel building

It was observed during the pushover analysis that failure initiated from the yielding of lateral force-resisting braces and progressed along the building height starting from the ground storey braces. Once brace yielding was complete to a large extent, the yielding of ground storey columns initiated, progressing to the other storeys. In this way, the failure in the conventional structure was more evenly distributed rather than causing catastrophic failure, like in the modular building.

Apart from these observations, the progressive nature of failure distribution was evident from the smoothness of the pushover curve, which exhibits a highly ductile plateau. In contrast, the pushover curve for the modular building presented in Figure 3-17 illustrates a brittle nature of damage variation.

3.3.3 Capacity Spectrum Analysis

A capacity spectrum analysis was conducted on the conventional steel frame, employing the ground motions listed in Table 3-3, which were scaled to match the target spectrum for Melbourne, as illustrated in Figure 3-18. The capacity curves obtained from the pushover curve in Figure 3-31, combined with the demand curves obtained, is shown in Figure 3-32. It can be concluded from the ADRS diagram that the structure performed safely under all three ground motions considered.

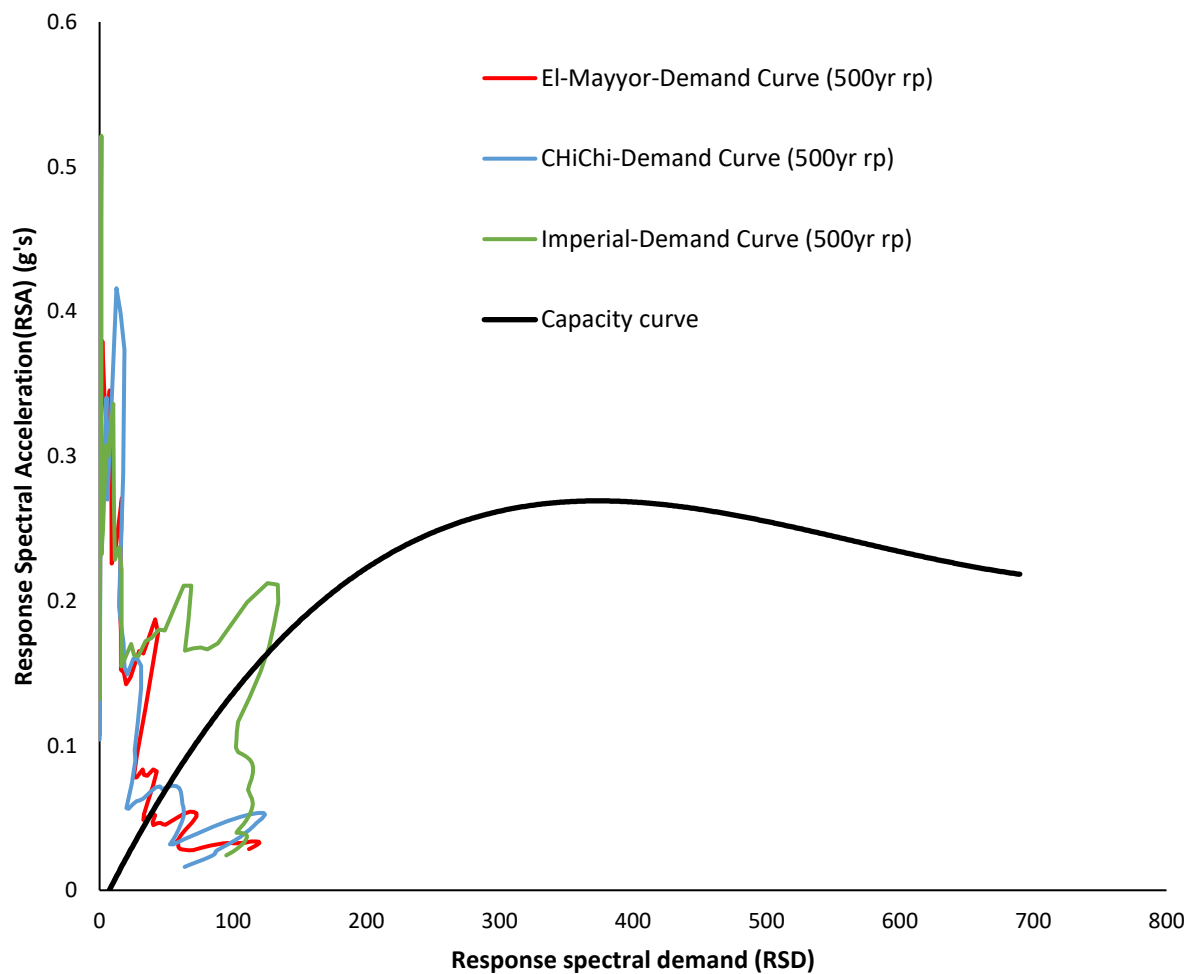


Figure 3-32 ADRS diagram corresponding to the conventional steel structure

3.3.4 Dynamic Analysis

Following the sequence of analyses conducted on the prototype modular structure, a dynamic THA was conducted on the prototype model of the conventional steel frame, subjecting the structure to the same ETEF as introduced in Section 3.2.4.1.

3.3.4.1 Behaviour of the Prototype Conventional Steel Building Under the ETEF

Nonlinear dynamic THA was conducted on the OpenSees model of the conventional steel frame. With increasing seismic intensity, the structural failure progressed in a similar fashion to that observed in the pushover analysis. The failure initiated from the yielding of braces on the ground floor and then the yielding of braces progressed up along the storeys. In response to the gradual loss of braces due to yielding, the ground storey columns became critical, and they started to yield. The failure along the building height was more distributed rather than being concentrated to a single location.

Figure 3-33 presents the observed roof displacement time history, which was subsequently converted to the corresponding IDA curve depicted in Figure 3-34. The IDA curve and the roof displacement time history indicate no sudden damage variation, as was observed for the modular building. The damage is more gradually distributed with seismic intensity.

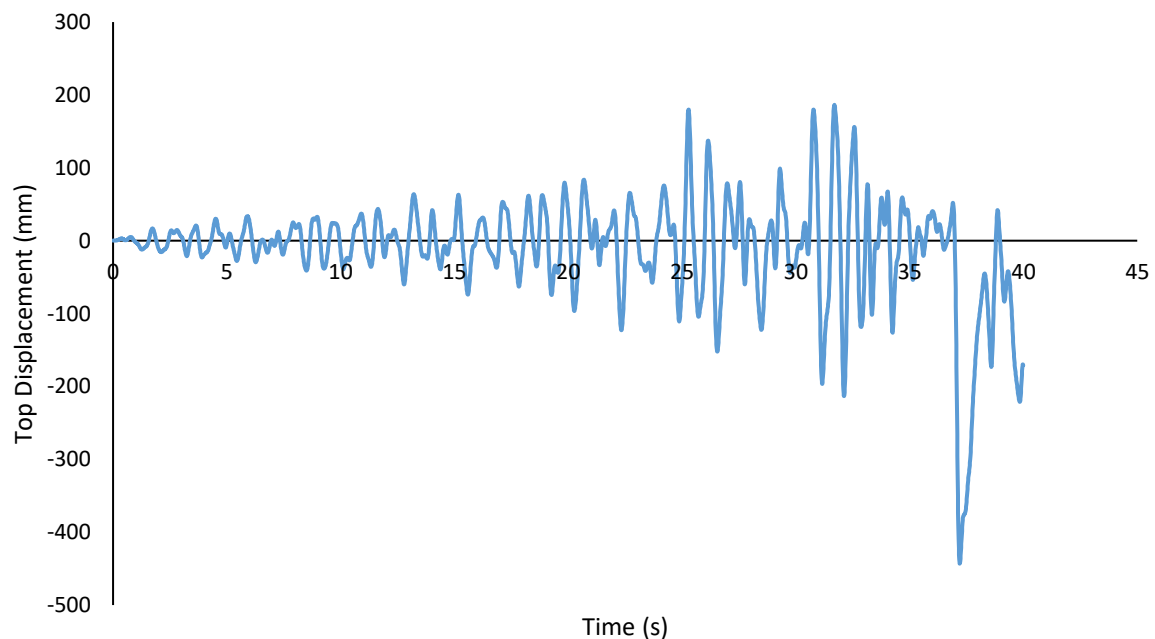


Figure 3-33 Roof displacement TH of the conventional steel building under the ETEF

An RSA value of around 0.25 g corresponds to a hazard return period of 500-years, and an RSA value of around 0.5 g corresponds to a hazard return period of around of 2,500 years. It can be concluded from Figure 3-34 that the structure did not undergo any abnormal increase in the rate of progression of damage under a seismic intensity higher than the design level.

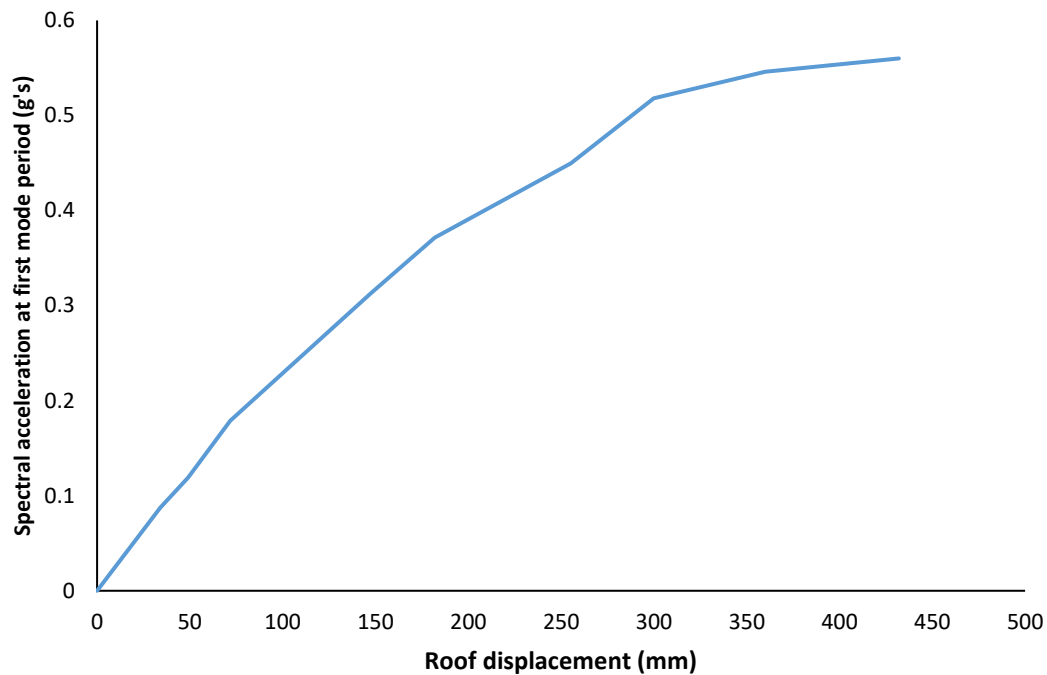


Figure 3-34 Corresponding IDA curve for the conventional steel frame

It is evident from the results from FE analysis of the modular and conventional steel buildings that a pure modular building designed to code provisions can be safe under static loadings and dynamic loadings within design limits. However, it is clear from the results that when design limits are exceeded, the conventional building still failed in a safe, progressive manner. In contrast, in the modular building, damage spread rapidly, causing a massive section of the superstructure to completely separate from the first storey in an unzipping manner across the storey. It was also noted from the IDA plot that the damage (roof displacement) increased rapidly as soon as connection failure initiated. This was hypothesised at the beginning of the study based on the construction nature of modular buildings.

Given these observations for the FE model of the prototypes, this study was extended to develop scaled-down models of the prototype modular and conventional buildings to study their performance during shaking table experiments.

3.4 Summary and Conclusions

A summary and main conclusions drawn from this chapter are presented below.

- In this chapter, the feasibility of a mid-rise pure modular building (without the requirement for any external LFRS) was explored.

- Modal, pushover, capacity spectrum and nonlinear dynamic analyses conducted on the structure indicated satisfactory performance in terms of all code requirements, within the design limits. Hence, the structure was deemed feasible in terms of code requirements.
- Pushover and dynamic analyses revealed that the modular structure failure initiated from the first-storey edge connection and suddenly progressed to the subsequent connections of the same storey, causing failure of all connections in that storey, in an unzipping manner.
- Variation in damage to the structure plunged as soon as the first connection failure occurred.
- The failure of all connections in the first storey resulted in the modular structure separating into two components: a ground component consisting of the ground storey modules; and a stack of nine storeys of modules freely standing on these ground modules. However, the FE model was not able to capture the response of the superstructure component, which separated due to the failure of all the connections in the first storey, under ongoing ground motion. Hence, further analysis involving experimental and numerical models is conducted in the following chapters.
- Pushover and dynamic analyses on the conventional steel frame indicated that failure in the structure was very much distributed throughout the structure and occurred in a progressive manner, unlike what was observed for the modular structure. The progressive and sudden nature of the damage progression in the conventional steel frame and modular structures was abundantly clear from the smooth and sudden (ductile and brittle) nature of their pushover and IDA curves.
- Although both the modular and conventional structures performed safely under design seismic conditions, the modular structure experienced a catastrophic failure mode, with a sudden increase in damage variation in the structure, under lateral loadings exceeding the design limit.
- Experimental and further numerical analysis is required to understand the collapse behaviour of the modular building after all the connections fail.

Chapter 4: Experimental Tests on Scaled-down Models

The results from Chapter 3 indicate that the collapse involved in the modular structure was entirely different from a collapse that may be seen in conventional structures. Apart from the failure of the connections, a massive block of the structure above the first storey was left free-standing because of the unzipping of the connections across the first storey. This resulted in the requirement to conduct further analysis to understand the response of this free-standing superstructure component under dynamic events. This chapter is aimed at developing experimental models of the prototype modular and conventional buildings studied in the previous chapter to conduct experimental analysis on them. The physical testings are aimed at observing the collapse response in real, and to validate finite element models of the modular and conventional structures which are discussed under Chapter 5.

4.1 Scaling and Dynamic Similitudes

Previous studies on shaking table experiments of scaled-down buildings presented substantial evidence that the choice of the scaling factor was mostly made by considering the experimental constraints such as shaking table capacity and laboratory limitations. In a study reported in Ref. [156], the authors employed a scaling factor of 30 to represent a 56-storey reinforced concrete building. It was reported that the scaling factor of 30 was chosen when considering the shaking table size and the height requirements of the laboratory. In another study reported in Ref. [157] a scaling factor of 50 was chosen to scale down a 101-storey building in a study. The choice of the scaling factor was justified in view of the capacity and size of the shaking table available. In yet another study reported in Ref. [158], a shaking table experiment was conducted on a 30-storey building. In this study, a scaling factor of 25 was used in view of the shaking table capacity as the primary constraint. Many other references such as Refs. [159, 160] put the focus on the response behaviour of the scaled-down models as observed on the shaking table. All these experimental studies were subject to constraints such as capacity of the actuator which drove the shaking table, size of the shaking table, and laboratory space constraints when deciding on the scaling factor. The considerations taken by researchers of studies reported in the literature, were used as the basis of the choice of the scaling factors to be adopted in the research project reported in the thesis.

4.2 Development of Experimental Models

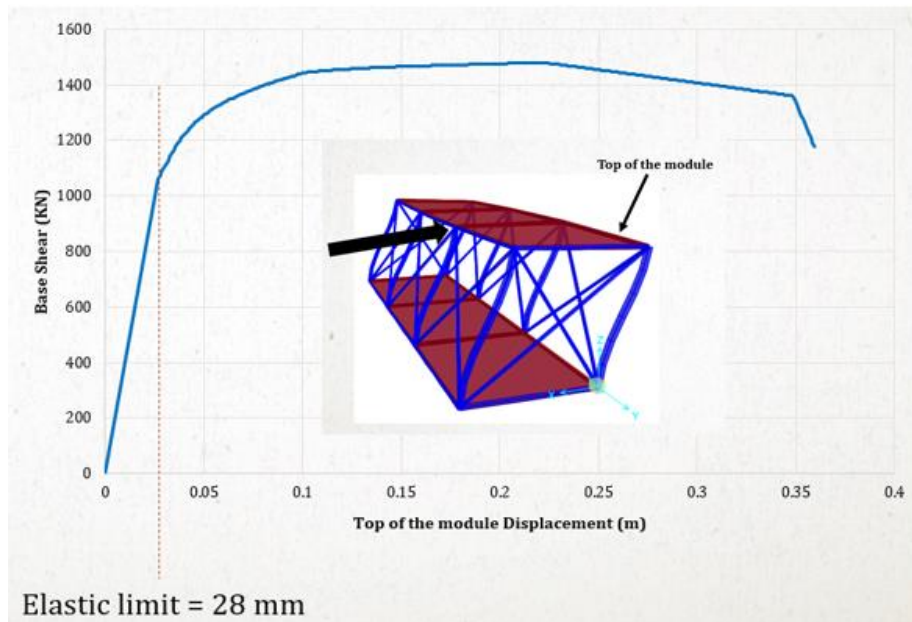
This section of the thesis describes the development of experimental models and justification for the choices made during the model development.

4.2.1 Comparison of Lateral Deformation Capabilities of a Connection and a Module of the Prototype Structure

Before experimental scaled-down models were built, a study was conducted to understand the relative contribution from the modules and the connections to deformation of the structure.

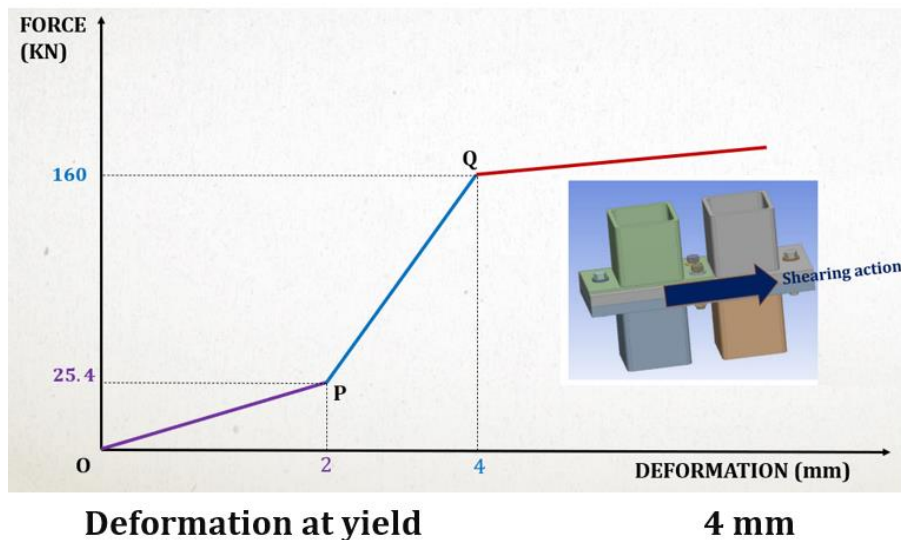
Figure 4-1 (a) & (b) illustrates the lateral force-deformation behaviour of a single module and an inter-modular connection, respectively. If the elastic range of deformation is considered, it can be seen that the module deformation is very high compared with that of a connection. This proves that most of the building deformation during the elastic range stems from deformation of the modules.

However, it was observed from the FE analysis results discussed in Chapter 3 that the connections govern failure and yielding in the structure. It was the yielding of the connections that initiated the yield in the building, and it was the failure of just a single connection that triggered the failure of subsequent connections in the same storey and then a complete failure of the building. This can be related to the relative strengths of connections and modules. Modules are more robust than connections. Moreover, modules are more able to distribute the stresses throughout their height because of their large height compared with that of a connection ($3,000/300 = 10$ times taller) and its large sections of columns, braces, and other components.



Deformation at yield 28mm

(a)



(b)

Figure 4-1 Lateral force–deformation plateau of (a) a single module and (b) a connection

4.2.2 Development of the Experimental Model of the Modular Building

One of the major challenges in developing a scaled-down model was finding an appropriate inter-modular connection. Figure 4-1(b) shows that deformation of the inter-modular

connection in the prototype building was 4 mm at the time of yielding. Thus, if direct scaling laws were applied to compute the elastic deformation required for the connection in the scaled-down experimental model, it would be $4/20 = 0.2$ mm. It would be near impossible to find a suitable specimen that meets this requirement, and its inclusion would have a significant effect on the accuracy of measurements because of the very small values involved.

Another critical aspect worth noting during the design of the experimental models is that there was not much contribution from the failure of the modules to the damage variation of the structure. The failure was mainly in the connections, although the deformation was mostly in the modules. This implies that the modules are stronger and more elastic than the connections, and the FE model must be capable of simulating these properties accurately.

In this study, both these challenges were overcome by employing rigid boxes to represent the modules in the experimental model. In this way, the modules were forced to be stronger throughout the analysis, representing the case observed in the FE model. However, rigid modules are less capable of deformation. This was overcome by using connections that could provide the deformation that the modules would have provided.

In this way, the challenge of requiring connections with a very low deformation capacity had they been directly scaled down was avoided. Since the connection was required to have a higher deformation capacity (to enable the deformation that would otherwise have been provided by the modules), the burden of finding a suitable connection for the experimental model was eased, and the problem that would have arisen regarding the accuracy of measurement from smaller deformations was avoided.

For these reasons, the deformations that would have occurred in the modules were included in the connections, and fully rigid modules were chosen, simplifying the model in terms of choosing a connection; the accuracy of measurement; and complications in developing the experimental models.

4.2.2.1 Modules Used in the Experimental Model

The modules of the experimental model were made of rigid aluminium boxes with the required mass added inside them. The boxes were 0.45 m long, 0.15 m wide and 0.15 m tall, considering a scaling factor λ of 20. Figure 4-2 illustrates the relationship between the dimensions of the scaled and prototype modules. The scaled modules were manufactured with protrusions at their

corners, as shown in Figure 4-3, to provide vertical and horizontal connectivity between modules.

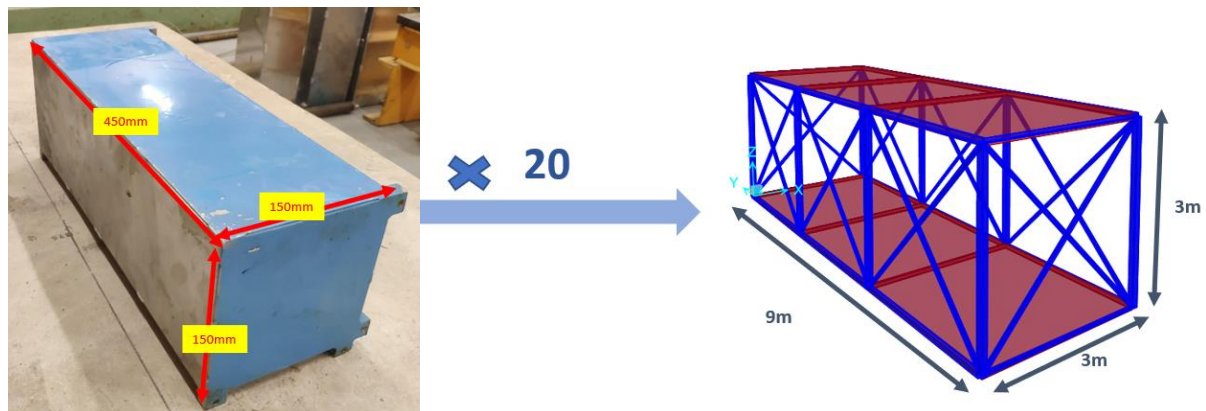


Figure 4-2 Relationship between the scaled and prototype modules



Figure 4-3 Protrusions at the module face providing connectivity between modules

4.2.2.2 Connection Used in the Experimental Model

The connections employed in the experimental model were made of 3D printed material (Vero yellow material) so that they could be easily manufactured with the desired size and length on the university premises. Moreover, 3D printing provided the flexibility to change the dimensions of connection specimens so that their strength and stiffness could be adjusted as required. A cylindrical specimen with the dimensions shown in Figure 4-4 was chosen as the connection.

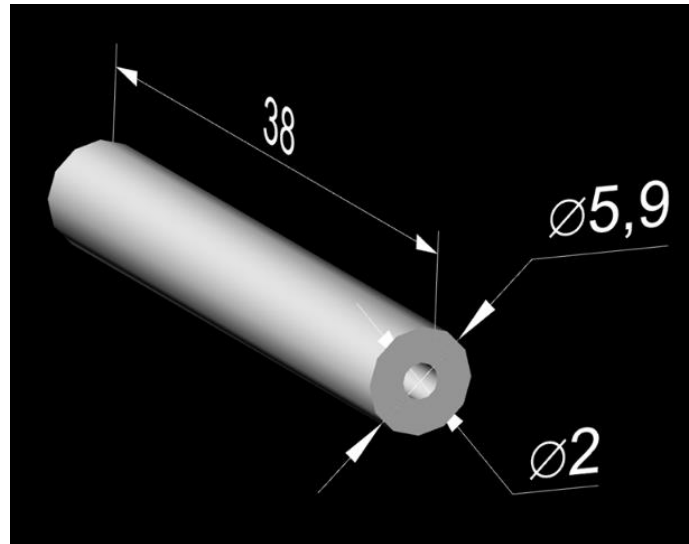


Figure 4-4 3D view of the chosen connection specimen

The specimen was tested in the MTS testing facility at the University of Melbourne to obtain its force-deformation plateau (see Figure 4-5). The internal diameter of the specimen was adjusted to fine-tune stiffness and strength to achieve the desired values. The force-deformation plateau obtained for the connection employed in this study is illustrated in Figure 4-6.

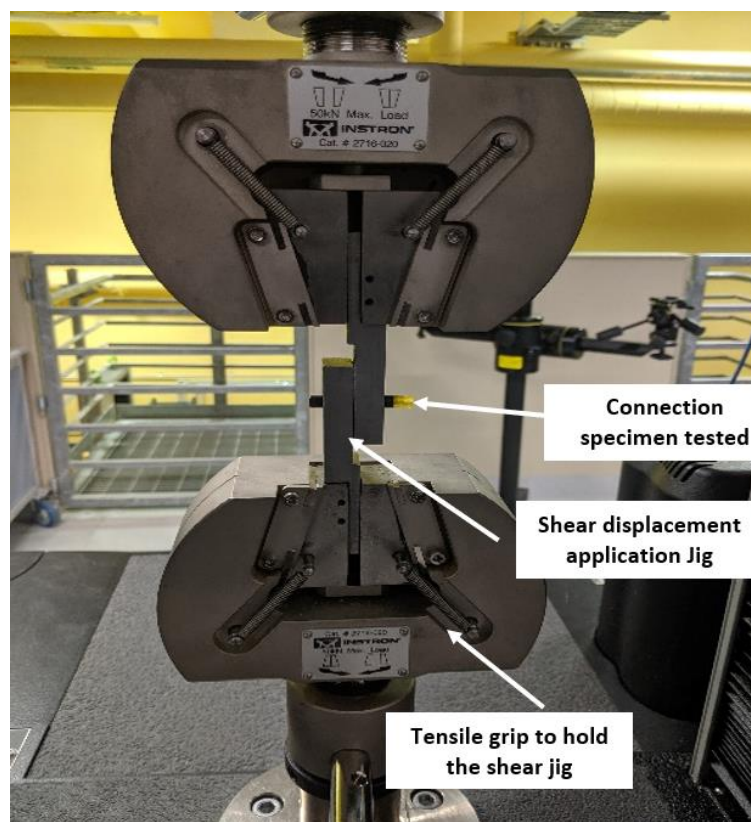


Figure 4-5 Experimental set-up to obtain the force-deformation plateau of the connection

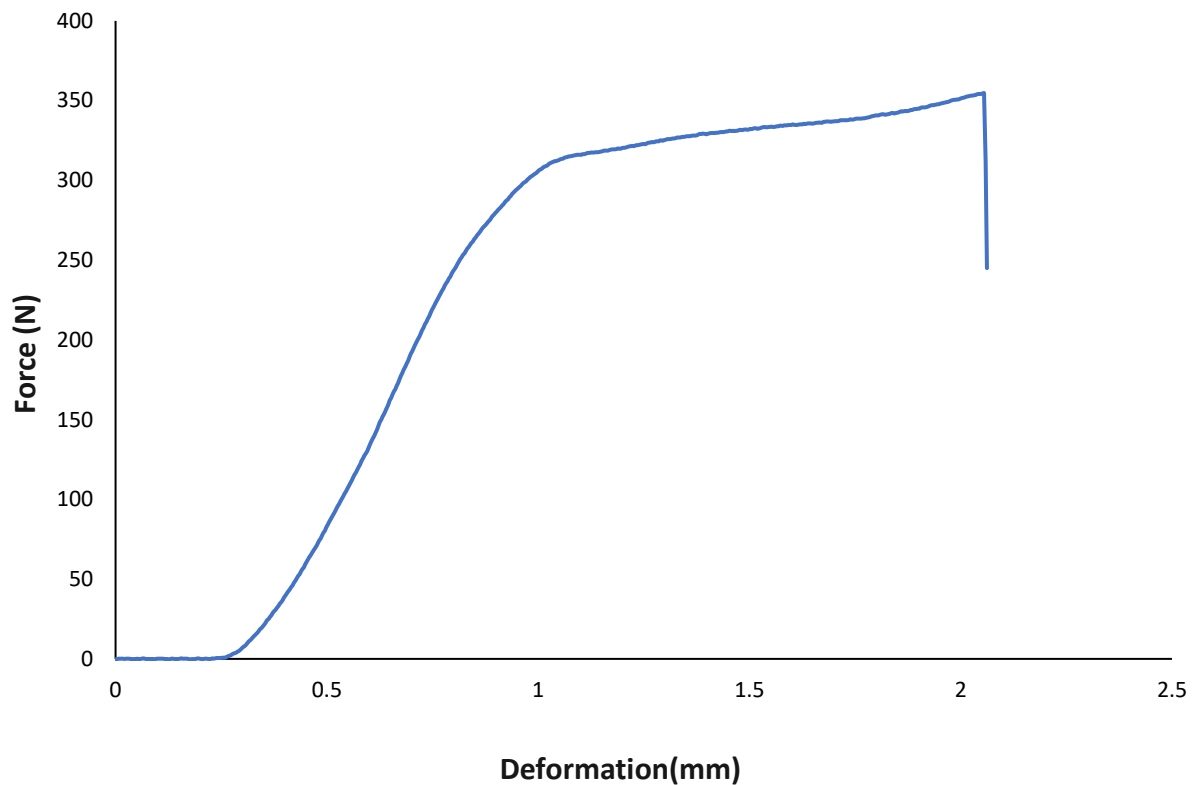


Figure 4-6 Force–deformation behaviour of the connection used

4.2.2.3 The Final Experimental Model of the Modular Building

Once the connections and modules were developed, they were assembled to form the final building, as shown in Figure 4-7. The building constructed was 1635 mm tall, 315 mm wide and 450 mm long, employing a scaling factor of 1/20 relative to the prototype model. The base modules were rigidly fixed to a wooden board on which the building was placed and fixed to the shaking table or ground depending on the type of test conducted (dynamic or pushover analysis).

Although the prototype and scale models are not all that similar (other than their primary dimensions), the assumptions made to develop the scaled model to simplify the model development process are justified in section 5.5. This is done by comparing the results from the prototype and scaled-down model analyses.



Figure 4-7 Experimental model of the modular building

4.2.3 Free-standing stack of modules as a control model

Aligning with the hypothesis formed under the research problem (section 1.2), the observations on the prototype modular structure under the intensifying dynamic loadings (section 3.2.4.2.1) revealed that the first storey edge connections of the modular building failed in an unzipping manner to form a free-standing stack of modules which experienced the rest of the ground motion applied.

Results presented in section 4.3.3.2 based on the observations of the response of the modular building under the applied ETEF shows that the superstructure component formed, experienced rocking, sliding, and overturned. Hence, as also presented under section 3.1.3, a control model which consisted of the nine storeys above the ground storey models (exactly representing the free-standing superstructure component formed from the failure of first storey connections), as shown in Figure 4-8 (b) below was subjected to the ground motion stretch corresponding to the rock, slide and overturn phase.

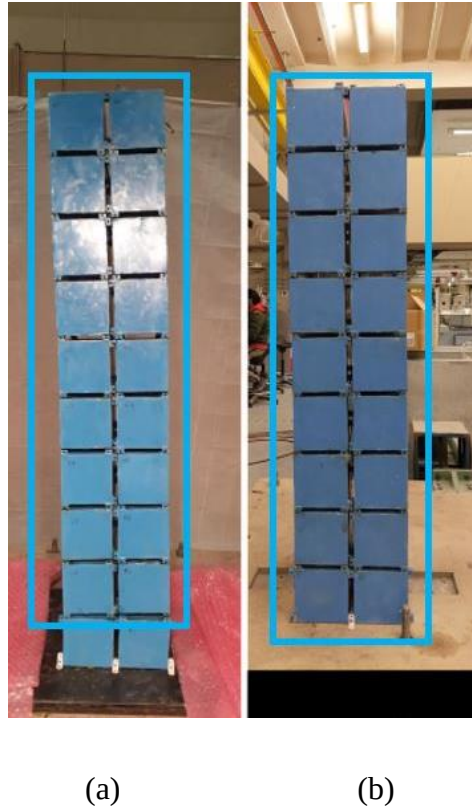


Figure 4-8 (a) Modular building representing the superstructure component formed (b) Free-standing superstructure component placed on the shaking table

4.2.4 Development of the Experimental Model of the Conventional Steel Building

A scaled-down experimental model of the conventional steel building was also constructed with the same primary dimensions as the modular building, which also matched the scaled dimensions of the prototype conventional building. The structure was entirely fabricated using aluminium frame elements.

Since the structure considered was only 10 storeys, the section sizes were kept uniform for every storey. Columns were made of rectangular cross-sectioned aluminium frame elements that were 25 mm deep and 3 mm wide. Beams were made of aluminium ‘L’ sections of 25-mm outside vertical leg and outside horizontal leg lengths, and 1.4-mm thickness. Finally, the braces were made with 2×2 -mm square cross-sectioned aluminium frame elements. The frame elements were connected firmly using bracket elements and screws.

Figure 4-9 illustrates the experimental model of the conventional steel frame used in this study. The frame was fixed to a wooden board that could be moved to be fixed onto the shaking table

or ground depending on the type of analysis conducted on the structure (dynamic or pushover analysis).

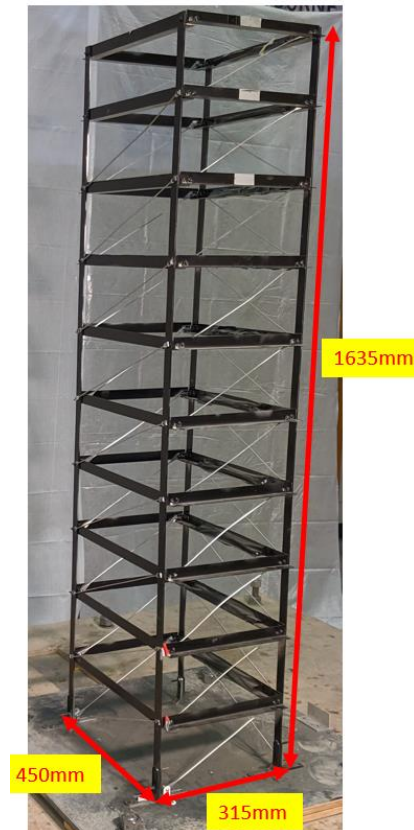


Figure 4-9 Experimental model of the conventional steel frame

4.3 Experimental Analysis of the Scaled-down Model of the Modular Building

4.3.1 Modal Analysis Results

The first test carried out on the experimental model was a pluck test to record the free vibration response of the structure and identify the fundamental periods of the structure. The first three fundamental periods of the structure thus identified were 0.19 s, 0.13 s and 0.09 s, respectively.

The natural frequency of the experimental model and the prototype model are related by a factor of $\sqrt{\lambda}$, where $\lambda=20$. If this factor scale up the first mode period value of the experimental model, it will result in a period vale of 0.85s, which agrees very well with the first mode period value obtained for the prototype structure under section 3.2.1 (Figure 3-15). These results which agree very well with each other can be considered as a good promising start-up of a reliable scaling of the prototype model.

4.3.2 Pushover Analysis

A pushover analysis was conducted prior to dynamic analysis, to obtain a preliminary idea of how the structure would respond under the dynamic loadings. Moreover, the pushover analysis was intended to provide a statistical validation of the structure response with the numerical results presented in Chapter 5.

The structure was initially fixed to a thicker wooden board, which itself was firmly fixed to the ground. Uniform displacement increments were applied using a caulk gun with its tip connected to the roof of the building with a load cell in the middle of the connecting string, as shown in Figure 4-10(a). The load cell was used to measure the applied loading. A laser measuring device was connected at the other side of the structure (opposite to where the caulk gun was placed) targeting the roof of the structure to measure roof displacement (Figure 4-10 (b)).

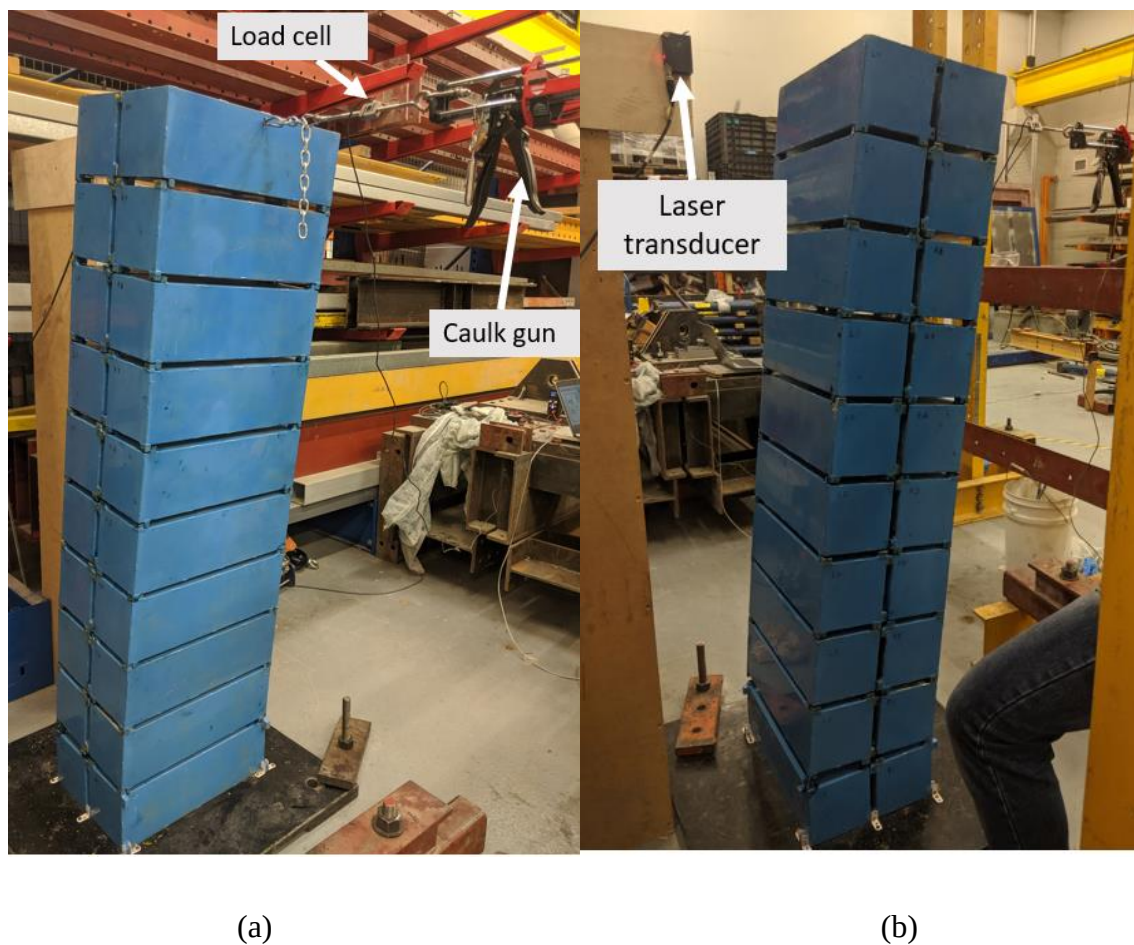


Figure 4-10 Pushover testing experimental set-up (a) displacement application and load measuring set up (b) displacement measuring set up.

With gradual increments in roof displacement, the structure began to deform with more stresses concentrated at the first-storey connections. Since the structure was rigidly fixed to the wooden base, there was no possibility of the structure deforming about the foundation. Hence, the structure was observed to be bending more about the first storey connections. In contrast, all other storeys displayed minimal flexing about the vertical line running through the centroid of the structure (as shown in Figure 4-11). With further increases in roof displacement, the first-storey edge connections in the windward side started failing, and the structure opened up along the windward edge of the first storey, as illustrated in Figure 4-12. Once displacement had increased beyond this point, failure progressed to the interior connections of the same storey and the levered edge connections, opening up the structure in an unzipping manner across that storey.

The pushover curve resulting from the above experiment is depicted in Figure 4-13, which indicates a brittle behaviour, similar to that observed in Figure 3-17 for the prototype modular building. The first deviation from linearity indicates the first failure of the connection, moments after which the whole structure overturned about the first storey due to the failure of all connections in that storey.



Figure 4-11 Structure deformation under incremental displacement

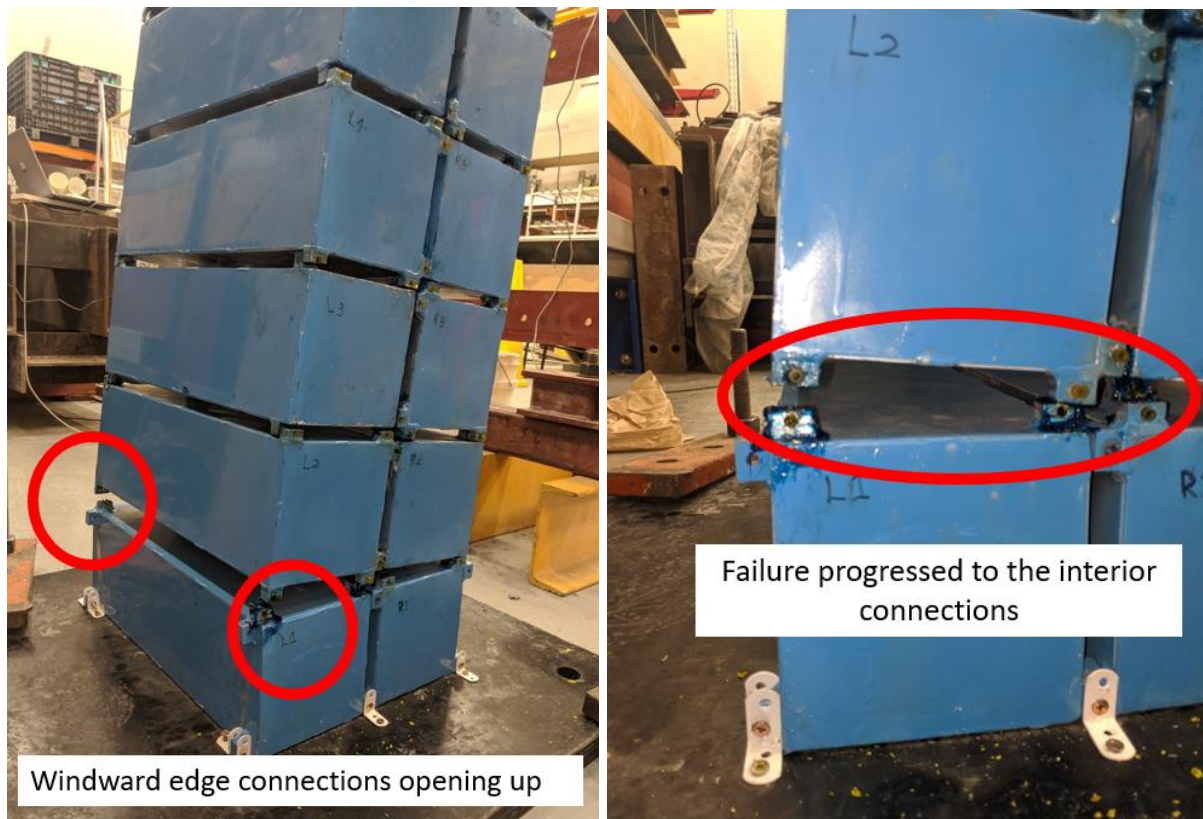


Figure 4-12 Failure progression during the pushover analysis

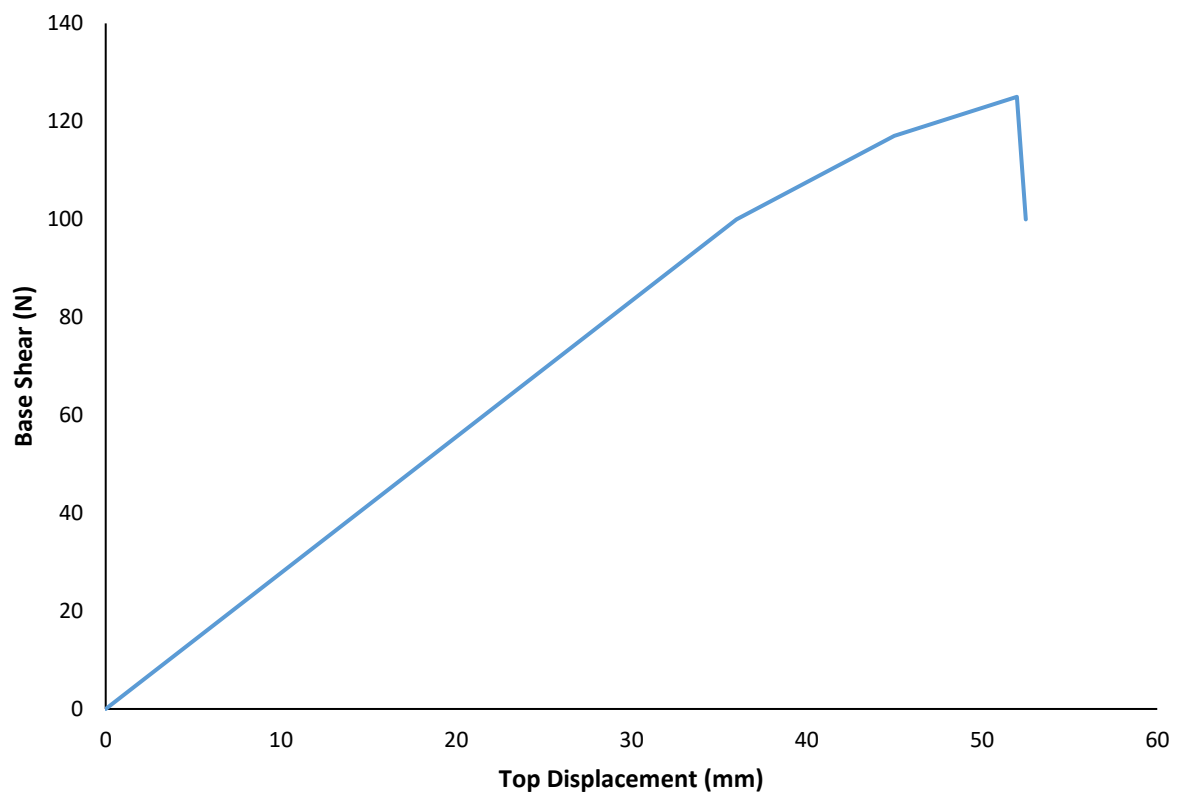


Figure 4-13 Pushover curve for the experimental modular building

4.3.3 Dynamic Time History Analysis Under the ETEF

The structure was subjected to a dynamic time history using the ETHA, on the shaking table (Figure 4-14). Laser transducers were employed to monitor roof displacement. Shaking table displacement was also monitored using laser transducers to cross-check whether the applied ground motion had been correctly received by the shaking table.



Figure 4-14 Experimental set-up of the modular building on the shaking table

4.3.3.1 Ground Motion Applied

The ETEF used on the shaking table was a scaled version of the ETEF described in Section 3.2.4.1. The time axis of the excitation was scaled down by a factor of $\sqrt{\lambda}$ as per the Cauchy similitude conditions specified in Table 2-6. The resulting ETEF is illustrated in Figure 4-15.

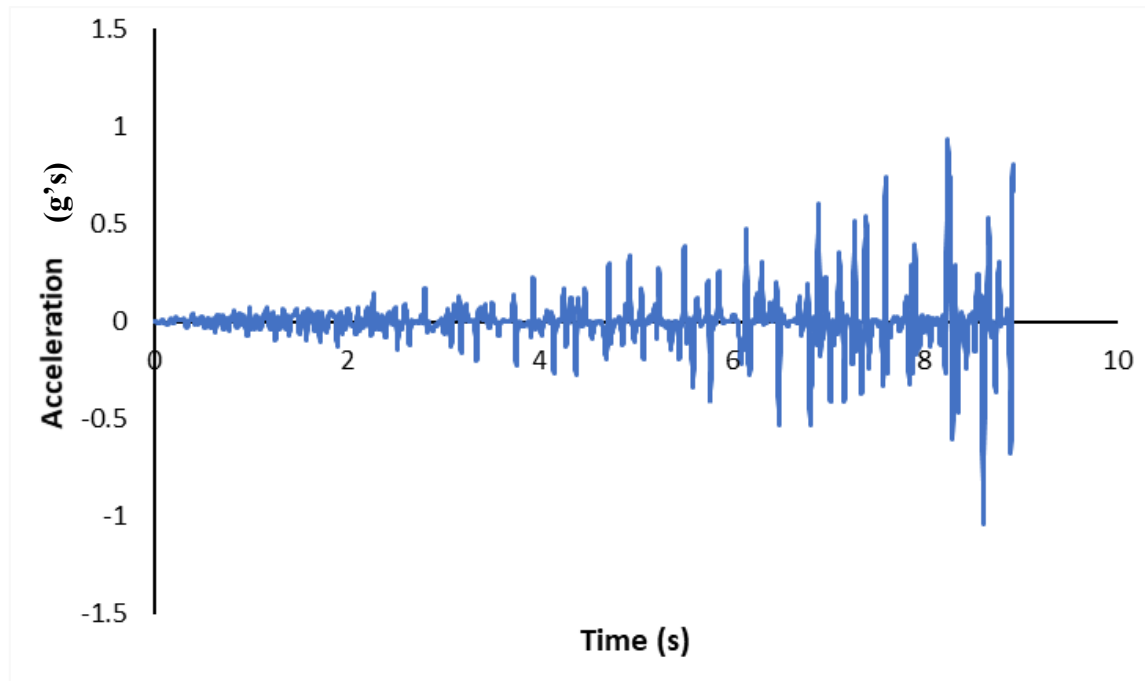


Figure 4-15 ETEF applied to the shaking table model

4.3.3.2 Shaking Table Experimental Results for the Modular Building

The endurance time history was applied to the structure in the form of a displacement time history on the shaking table. It was observed during the experiment that the structure, starting from rest while experiencing the ETEF, responded in a very safe manner during the first half of the ETEF.

However, with the gradual increase in applied acceleration, the structure began showing signs of excessive flexing around the first-storey edge connections. This was also an important observation in the pushover analysis. Gradually, when the acceleration time history further intensified, initial failure was observed along the connections in one edge of the first storey. In a matter of no time at all, the connections in the opposite edge also failed. This placed the interior connections in a critical situation by attracting higher stresses (as they are the only connections available in the storey) and hence caused the failure of those connections.

Once all the connections of the first storey had failed, a slip of the superstructure (stack of modules above the first storey) was observed, followed by overturning and collapse. This slide, rock and overturning observed was something not captured in the observations during the analysis of the FE model of prototype modular building, as discussed in Chapter 3. Even though the FE model indicated that all the connections failed, and a superstructure component separated from the modular building, it could not capture what really happened just after the

failure of all the connections. Observing and identifying this occurrence was one of the aims of expanding the study to conduct experimental analysis. Results of experimental analysis of the free-standing stack of modules (methodology explained in section 4.2.3) is discussed in section 4.4.

Figure 4-16 illustrates the recorded variation in roof displacement with time. It can be observed that the roof displacement increased disproportionately when the connections began failing. Some of the snaps from the shaking table experiment at different stages of modular building behaviour are shown in Figure 4-17. A video of the behaviour of the structure during the shaking table experiment can be viewed at <https://youtu.be/5SFnB0UdYpw>.

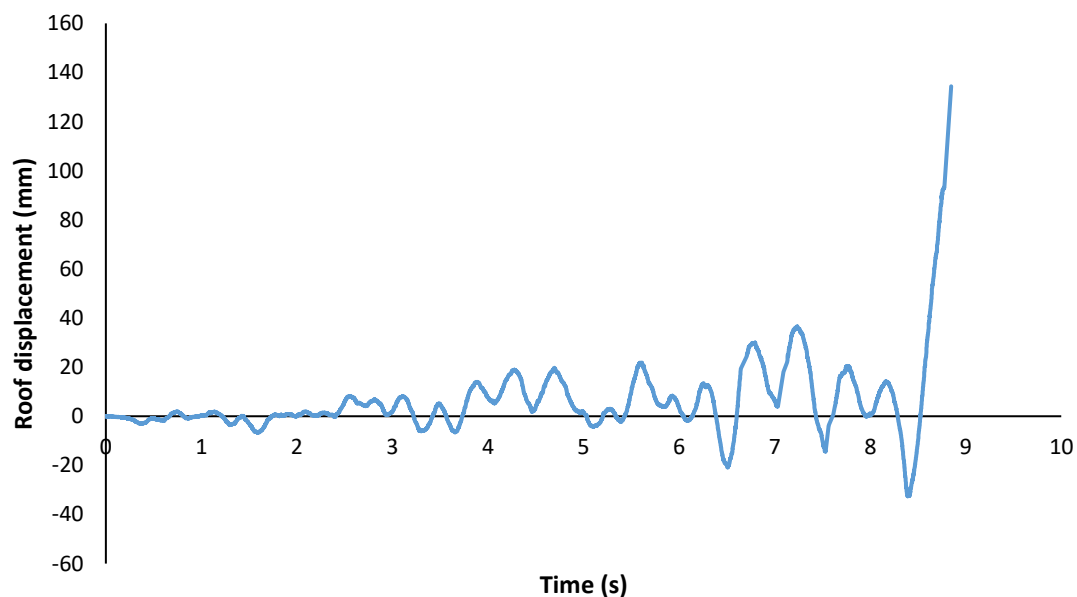


Figure 4-16 Roof displacement time history recorded under the shaking table experiment of the modular building

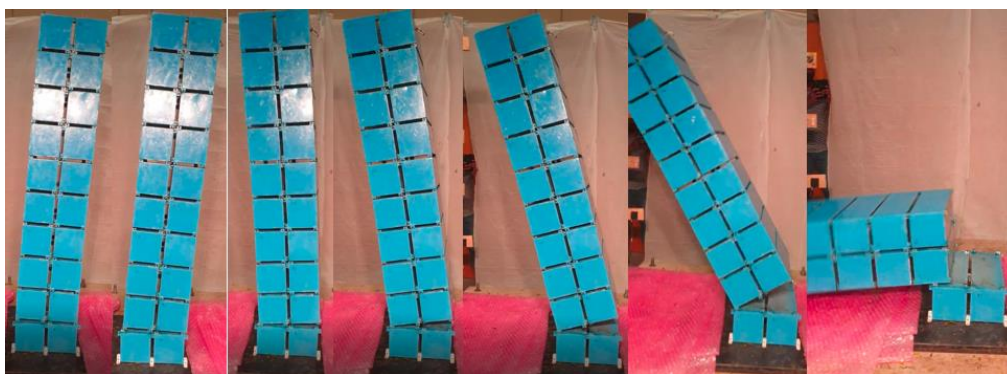


Figure 4-17 Behaviour of the modular building under the ETEF, images from the experiment

The roof displacement time history shown in Figure 4-16 was converted to its corresponding IDA curve using the approach followed in Section 3.2.4.2.2, as shown in Figure 3-23 and Figure 3-24. The IDA curve obtained is presented in Figure 4-18.

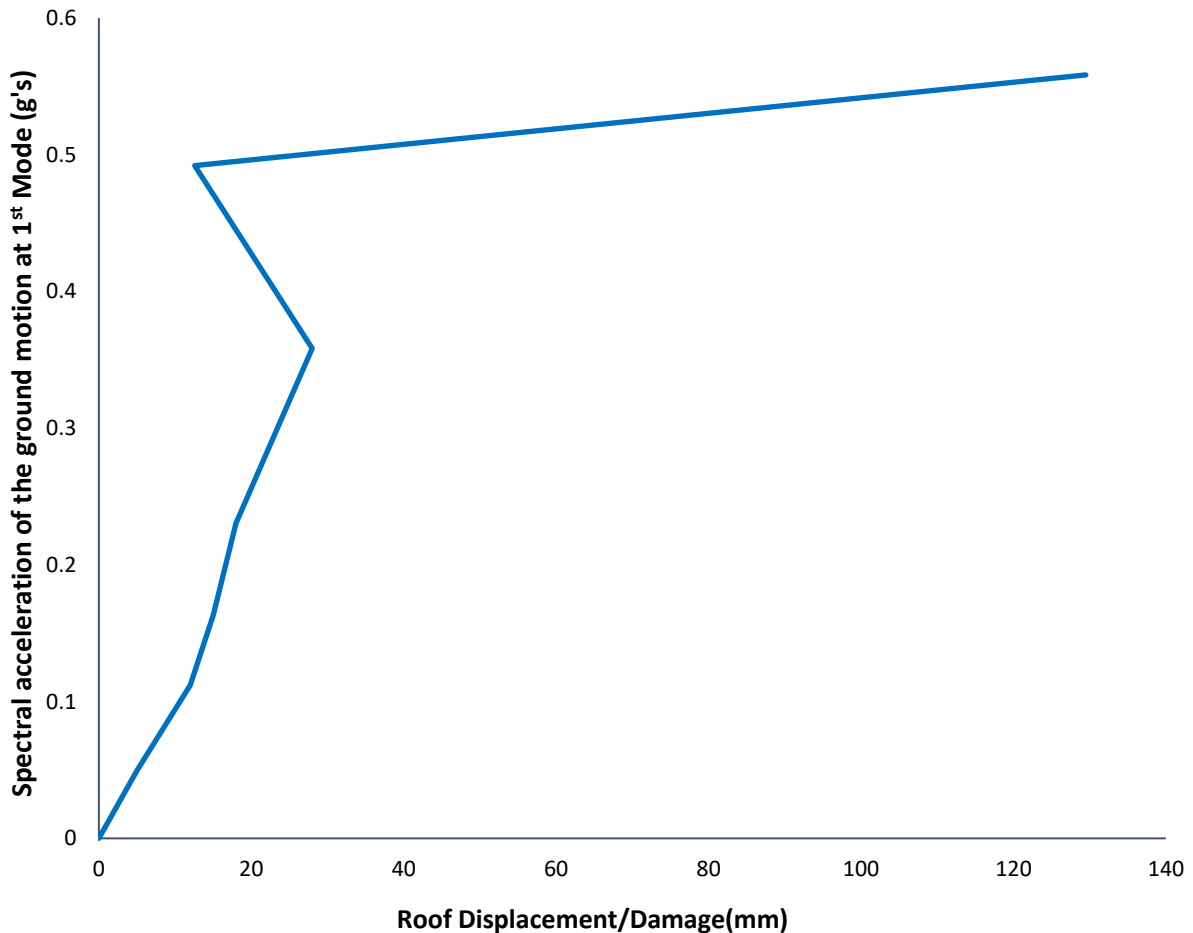


Figure 4-18 IDA curve corresponding to the roof displacement time history of the modular building shown in Figure 4-16

IDA can be utilised to interpret the results observed in a more meaningful manner. It can be observed from the IDA plot (Figure 4-18) that the structure performs in a safe manner up to spectral accelerations (seismic intensities) of around 0.4 g. However, around a spectral acceleration of 0.5g, there is a sudden increase in damage (roof displacement). This sudden increase in damage occurred as a consequence of the failure of connections.

Expressing the results in terms of seismic intensities such as spectral acceleration or PGA values might not convey a message easily understandable by all. Hence, the dependent variable (y-axis, seismic intensity) axis was converted to return period. Expressing results in terms of return period values may more starkly indicate the risk involved.

Figure 4-19 illustrates the IDA curve together with the return periods computed for Melbourne ground conditions based on the spectral acceleration values. This plot can be used to highlight that the structure behaved within the acceptable specifications under a ground motion of a 500-year hazard return period. In contrast, the damage in the structure increased disproportionately and catastrophically overturned the structure under a seismic event with a 2,500-year hazard return period.

General design conditions for structures are for a seismic event with a 500-year hazard return period. However, there is still a chance that a structure may experience a seismic event beyond this design limit.

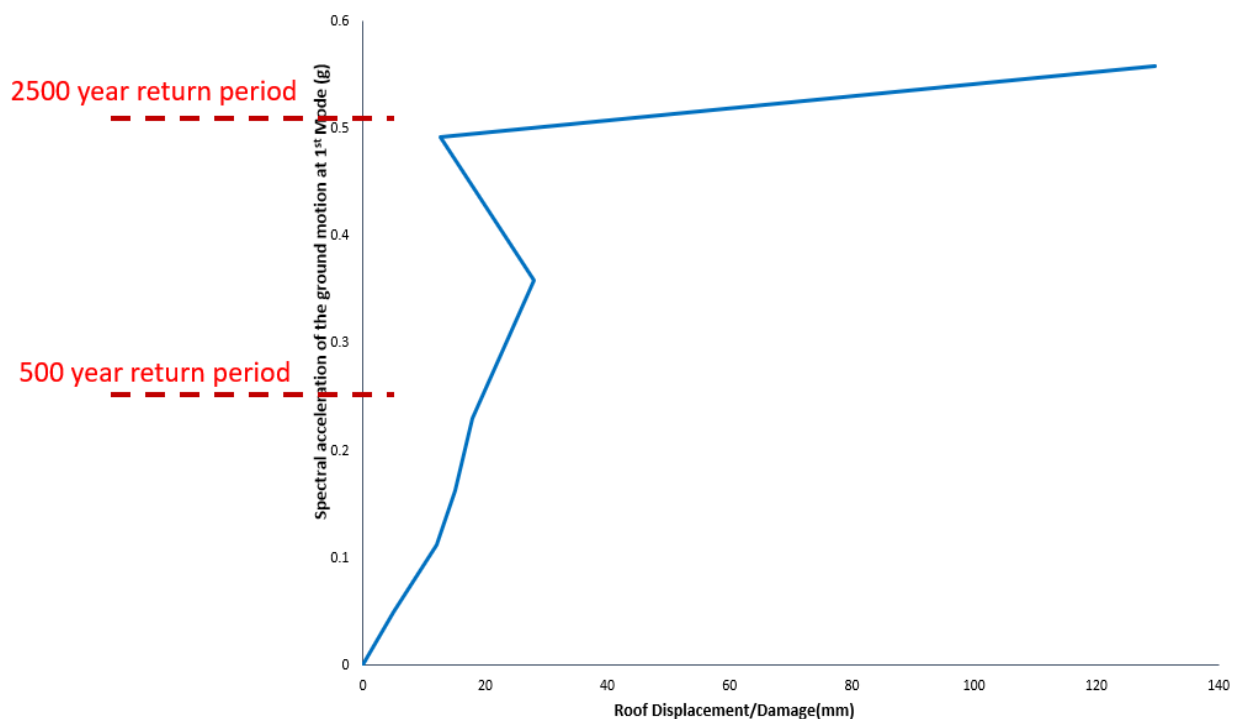


Figure 4-19 IDA curve for the modular building together with hazard return periods of earthquake

4.3.3.3 Maximum Elastic Drift During the Shaking Table Experiment

It is of paramount interest for researchers to have an idea of the maximum elastic capacity of a structure to make judgements and conclusions about the structure performance. Hence, the elastic component of the roof displacement time history was extracted in this study to estimate the elastic drift capacity of the structure.

Connection failure was identified using the sound of breaking of connections. In addition, major deviations in the roof displacement time history were identified to detect the first yield/failure in the structure. Finally, the FE results, which are presented in section 5.2.3, were employed to verify the time of occurrence of significant changes in the modular building behaviour until collapse.

Based on the shaking table experiment results, the first yield in the structure occurred between the fifth and sixth second of the roof displacement time history. The corresponding roof displacement time history, distinguished by the time at which the first yield occurred, is presented in Figure 4-20.

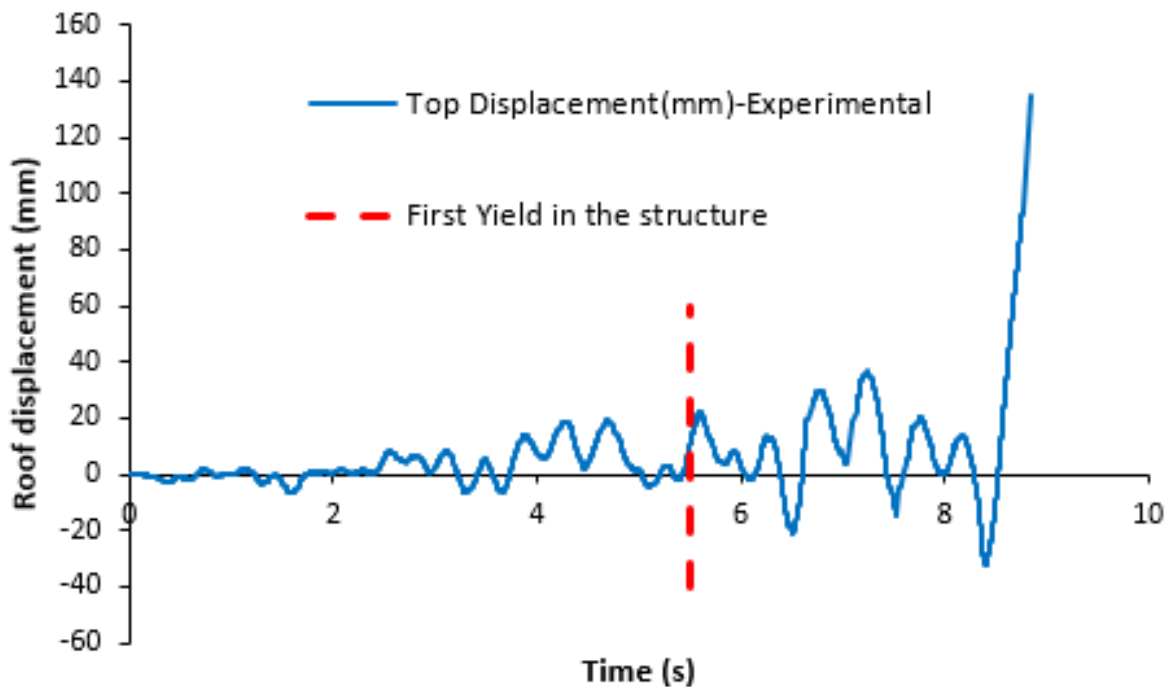


Figure 4-20 Roof displacement time history with a defined elastic range

Considering the elastic range of the roof displacement time history, the maximum elastic deformation observed falls in the range 15-20 mm, which converts to a drift range of 1-1.2%, as is generally the case for most structures available in practice. Moreover, the maximum elastic drift obtained using the experimental analysis agrees very well with that calculated for the prototype modular building in Section 3.2.4.2.2 and 3.2.4.2.3.

4.4 Experimental Analysis Results for the Free-standing Stack of Modules

The superstructure component which underwent rocking, sliding and overturning under the application of the ETEF on the modular building (section 4.3.3) was set free-standing and subjected to the same stretch of ground motion as explained in section 4.2.3. It was observed during this ground motion that the free-standing stack of modules did not overturn. The structure was observed to rock purely under the ground motion. Once the ground motion ceased, the structure oscillated freely until its motion was dampened, and it came to rest. A video of the above observation, together with a comparison of the overturning of the modular building superstructure, can be viewed at: https://youtu.be/uc-XLpri1_0 and <https://youtu.be/ywu14bbm-uw>.

The roof displacement measured is employed to compute the angle of rotation of the free-standing body, assuming the stack of modules behaves as a rigid body. The angle of rotation time history thus obtained is shown in Figure 4-21. The variation in the angle of rotation confirms that the rigid body did not reach the critical angle for overturning, and hence did not overturn (calculation of the critical angle for overturning, assuming the free-standing superstructure as a rigid body is presented under section 6.3).

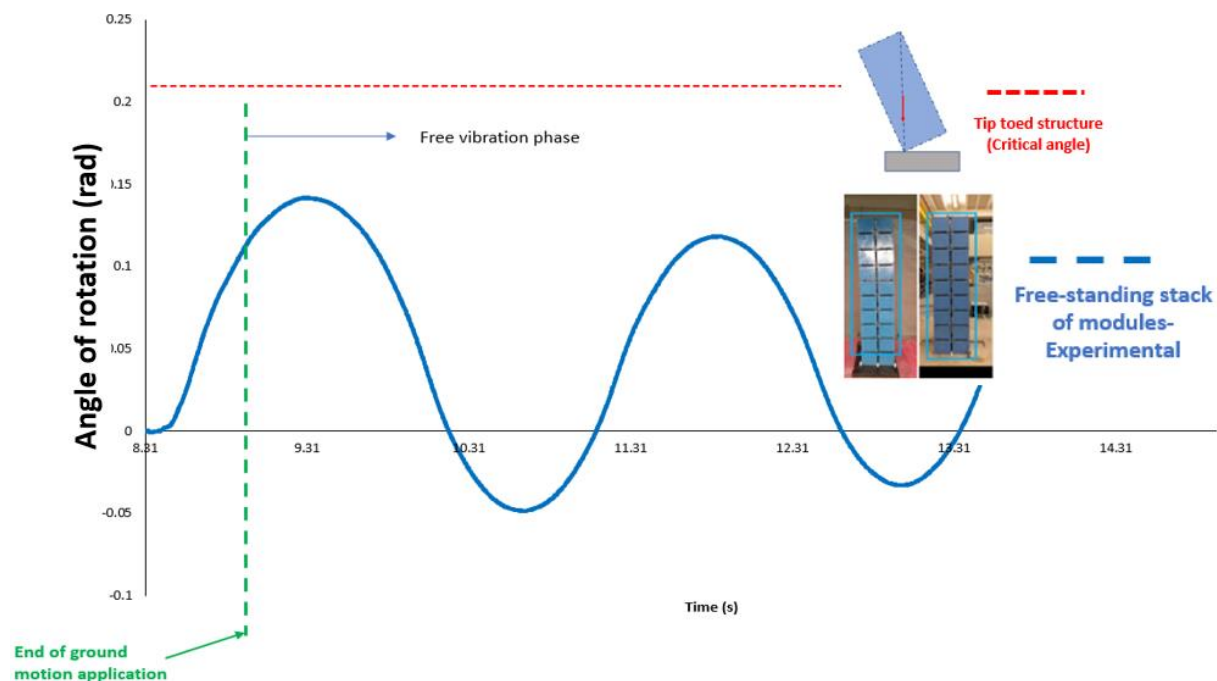


Figure 4-21 Experimental results for variation in the angle of rotation of the free-standing rigid block

The results obtained under this section in comparison with the results for the overturning of the modular building superstructure obtained under section 4.3.3.2 indicates that, under the same considered ground motions stretch, the modular building superstructure which was formed as a result of the failure of connections, overturned, while the exact same superstructure component which was set to start from rest, free-standing on the shaking table, did not overturn. These observations are important in understanding the cause of overturning that was observed in the modular building. It is evident at this stage that the modular building superstructure component which overturned, started the free-standing phase with some initial conditions gained from the failure of the connections while the control model started from rest. Hence, the initial conditions that the modular building superstructure gained from the failure of the connections are the cause of overturning. Further discussions on this together with numerical models of the free-standing body are presented under section 6.4.1.

4.5 Experimental Analysis Results for the Scaled-down Model of the Conventional Steel Building

4.5.1 Modal Analysis Results

A pluck test was carried out on the experimental model of the conventional steel frame to extract its primary modal period values using the free vibration response. The experimental results showed that the first, second and third mode period values were 0.20 s, 0.16 s and 0.12 s, respectively. The natural frequency of the experimental model and the prototype model are related by a factor of $\sqrt{\lambda}$, where $\lambda = 20$. If this factor scales up the first mode period value of the experimental model, it will result in a period value of 0.89 s, which agrees very well with the first mode period value obtained in Section 3.3.1 for the prototype structure (Figure 3-30).

4.5.2 Pushover Analysis

Pushover analysis mainly served as validation of the numerical models developed and to obtain a preliminary idea of the structural performance under nonlinear dynamic loading. Hence, a full pushover analysis covering the destruction of the whole structure was not intended. However, a pushover analysis was conducted to capture the elastic phase of the structure deformation and the inelastic phase covering the failure of braces and some yielding in other components of the structure. Pushover analysis of the FE model of the experimental model covering the full range until collapse is presented in Section 5.3.2.

The test arrangement was very similar to that used for conducting pushover analysis on the experimental model of the modular building, as discussed in Section 4.3.2. The structure was subjected to displacement increments at the roof level. In the meantime, the applied force was measured using a load cell, and the roof displacement was monitored using a laser transducer (Figure 4-22).

With the gradual increase in roof displacement, it was observed that the structure began experiencing failure from the braces in the ground storey. The failure of these braces progressed slowly upwards along the structure, resulting in the loss of many braces. Once the yielding of the braces was complete, the structure experienced yielding of its ground storey columns, which underwent permanent deformation.

The pushover curve corresponding to the conventional steel frame is shown in Figure 4-23. It can be observed that the pushover curve varies very smoothly compared with that obtained for the scaled-down model of the modular building. The gradual failure of braces and other components of the structure, such as columns, did not result in a sudden progression of damage as was observed in the modular building. This structure had a more distributed mode of dissipating failure rather than failure being constrained to one specific location of the structure.

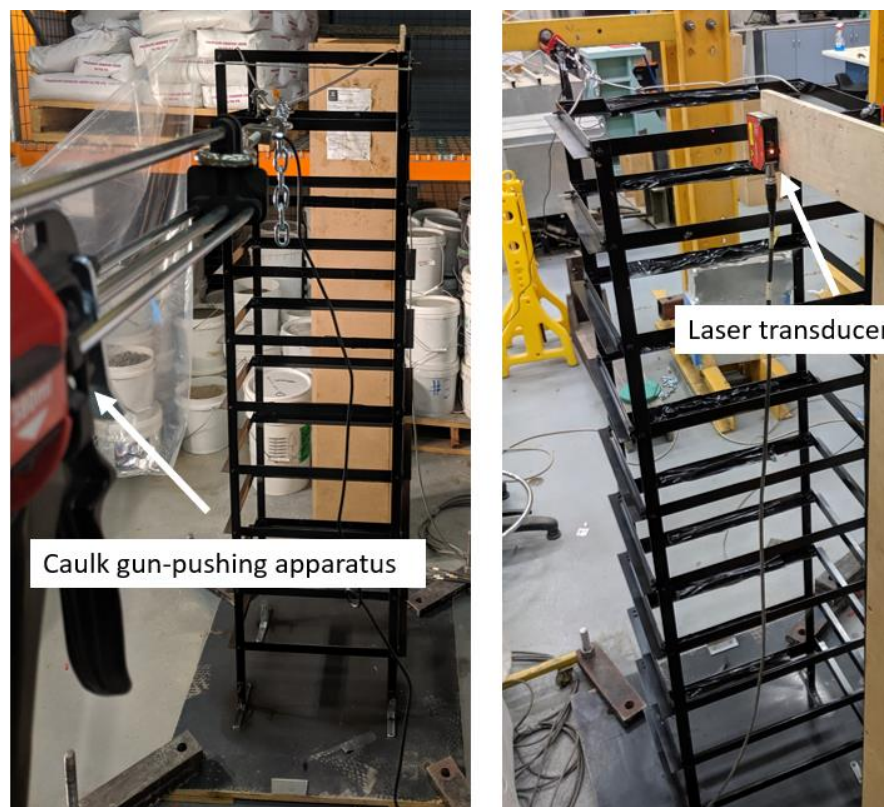


Figure 4-22 Experimental set-up for pushover analysis of the conventional steel building

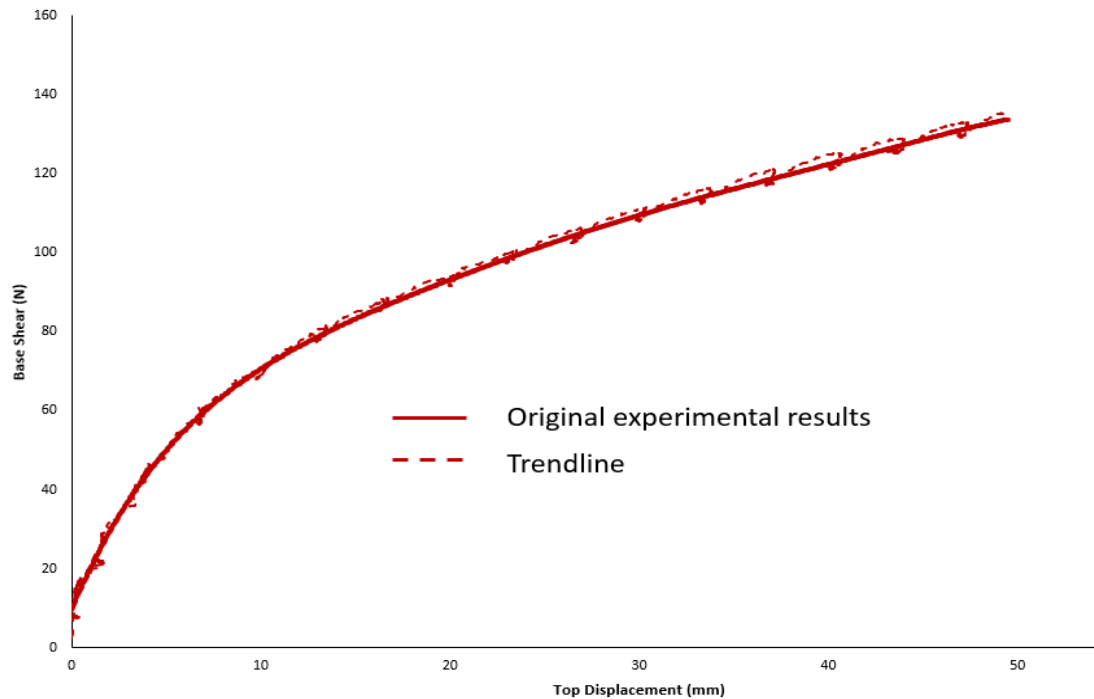


Figure 4-23 Pushover curve for the conventional steel building

4.5.3 Dynamic Time History Analysis Under Endurance Time Excitation Function

The endurance time history function explained in Section 4.3.3.1 was applied to the scaled-down model of the conventional steel frame. The procedure followed in studying the response of the experimental model of the modular building as described in Section 4.3.3 was employed in this study. The experimental set-up on the shaking table is shown in Figure 4-24.



Figure 4-24 Experimental set-up of conventional steel building on the shaking table

The ETEF was applied through the shaking table in the form of a displacement time history. In contrast to the modular building response, no catastrophic failures were observed. It was observed that with time when the intensity was increased, the primary LFRS in the structure (i.e. the braces) began failing. At the end of the application of the ETEF, there was a slight permanent deformation of the structure due to yielding of bottom storey columns.

The roof displacement recorded as the damage parameter is depicted in Figure 4-25. The time history obtained was converted to its corresponding IDA curve following the approach outlined in Section 3.2.4.2.2. The resulting IDA curve is presented in Figure 4-26, showing that variation in the damage with seismic intensity is more of a progressive, distributed variation, unlike what was observed for the modular building under the same loading (Figure 4-18).

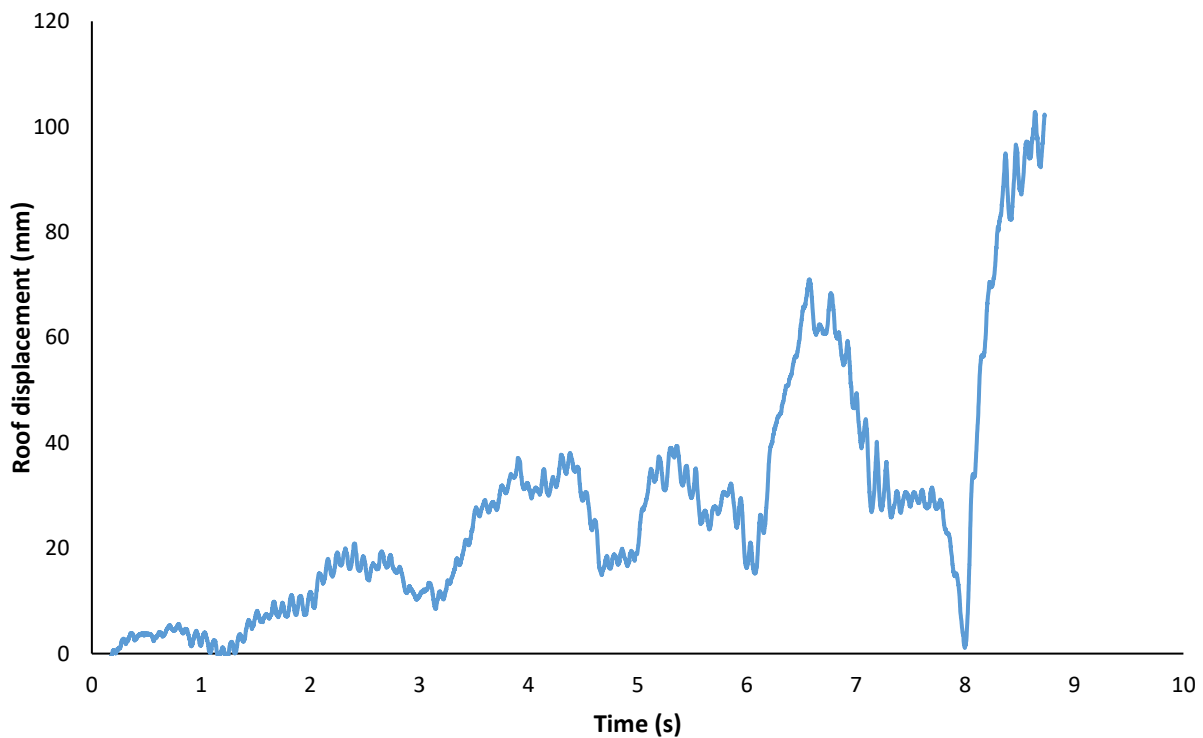


Figure 4-25 Roof displacement time history of the conventional steel building under the ETEF

It was also possible to convert the IDA curve shown in Figure 4-26 to a plot involving a more meaningful seismic intensity parameter like the hazard return period of a probable earthquake. Such a plot was also developed for the experimental model of the modular building (Figure 4-19). The IDA curve corresponding to the conventional steel frame is shown in Figure 4-27. (together with hazard return period)

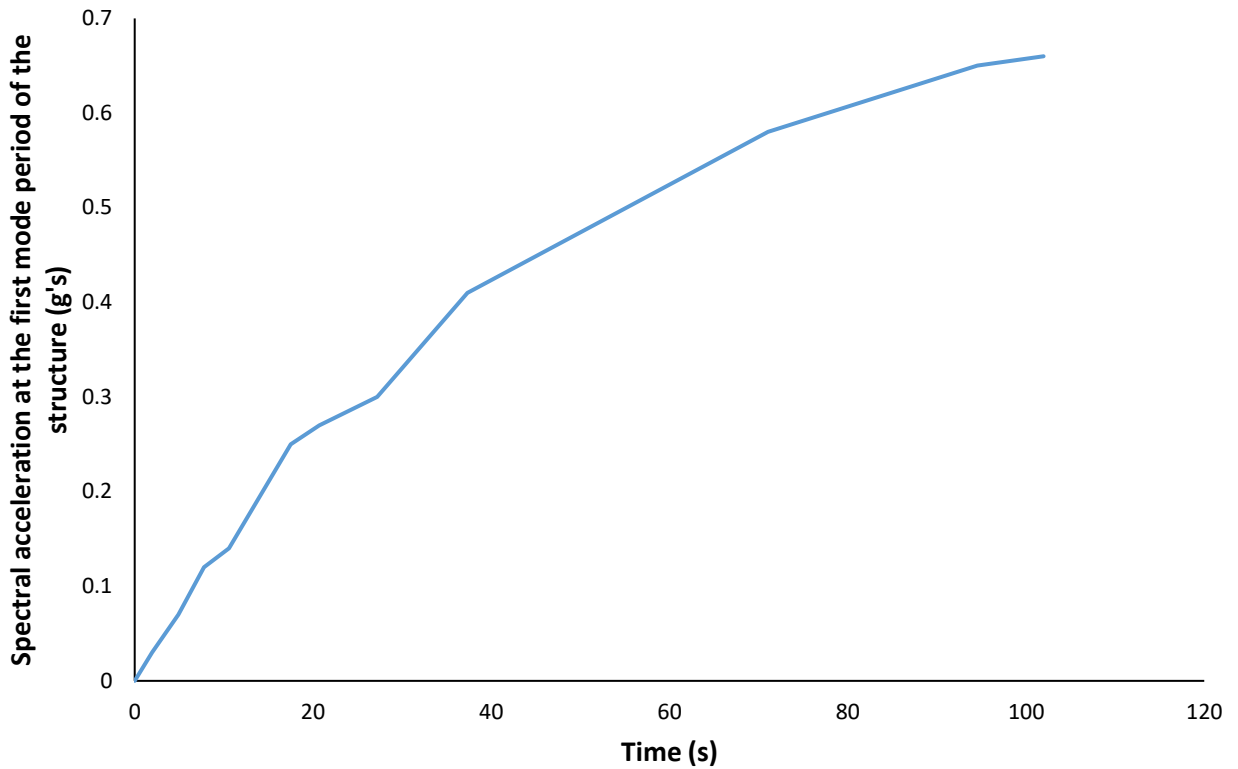


Figure 4-26 IDA curve corresponding to the roof displacement time history presented in Figure 4-25

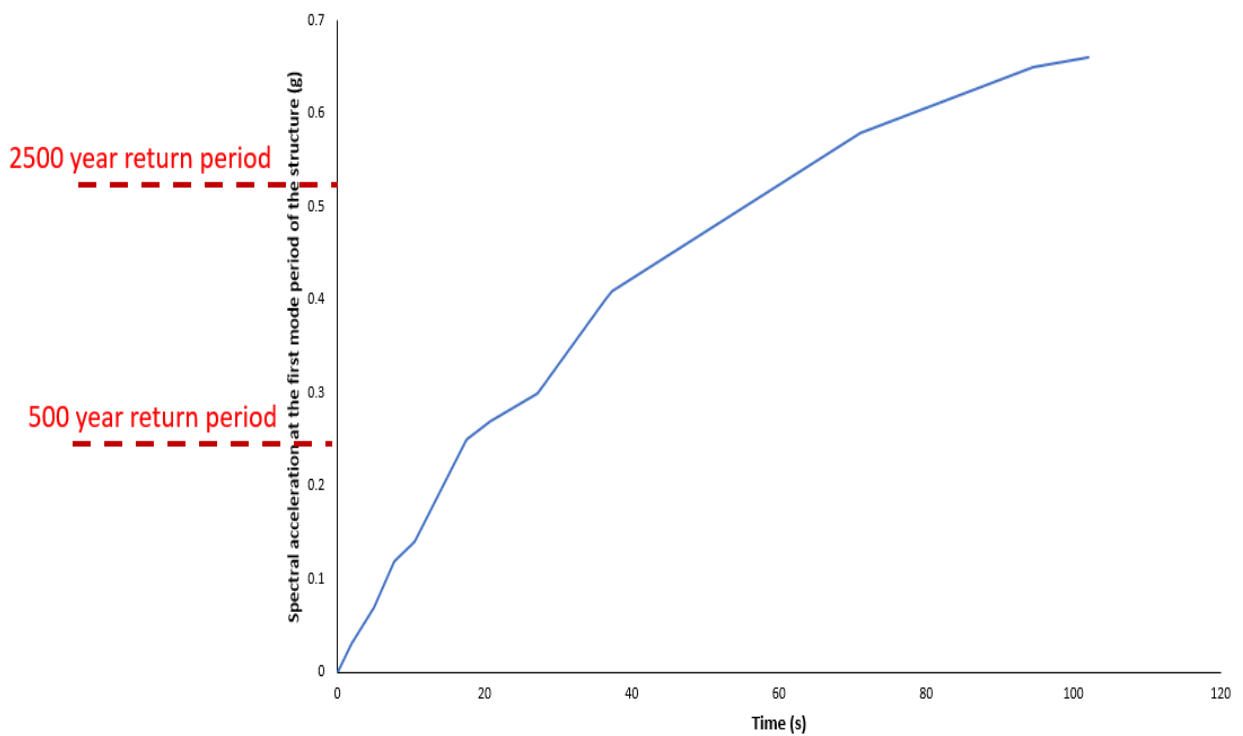


Figure 4-27 IDA curve for the conventional steel frame together with the return periods of seismic events

Figure 4-27 shows that the structure damage progression occurs at almost the same rates for earthquakes with hazard return periods of 500-years and 2500-years. Moreover, the damage progression rate is very moderate, as can be observed from the plot itself. There is no sudden spiking in the damage progression rate despite an increase in seismic intensity. This is entirely the opposite of what was observed in Figure 4-19 in the case of the modular building, where the damage progression increased abruptly when the seismic intensity increased to almost double the design limit.

These observations arise from the nature of the construction of these two types of building. The damage progression was more concentrated to a particular storey in the modular building, leading to a structural unzipping of that storey and overturn of the structure about that storey.

However, in the case of the conventional steel building, the damage was more to its primary lateral load resisting elements, which were also more distributed throughout the structure height. Hence, the increase in seismic intensity did not lead to just one specific area being critical in the structure. The damage spread throughout the structure, reducing the global risk of significant damage occurring in any primary element of the structure. Despite the presence of lateral force-resisting elements such as braces in the modular structure as well, the lateral loadings imparted on the bracings were finally transferred to the connections between the modules. This resulted in the connections being the most vulnerable point in the structure.

Providing an in-depth comparison of the behaviour of the modular and conventional steel buildings is beyond the scope of this chapter. This chapter was mainly aimed at presenting the models developed and their experimental observations. The experimental observations presented in this chapter are employed to validate numerical models developed for modular and conventional steel buildings in the next chapter. These numerical models are used to provide a more in-depth comparison of the dynamic performance of both structures, together with more findings.

4.6 Summary and Conclusions

A summary of the important content that was highlighted in this chapter is presented below.

- This chapter mainly focussed on developing experimental models of the prototype modular building and conventional steel frame under consideration and conducting experimental analysis on them.

- Based on the observations from the prototype model of the modular building, a simplified scaled-down model of the modular building was developed to conduct experimental analysis. Parallel to that, an experimental model of the conventional steel frame was also developed.
- Experimental observations on the response of the modular structure under pushover analysis and ETHA were very similar to those under the FE analysis of the prototype modular building. However, it was observed during the experimental analysis that the superstructure component, after becoming free standing due to the loss of connections, underwent sliding while rocking, and then overturned.
- The experimental observations on the free-standing stack of modules (control model) reveal that the overturning of the modular building superstructure was due to the initial conditions (to the rocking, sliding and overturning phase) obtained from the failure of the connections.
- The experimental observations for the model of the conventional steel frame closely matched those for the prototype model.
- The experimental observations verified the hypothesis formed regarding the failure of modular buildings. As hypothesised, and according to the observations from the FE analysis of the prototype model, the failure in the modular building (experimental model) initiated from the edge connections and progressed to the other connections to cause the failure of the structure in an unzipping manner across the first storey. Moreover, the initiation of connection failure triggered a sudden increase in damage to the structure.
- The experimental model of the conventional steel frame behaved in a safe manner, dissipating the failure progressively even under higher seismic intensities.
- Based on the experimental observations, it can be concluded that the modular structure damage progression was abrupt compared with the conventional steel frame, when seismic intensity exceeded the design limit.

Chapter 5: Collapse Analysis and Comparison with Recordings from Physical Testings

Experimental results obtained in Chapter 4 were useful in terms of highlighting the research problem identified in Section 1.2, in terms of specific expected observations and measurements (collapse behaviour and sudden increase in damage). However, there were many practical constraints to physical testing, especially in terms of recording certain observations. For example, it was very difficult to record the forces on the connections of the most critical storey, to understand their variation under the ETHA. Moreover, it was observed that at one point during the ETHA, all the connections failed, and the superstructure component became free-standing. Measurements taken during physical tests may not be sufficient to understand this type of behaviour of the structure entirely. For these reasons, it was essential to develop numerical models of the experimental models to understand and elaborate on the experimental observations much more effectively.

As mentioned previously, the collapse of the modular structure occurred in two phases. The first involved failure of the connections and formation of a free-standing superstructure component. The second involved rocking, sliding, and overturning of the free-standing superstructure. The work described in this chapter aimed to develop the FE model of the experimental models of modular and conventional steel buildings. Further, the developed FE models were validated using the results obtained from the experimental analysis. The validated FE models were used to conduct further analysis to develop a better understanding of the collapse behaviour.

However, the analysis here was limited to understanding the first stage of collapse, which falls into the category of *collapse analysis* as per section 2.10. Hence, the analyses in this chapter are limited to studying everything that happened in the structure until the formation of a free-standing superstructure component. Chapter 6 describes a study on the second stage collapse, which involves *collapse simulation*, to which inputs from the FE models developed in this chapter were crucial.

5.1 Development of Finite Element Models of the Experimental Models

5.1.1 Finite Element Modelling of the Experimental Model of the Modular Building

The FE model was mainly developed in OpenSees [153] and subsequently verified by modelling in SAP2000 [161]. Except for the change in primary dimensions of the structure, the modelling of the module was the only change compared with the approach followed in Section 3.1.1.3.

Since the modules were manufactured using aluminium sheet elements, forming a rigid box, thick plate elements seeded with properties of aluminium were used to model the modules in both OpenSees and SAP2000. Moreover, since the plate elements provided the required rigidity under in-plane actions, diaphragms were not assigned to floor or ceiling levels. A typical module modelled in OpenSees is presented in Figure 5-1.

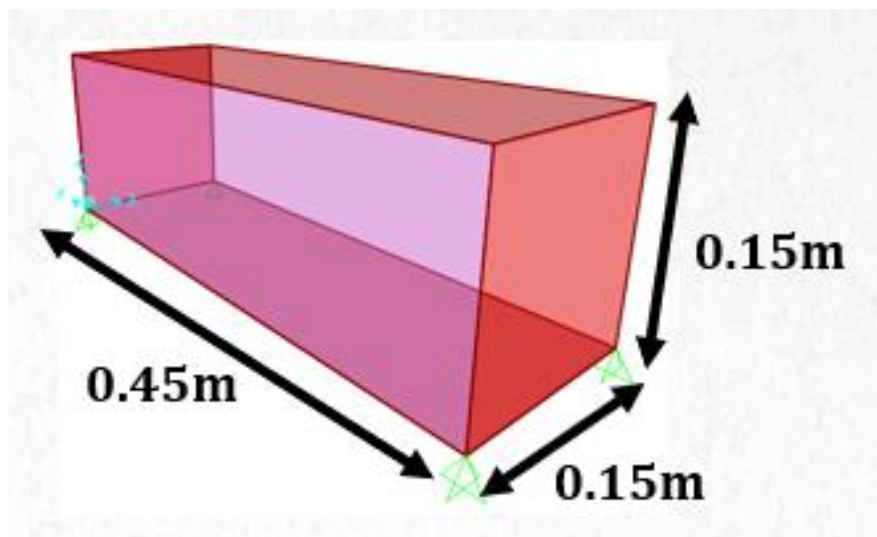


Figure 5-1 A module used in the FE model of the scaled-down model of the modular building

Modelling the inter-modular connections followed a similar approach to that employed in modelling the inter-modular connections of the FE model of the prototype modular building, as explained in Section 3.1.1.3. Two node-link elements were employed in both the OpenSees and SAP2000 software to represent horizontal and vertical connectivity between modules.

Connection properties obtained experimentally (see Section 4.2.2.2), as shown in Figure 4-6, were employed to provide the force-deformation relationship required for the link element. As was considered in modelling the vertical connection of the FE model of the prototype, a higher

stiffness was provided in vertical compression only, to simulate the limited capacity of the inter-modular connection to deform in axial compression.

Figure 5-2 below illustrates a 3D view of the FE model of the modular building developed, together with a closer view showing the connectivity between the modules, and a plan view of the building.

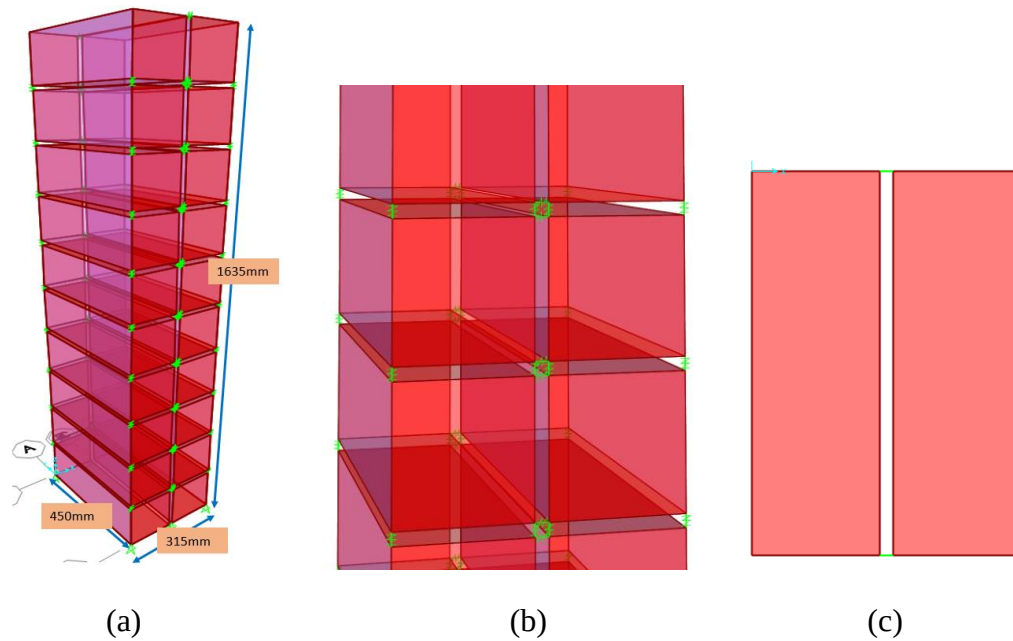


Figure 5-2 FE model of the scaled-down model of the modular building: (a) 3D view, (b) closer view of the connections between modules and (c) plan view

5.1.2 Finite Element Modelling of the Experimental Model of the Conventional Steel Building

Similar to the modular building, the experimental model of the conventional model was also modelled via the OpenSees platform and subsequently verified by modelling in SAP2000.

All frame elements making up the structure were modelled using the displacement-based beam-column element in OpenSees. The fibre element option in OpenSees was applied to all elements in the structure to capture their nonlinear behaviour.

Diaphragms were added to every storey level to incorporate the diaphragm action that would have been available via slab panels. All the masses due to the slabs and wall panels were lumped at the nodes. Figure 5-3 illustrates the FE model of the conventional steel building.

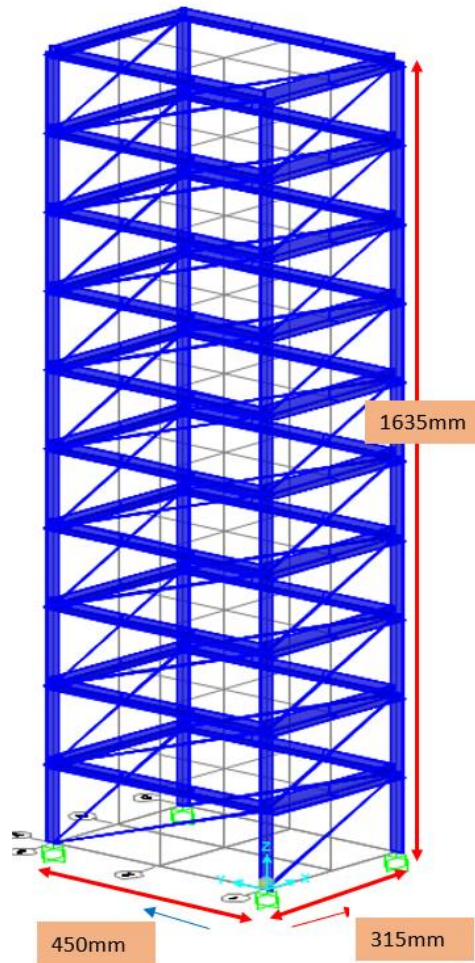


Figure 5-3 FEM of the conventional steel building

5.2 Comparison of FE and Experimental Analysis Results of the Scaled-down Model of the Modular Building

5.2.1 Modal Analysis Results

A modal analysis was conducted to obtain the natural period values and mode shapes for the structure. The modal periods and mode shapes obtained using experimental and numerical approaches are summarised in Figure 5-4. It can be seen that the modal period values obtained using these two approaches were very similar.

The natural frequency of the experimental model and the prototype model was related by a factor of $\sqrt{\lambda}$, where $\lambda = 20$. If this factor scales up the first mode period value of the experimental model, this will result in a period value of 0.85 s, which agrees very well with the first mode period value obtained for the prototype structure in Section 3.2.1(Figure 3-15).

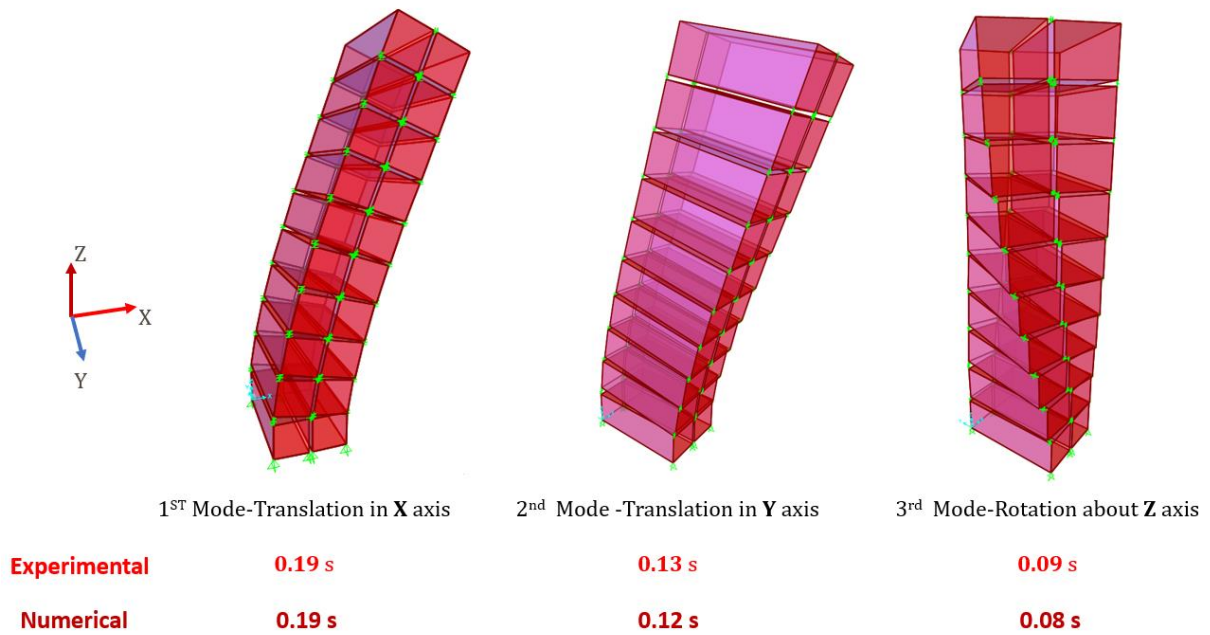


Figure 5-4 Modal analysis results for the scaled-down model obtained using numerical and experimental analyses

5.2.2 Pushover Analysis

A displacement-controlled pushover analysis was conducted in OpenSees to examine the static nonlinear behaviour of the FEM of experimental model of the modular building. Observations on the performance of the structure under incremental displacement loadings were very similar to the experimental observations discussed in Section 4.3.2.

Windward edge connections of the first storey failed with a gradual increase in roof displacement. Failure of these connections made the other connections more critical and saw them fail in an unzipping manner. Failure of all connections resulted in an overturning of the superstructure component above the first storey under subsequent displacement increments, as shown in Figure 5-5.

Apart from the cross-validation of the results with that of experimental, the FE analysis results indicated the time at which failure of the connections occurred. It was noted that the programme indicated that all the failure of connections occurred in a very short time after the initiation of the first connection failure, meaning that the failure of a single connection triggers the abrupt failure of the other connections.

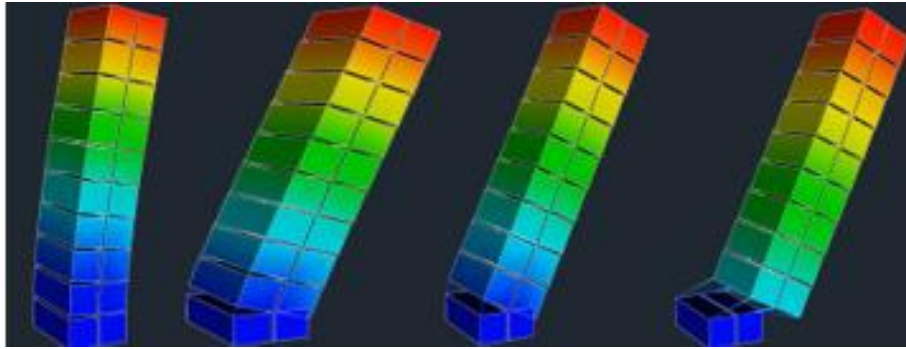


Figure 5-5 FE analysis observations on the response of the scaled-down modular building under pushover analysis

Figure 5-6 compares the pushover analysis results obtained using FE and experimental analyses. The pushover curves obtained using numerical and experimental approaches are extremely similar. This, together with closely matching modal analysis results, serves as a perfect initial validation of the numerical model of the scaled-down model of the modular building.

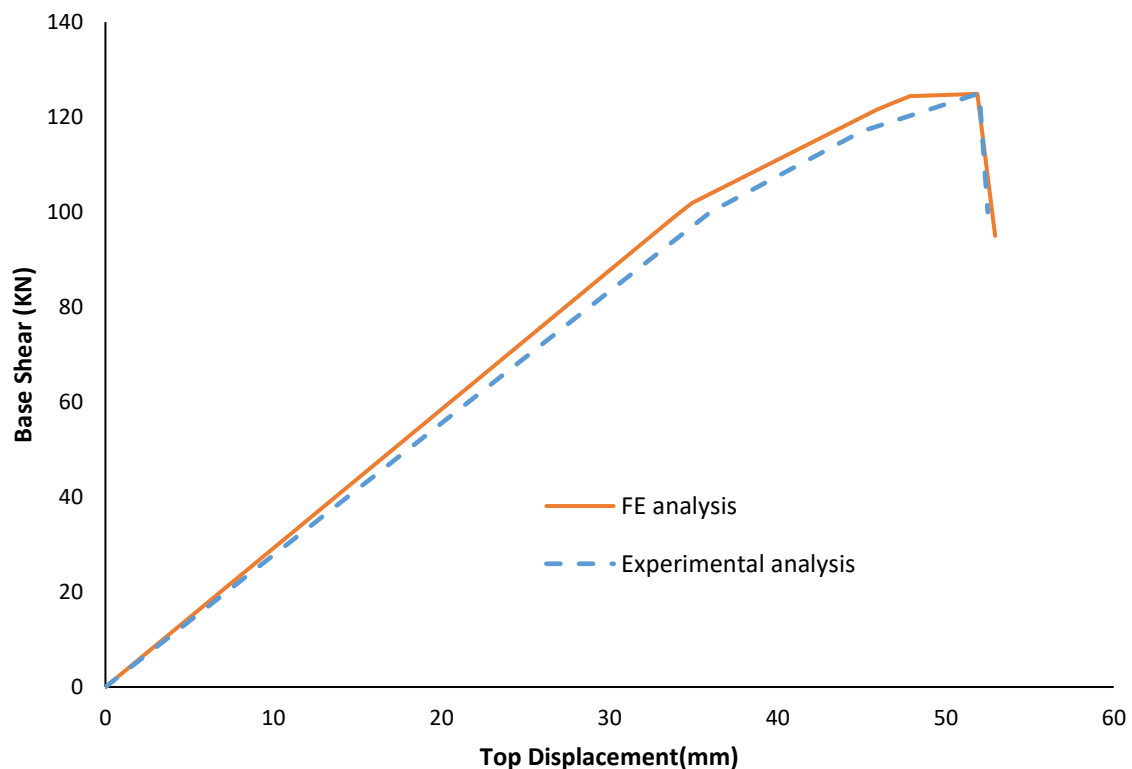


Figure 5-6 Comparison of pushover analysis results for the scaled-down modular building using FE and experimental analyses

5.2.3 Dynamic Time History Analysis Under the ETEF

5.2.3.1 Lateral Loading Under the Endurance Time Excitation Function

Since the analysis to be conducted was on a FE model of the scaled-down model of the modular building, the same ETEF presented in Section 4.3.3.1 was employed.

5.2.3.2 Behaviour Under the ETEF

Under the application of the ETEF, as observed during the experimental investigation on the shaking table, the modular structure showed high nonlinearity about the first storey connections. The recorded roof displacement was the primary damage parameter monitored during the time history application. The roof displacement time history recorded is shown in Figure 5-7. A video comparing the behaviours of the modular building under the applied ETEF obtained using experimental and numerical methods can be viewed at: <https://youtu.be/WasqydQvY18>.

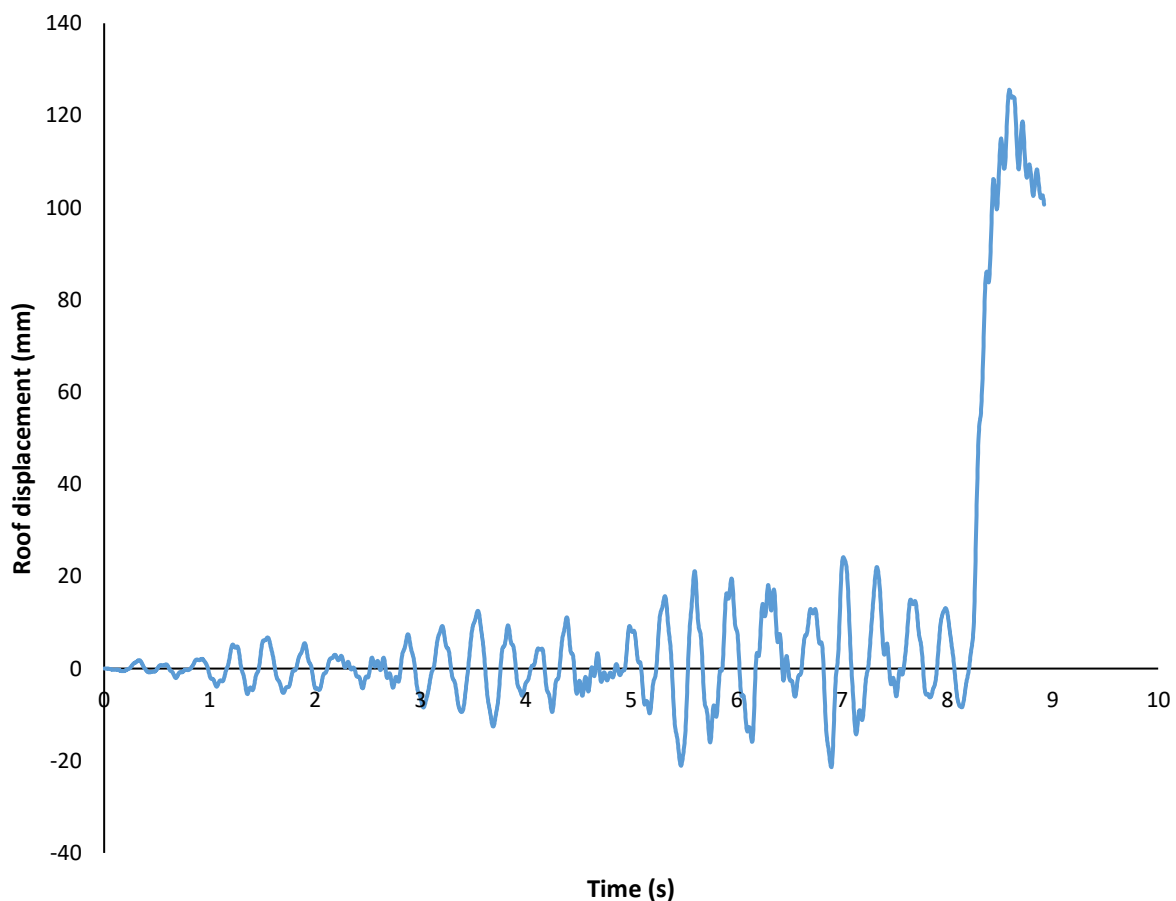


Figure 5-7 Roof displacement time history of the scaled-down model of the modular building obtained using FE analysis

The approach used in previous chapters (section 3.2.4.2.2) was employed to convert the roof displacement time history presented in Figure 5-7 to its corresponding IDA curve. The generated IDA curve, together with that obtained using experimental analysis, is presented in Figure 5-8.

Experimental results and FE results are in excellent agreement, serving as validation of the FE model. In addition to the results under dynamic loading, the pushover results (Section 5.2.2) and modal analysis results (Section 5.2.1) also were in perfect agreement with one another.

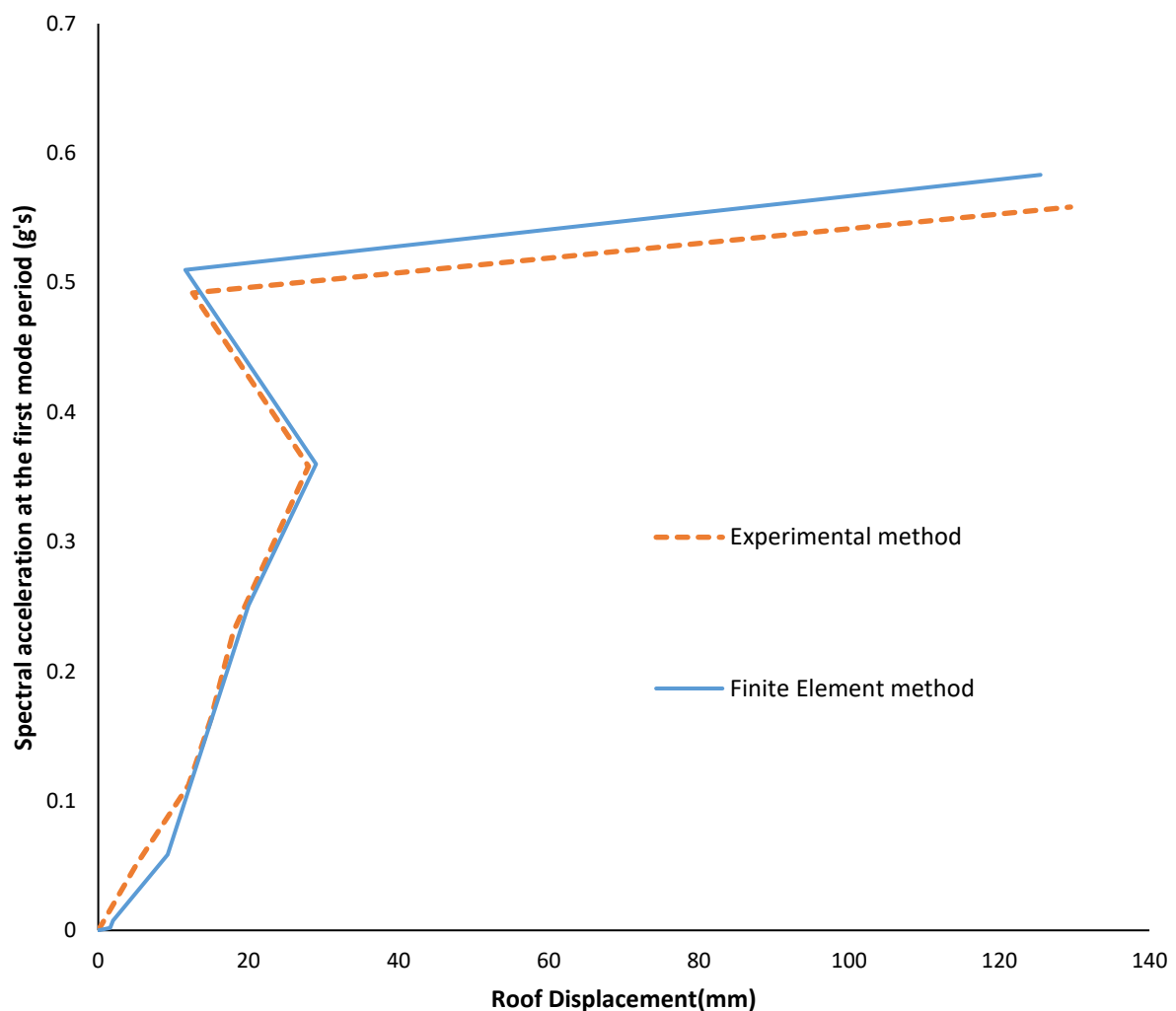


Figure 5-8 Comparison of IDA curves for the modular building obtained using experimental and FE analyses

Since the FE model had good accuracy in predicting the experimental results (modal, pushover, dynamic), the FE model was considered validated. The validated FE model was employed to extract further results on the response of the modular building under the ETEF. These results

were used to understand the cause of the abrupt variation in damage to the structure and failure of the connections.

5.2.3.3 Further Analysis Based on the Validated FE Model

The FE model developed in OpenSees was programmed to output any failure recognised during the analysis. This was very helpful in understanding the sequence of connection failure and drawing meaningful conclusions.

The sequence of failure thus obtained for connections in the scaled model of the modular building is depicted in Figure 5-9. The connections have been considered in groups of connections which fail together, as explained in the following paragraph.

Figure 5-10 illustrates the most critical storey in a modular building and a sectional view of that storey, which has four groups of connections identified as A, B, C and D based on similarity in their response. For example, all connections in group A deform and fail together because they are all situated on the same deformation plane.

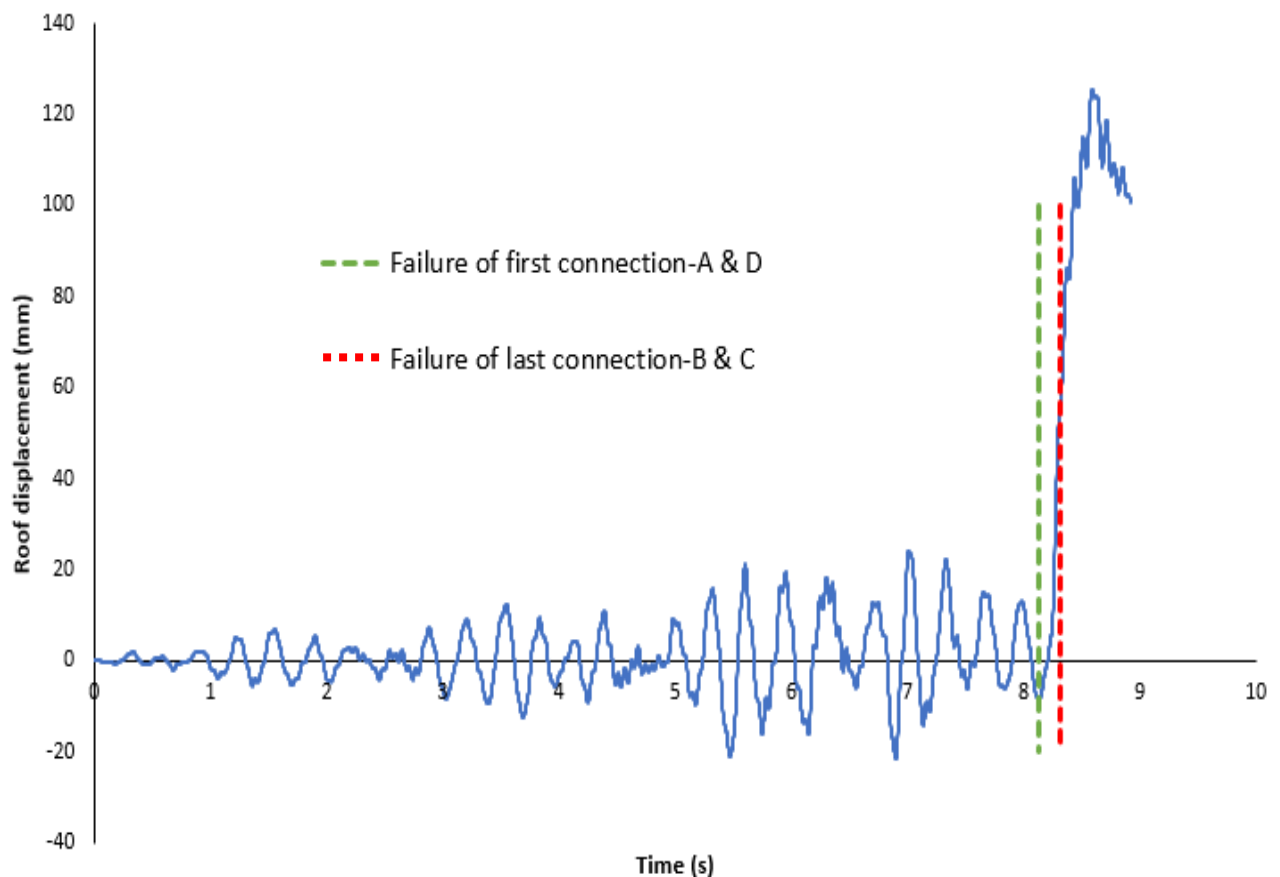


Figure 5-9 Roof displacement time history marking the critical connection failure times

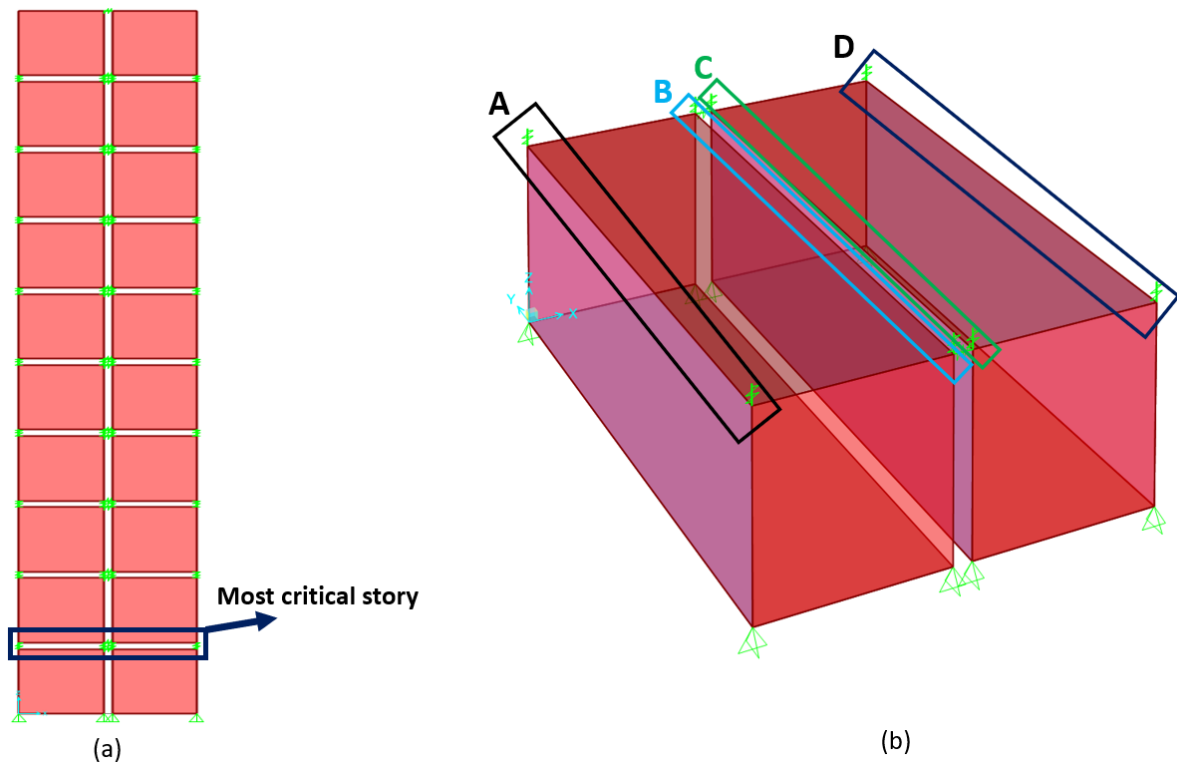


Figure 5-10 (a) Modular building showing the most critical connection storey and (b) sectional view of the most critical storey

During the application of the ETEF, it was observed that the initial failure occurred in edge connections of the critical storey (group A). Once this failure was complete, the other edge connections failed (group D), leading to failure of internal connections (group B and C). This resulted in a separation of the superstructure above the first storey.

It can be seen in Figure 5-9 that the time between the failure of the first connection and the failure of the last connection is very short, meaning that all failures occurred over a short period. This is also an indication that the failure of the first connection has triggered the failure of the rest of the connections.

Moreover, it is evident from Figure 5-9 that the failure of the first set of connections, that is, failure of group A, resulted in a sudden increase in damage progression in the structure while other connections in the same storey failed. This is because when connection group A fails, the stresses in that storey are redistributed to other connections, further increasing their stress levels, and making them more vulnerable to failure. This occurrence can be further understood by comparing the forces for all connection groups of the most critical storey.

Figure 5-11, Figure 5-12, Figure 5-13 and Figure 5-14 represent variation in connection forces (axial and shear) for group A, B, C and D, respectively. It is understood from these plots that the force variation follows a similar pattern for both groups of edge connections (A and D). Connections in groups A and D were axial force dominant throughout the time history (Figure 5-11 and Figure 5-14). Since the failure in the structure initiated from these edge connections, it can be concluded that the axial pull-out in the connection contributed more than the shear forces towards the failure of the connections. However, this can be expected at the edge connections due to the sway the structure undergoes during dynamic loading.

The interior connections, groups C and D also followed similar patterns of force distribution. However, they followed the exact opposite pattern to edge connections, being dominated more by shear force (Figure 5-12 and Figure 5-13).

Figure 5-9 shows that the first connections to undergo failure were edge connections. Figure 5-12 and Figure 5-13 illustrate the force-time histories experienced by the interior connections from the start of the time history. These force time histories indicate a sudden leap in forces experienced by these interior connections just after the failure of the edge connections (marked with broken red lines). This sudden increase in force induces failure in middle connections.

To assess the effect of failure of the connections in the first storey on those in the second storey (the connection storey above the most critical connection storey), the force-time history of one of the second-storey edge connections was plotted, as shown in Figure 5-15. It is evident from the plot that these connections also experienced a sudden increase in forces when the first failure in the building occurred (failure of group A connections). This can also be related to the sudden increase in roof displacement, which resulted in excessive sway of the structure, and hence forces on connections in other storeys.

These force time history results together with the time at which connections failure occur, obtained from the validated FE models serves very advantageous in understanding the cause of failure occurrence and the reason for the rapid progression of damage in the modular building.

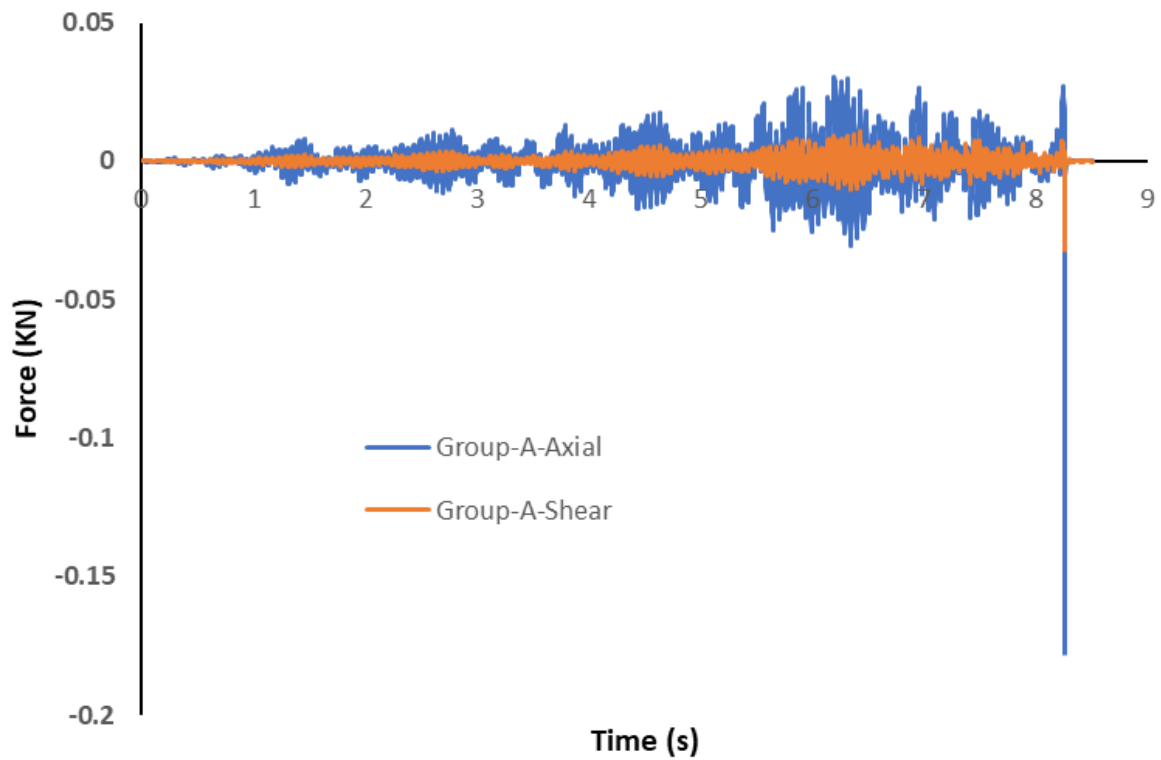


Figure 5-11 Variation of axial and shear forces of a connection in connection group A with time

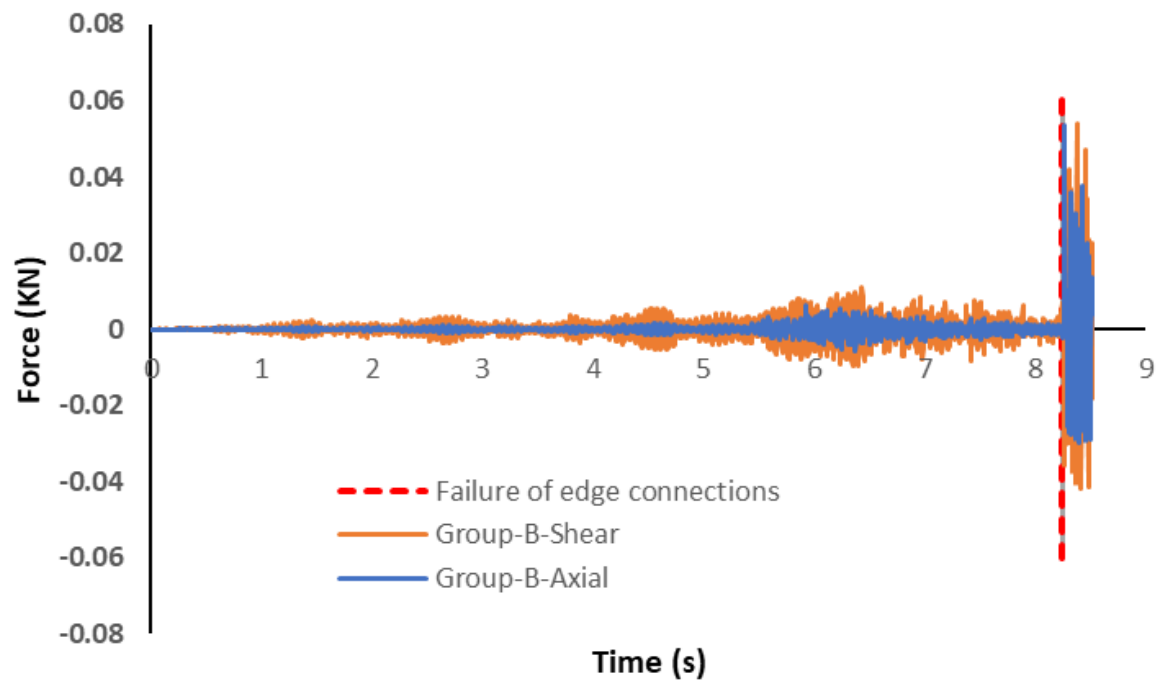


Figure 5-12 Variation of axial and shear forces of a connection in connection group B with time

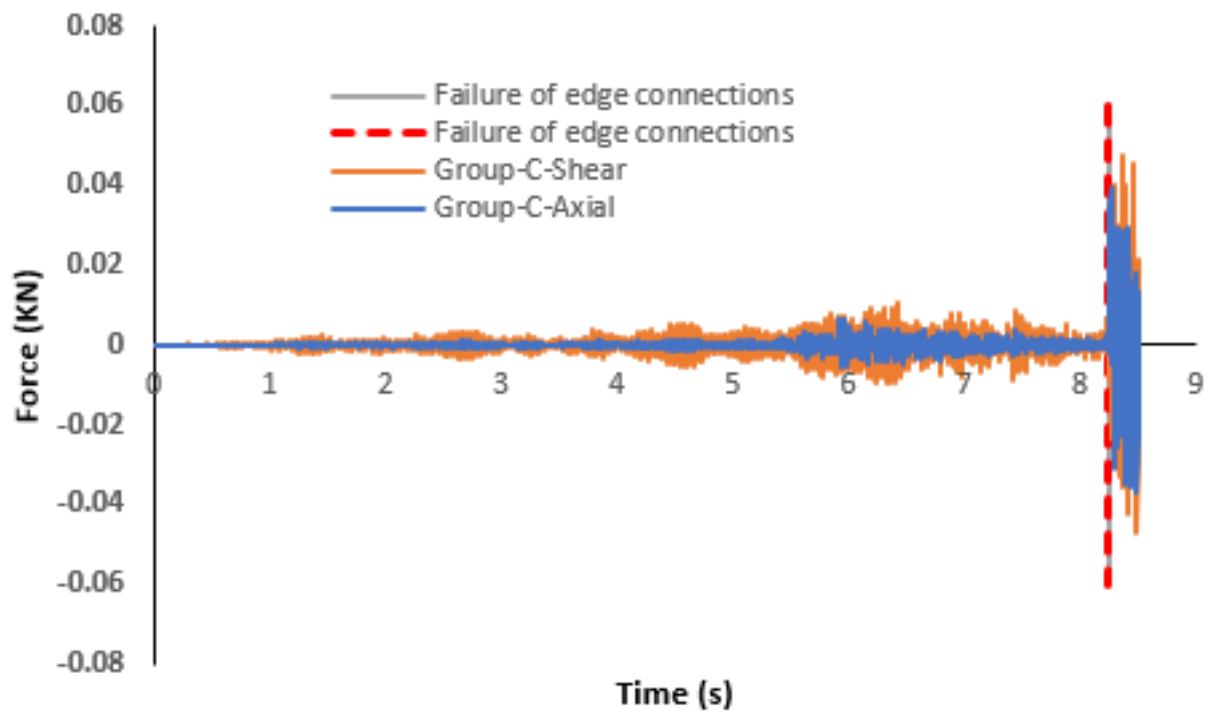


Figure 5-13 Variation of axial and shear forces of a connection in connection group C with time

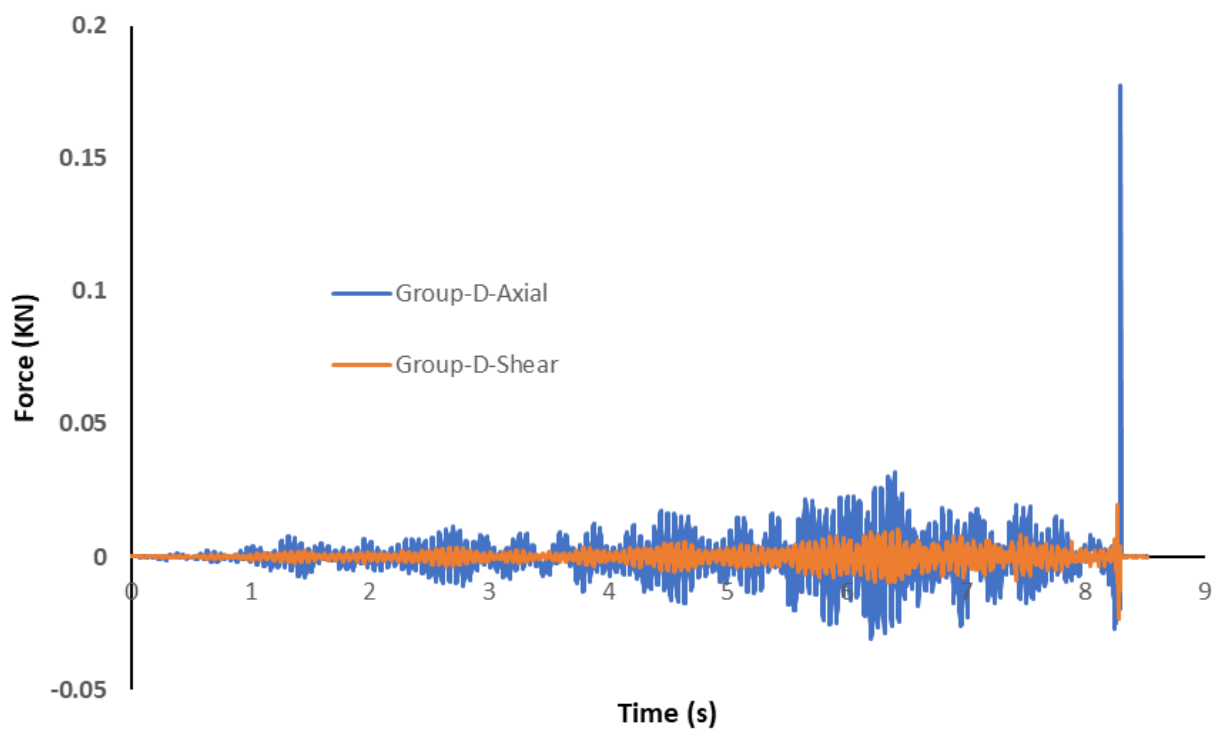


Figure 5-14 Variation of axial and shear forces of a connection in connection group D with time

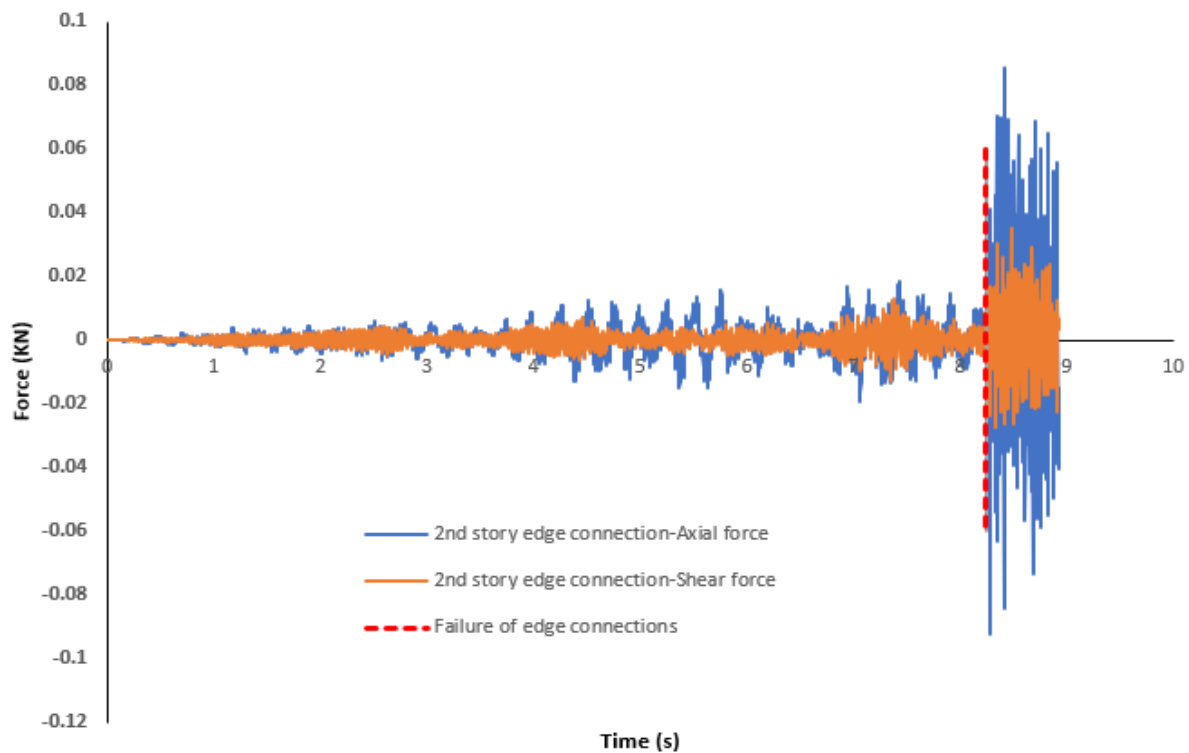


Figure 5-15 Variation of axial and shear forces of a second-storey edge connection with time

5.2.3.4 Meaningful Interpretation of the Incremental Dynamic Analysis Results

As discussed in Section 4.3.3.2, the IDA curve obtained for the behaviour of the structural damage with seismic intensity can be represented more meaningfully using the hazard return period for the earthquake as seismic intensity.

Figure 5-16 shows a comparative plot of the IDA results for experimental and FE models of the scaled-down model of the modular building, plotted together with the seismic hazard return period. It is evident from the plot that the modular structure is safe under a seismic event with a 500-year hazard return period, which is the design return period for Melbourne ground conditions. However, the structure shows a sudden increase in damage when the seismic intensity is around a 2,500-year hazard return period, which is almost twice the design intensity.

5.2.3.5 Maximum Elastic Drift Under the ETEF Obtained Using Numerical Analysis

The FE model indicated that the first yield in the structure occurred at 5.54 s in the time history. It was computed that the maximum roof displacement recorded prior to the first yield in the structure was 16 mm (Figure 5-17), which converts to a 1% drift. This agrees well with the

maximum elastic drift percentage calculated from the experimental data recorded during the shaking table experiment, explained in Section 4.3.3.3.

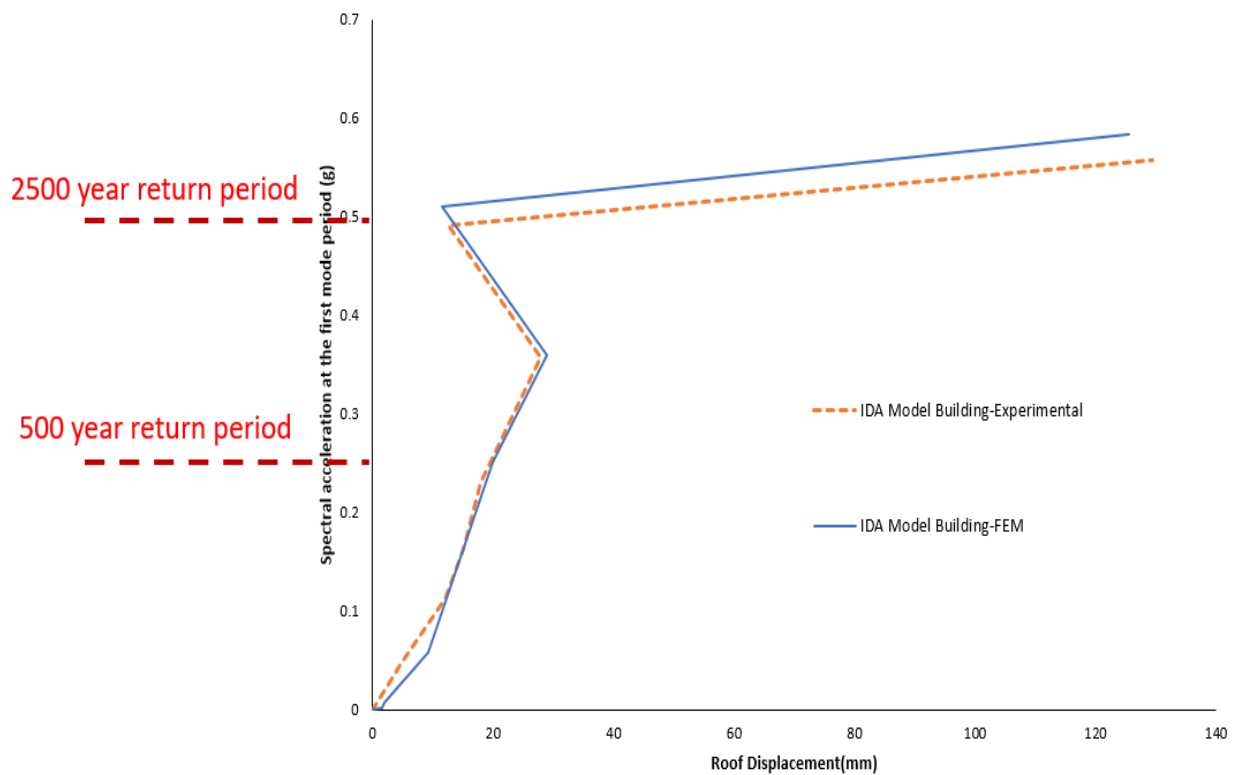


Figure 5-16 Comparison of the IDA curves for the scaled-down model of the modular building obtained using experimental and FE analyses

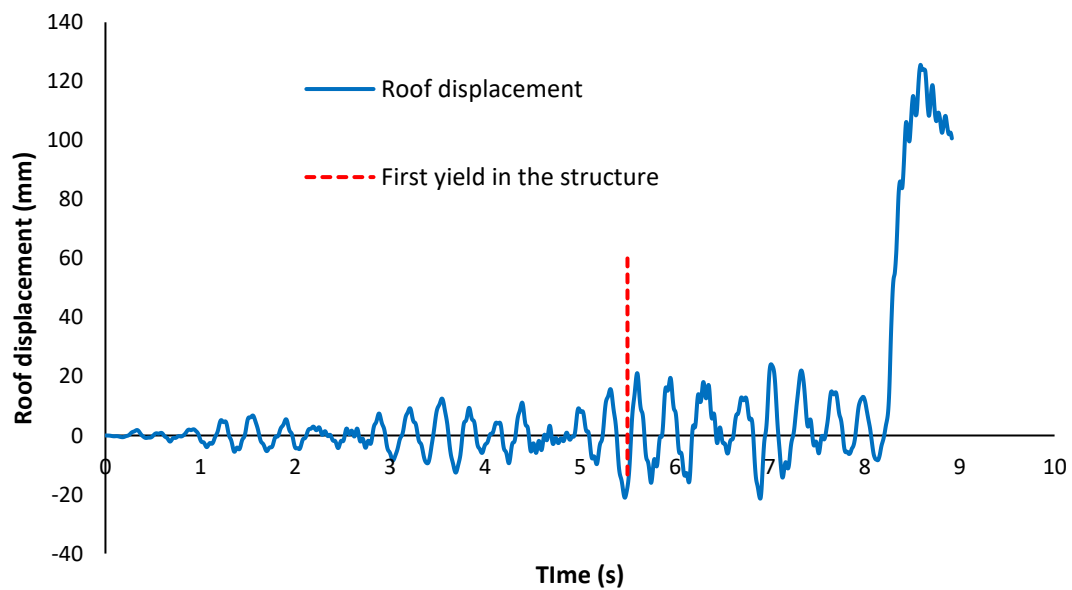


Figure 5-17 Roof displacement time history obtained using the FE model with the defined first yield in the structure

5.3 Comparison of FE and Experimental Analysis Results of the Scaled-down Model of the Conventional Building

5.3.1 Modal Analysis Results

The numerical model of the conventional steel frame developed in OpenSees was subjected to a modal analysis to obtain its primary modes of vibration, including the mode shapes. The OpenSees results were subsequently verified using the SAP200 model of the same building.

Figure 5-18 illustrates the mode shapes and period values for the structure. There is a close match between the experimental and numerical values obtained.

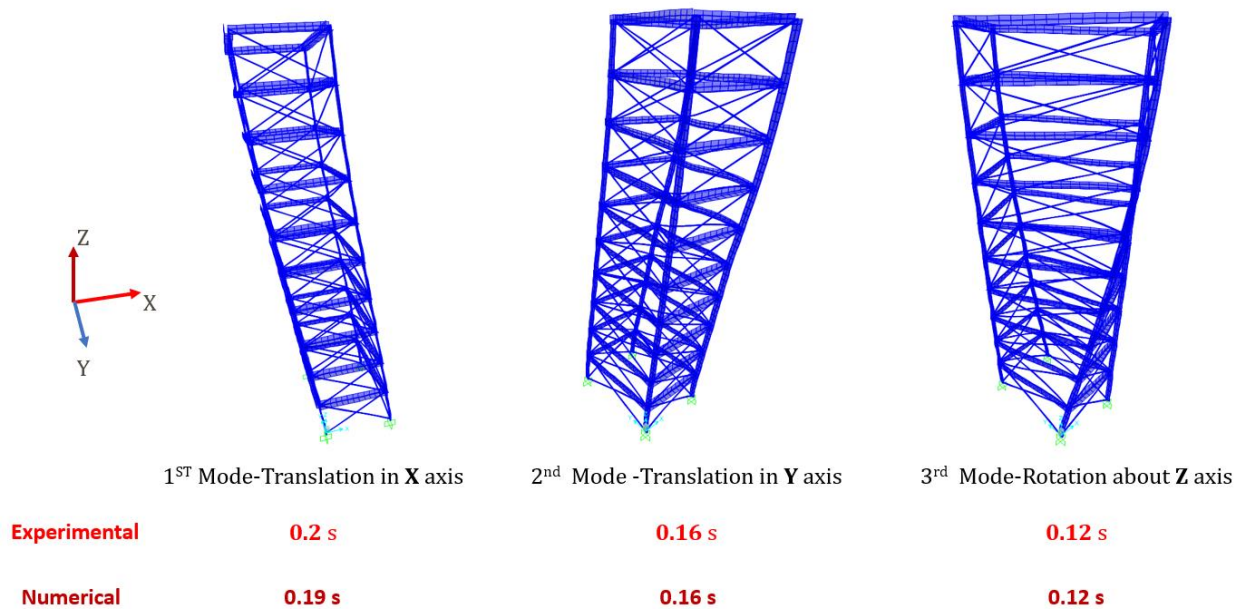


Figure 5-18 Comparison of modal period values and shapes obtained using numerical and experimental methods

The natural frequency of the experimental and prototype models is related by a factor of $\sqrt{\lambda}$, where $\lambda = 20$. If this factor scales up the first mode period value of the experimental model, it will result in a period value of 0.89 s, which agrees well with the first mode period value obtained for the prototype structure in Section 3.3.1 (Figure 3-30).

5.3.2 Pushover Analysis

Figure 5-19 presents a comparison of the pushover curves obtained using experimental and FE analysis methods. The pushover analysis conducted on the experimental model was terminated

at a certain roof deformation limit. However, the pushover analysis using the FEM continued until global instability of the structure was detected.

Observations from the FEM revealed that the initial failure in the structure started from failing of the braces and progressed to other primary components of the structure, such as columns and beams. In Figure 5-19, the pushover curves obtained using experimental and FE analysis methods are very similar, serving as a validation of the numerical model. The observations made on the failure and failure sequence of the components of the structure also showed a very close match between FE and experimental analysis methods.

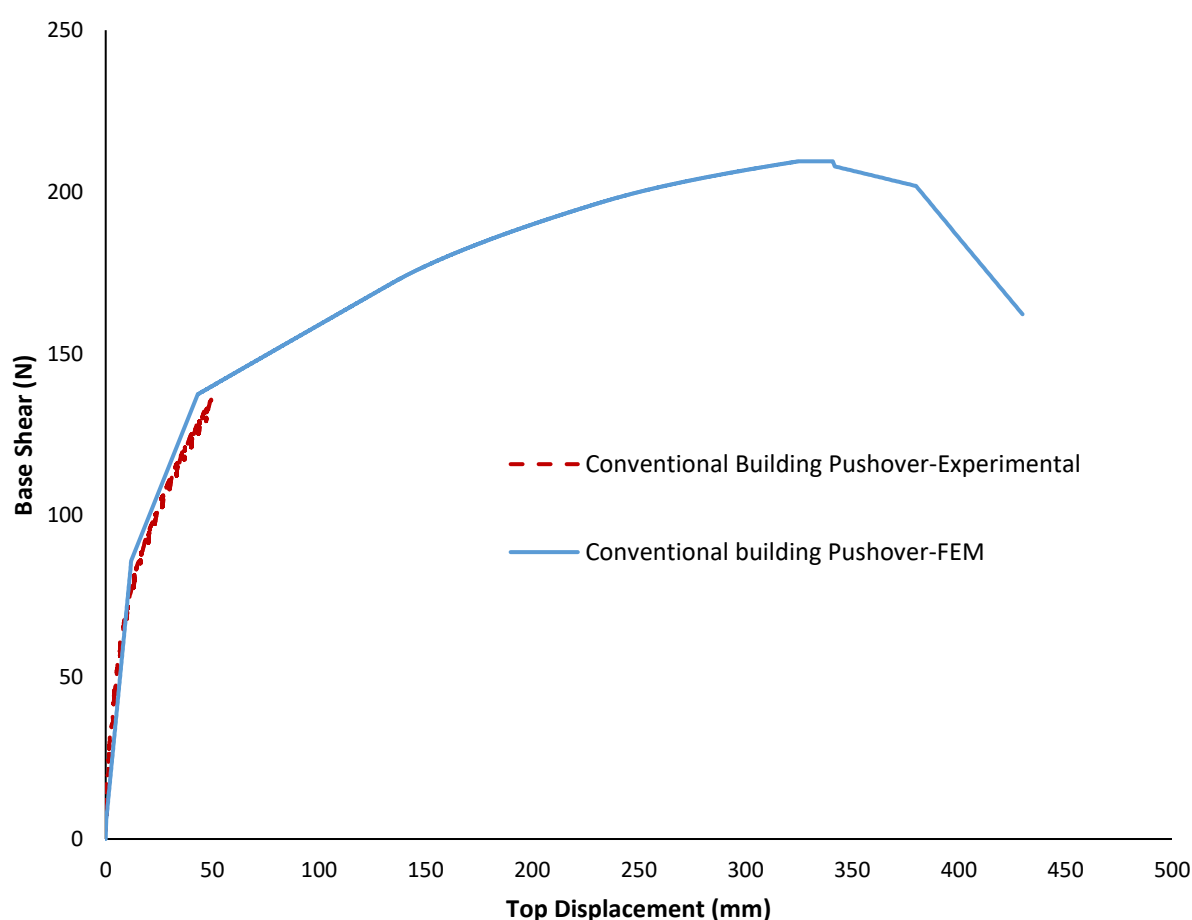


Figure 5-19 Comparison of pushover curves obtained using numerical and experimental analyses on the conventional steel frame

5.3.3 Dynamic Time History Analysis Under the ETEF on the conventional steel frame

Figure 5-20 presents a comparison between the IDA curves obtained for the scaled model of the conventional steel frame using experimental and numerical analysis methods. These IDA curves were obtained by converting the roof displacement time histories to the corresponding

IDA curves. It is evident from Figure 5-20 that the FEM is capable of reasonably predicting experimental model observations under the endurance time history applied. Both the FE and experimental model observations reveal that the conventional structure failed in a more progressive manner, resulting in a smooth IDA curve compared with that obtained for the modular structure.

The damage observed in the structure under FE analysis very closely matched that observed during the experimental analysis. The failure initiated from the ground storey braces and progressed upwards along the building height leading to yielding of the column elements starting from the ground storey. The application of the ETEF did not result in any catastrophic modes of failure in the conventional steel frame, as observed in the modular building.

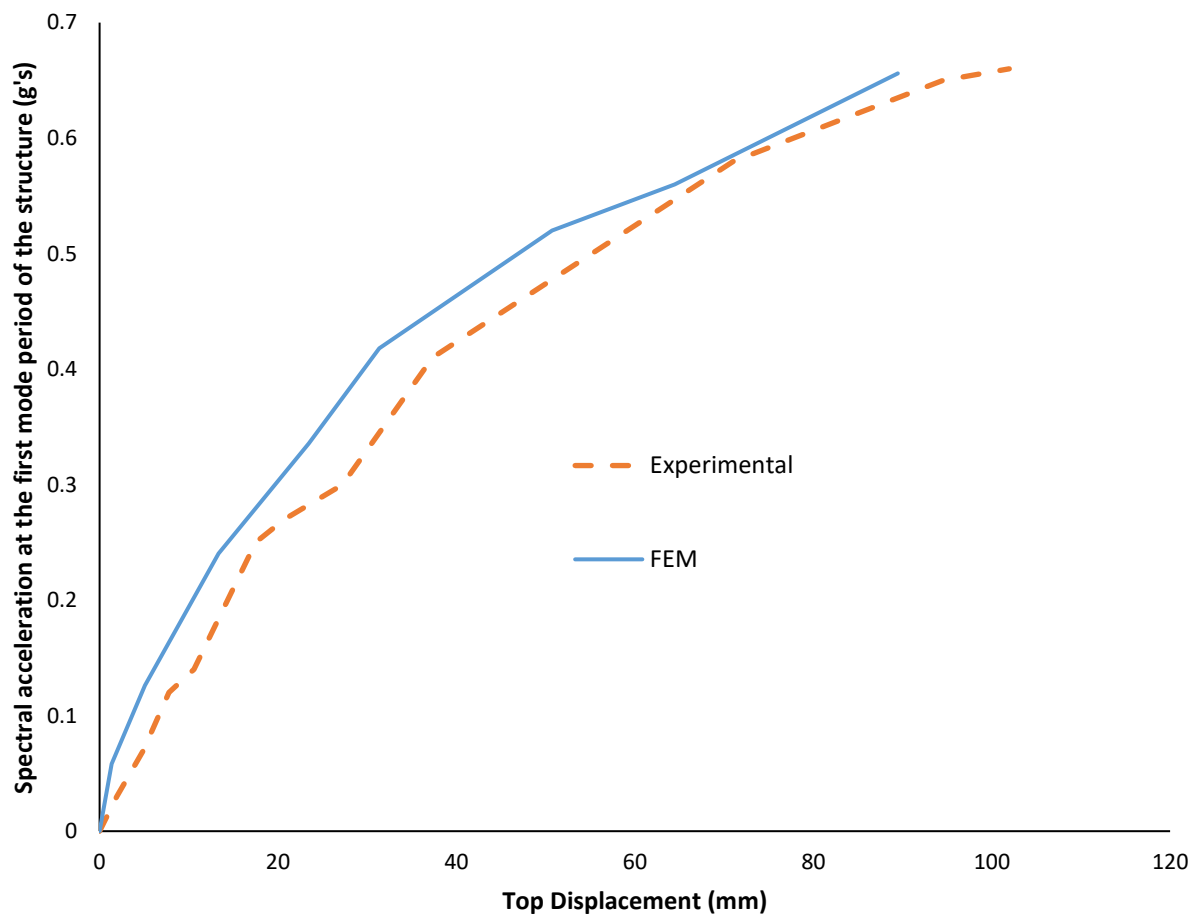


Figure 5-20 Comparison of IDA curves for the conventional steel building obtained via numerical and FE analyses

It can be further concluded from Figure 5-21 that the structure did not show any disproportionate collapse modes under seismic intensities exceeding the design limit. The

structure obviously experienced failure when the seismic design intensity jumped from a 500-year to a 2,500-year hazard return period, but the damage progression did not change abruptly as a result. This is due to the large number of components that undergo failure in a distributed manner. Moreover, the failing component were ductile enough to result in a ductile failure of the whole structure even beyond the design limits.

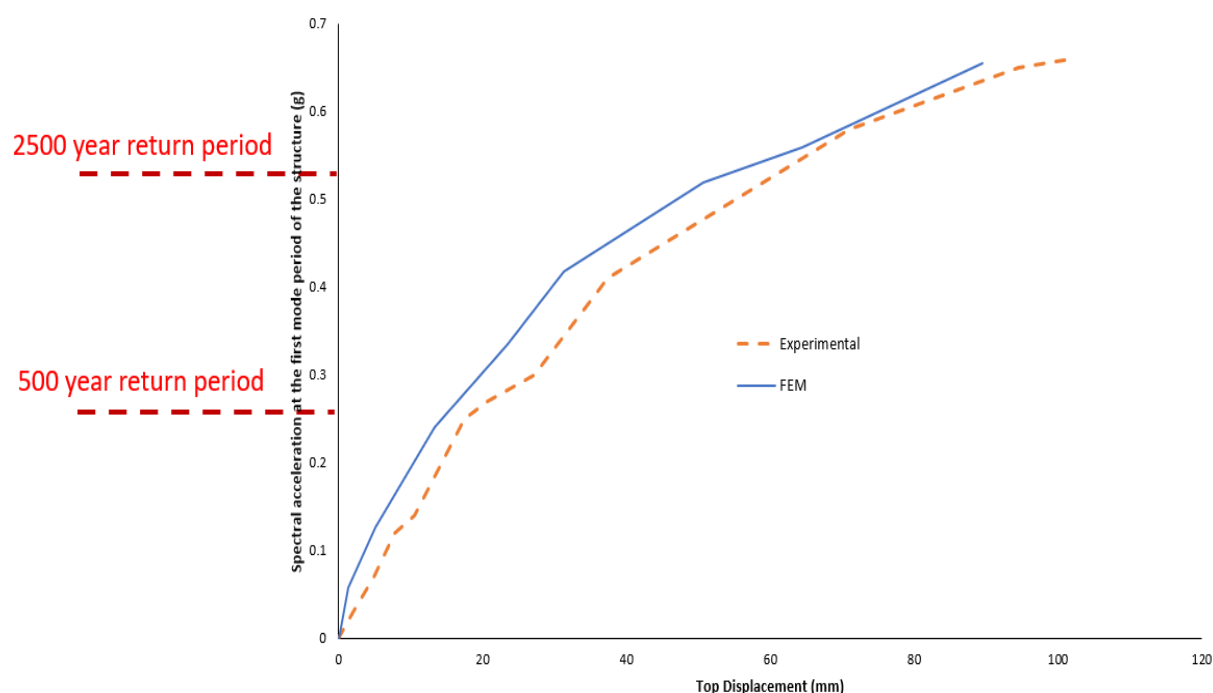


Figure 5-21 IDA curves for the conventional steel frame obtained using FE and experimental methods expressed together with the hazard return periods

5.4 Comparison of the Static and Dynamic Response of the Modular and Conventional Steel Buildings

One of the main aims of this study was to highlight any potential catastrophic nature that may be associated with the response of a modular building when subjected to seismic events exceeding design limits. However, any potential catastrophic nature would be better highlighted if the study were done in comparison with a benchmark structure. Even though the conventional steel building employed in this study was not a vital part of the study, it was intended to provide a context with which a broader audience would be familiar, that would allow them to compare the performance of the modular building and make meaningful judgements. Hence, this part of the research aimed to compare the performance of the modular structure and the conventional steel building under the various types of analysis conducted.

5.4.1 Comparison of Modal Analysis Results

In comparing modal analysis results, a direct comparison between the modular and conventional buildings would not be fair since they are categorised differently according to AS1170.4 [69] recommendations (based on structural system). A fairer approach to comparing the modal periods would be to compare them directly with code-recommended values. As per AS1170.4 [69], Eq 5-1 can be used to estimate the fundamental period of a structure:

$$T_1 = 1.25k_th_n^{0.75} \quad \text{Eq 5-1}$$

This equation was introduced previously as Eq 3-3 under section 3.2.1. The notations followed the same meanings.

Recommendations are provided for choosing values corresponding to k_t , in Clause 6.2.3 of AS1170.4 [69]. Accordingly, the modular structure can be assigned to the category ‘Other structures’, which carries a value of 0.05, resulting in a period value of 0.85 s for the prototype structure. This converts to a period value of 0.19 s in the scaled-down model, as per the scaling laws. This is in excellent agreement with period values obtained using experimental and FE analyses of the scaled-down model of the modular building, as presented in Section 4.3.1 and 5.2.1 respectively.

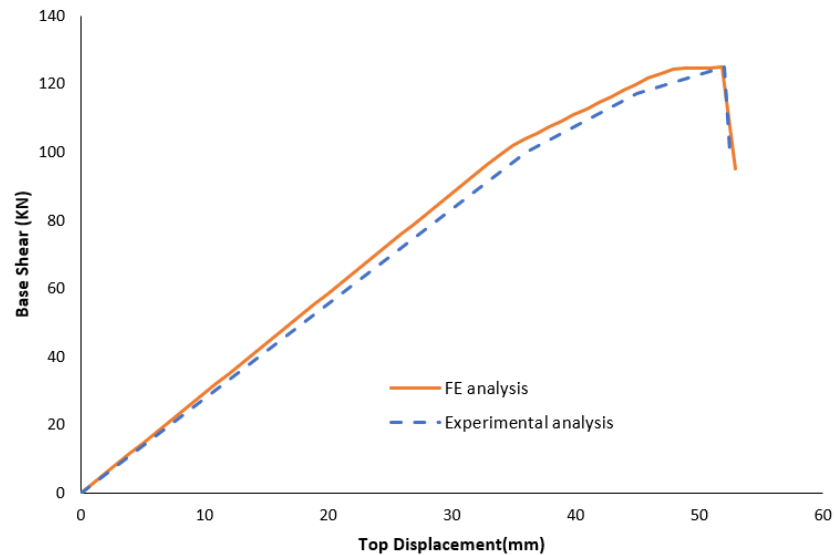
Similarly, the conventional steel frame falls into the category of ‘braced steel frames’, which corresponds to a k_t value of 0.06. This can be used to estimate the period value as 0.89 s using Eq 5-1. This period scales down to a period value of 0.2 s, which agrees very well with the experimental and FE analysis results for the fundamental period of the structure obtained in Section 4.5.1 and 5.3.1 respectively.

These results also provide a preliminary indication that the conventional and modular structures adhere to basic code requirements, taking into account design considerations.

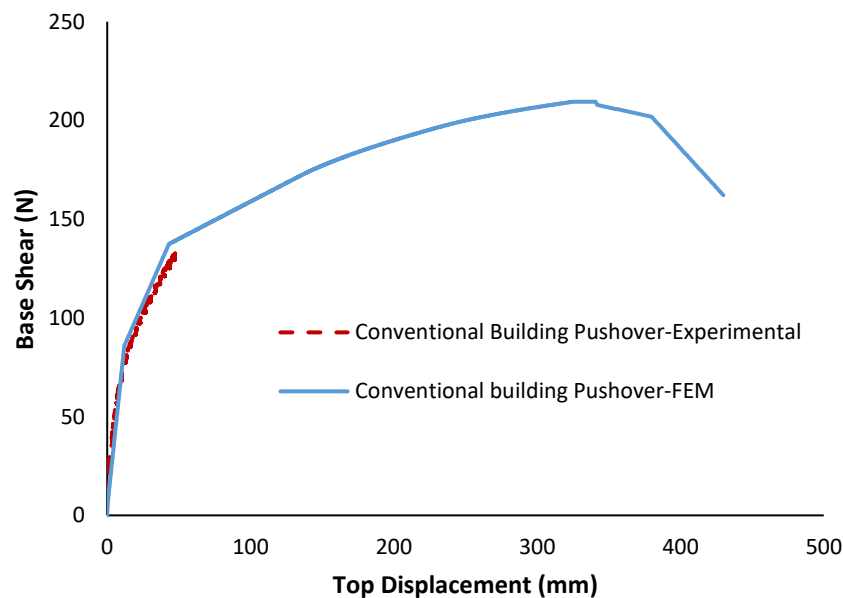
5.4.2 Comparison of the behaviour under static pushover analysis

Figure 5-22 depicts a comparative plot for the pushover curves of the modular building and conventional steel frame. It is evident from the plots that the modular structure displayed more of a brittle failure nature, while the conventional steel frame tended more towards ductile behaviour.

This observation can be related to the components that take failure in each of these structures. In the modular structure, the failure was taken directly by the bolted connections, which had little ductile capacity. In contrast, in the conventional steel frame, failure was initially taken by the braces, which had outstanding ductile capacity, and then passed on to other components of the structure.



(a)



(b)

Figure 5-22 Pushover curves obtained numerically and experimentally for the (a) modular and (b) conventional buildings

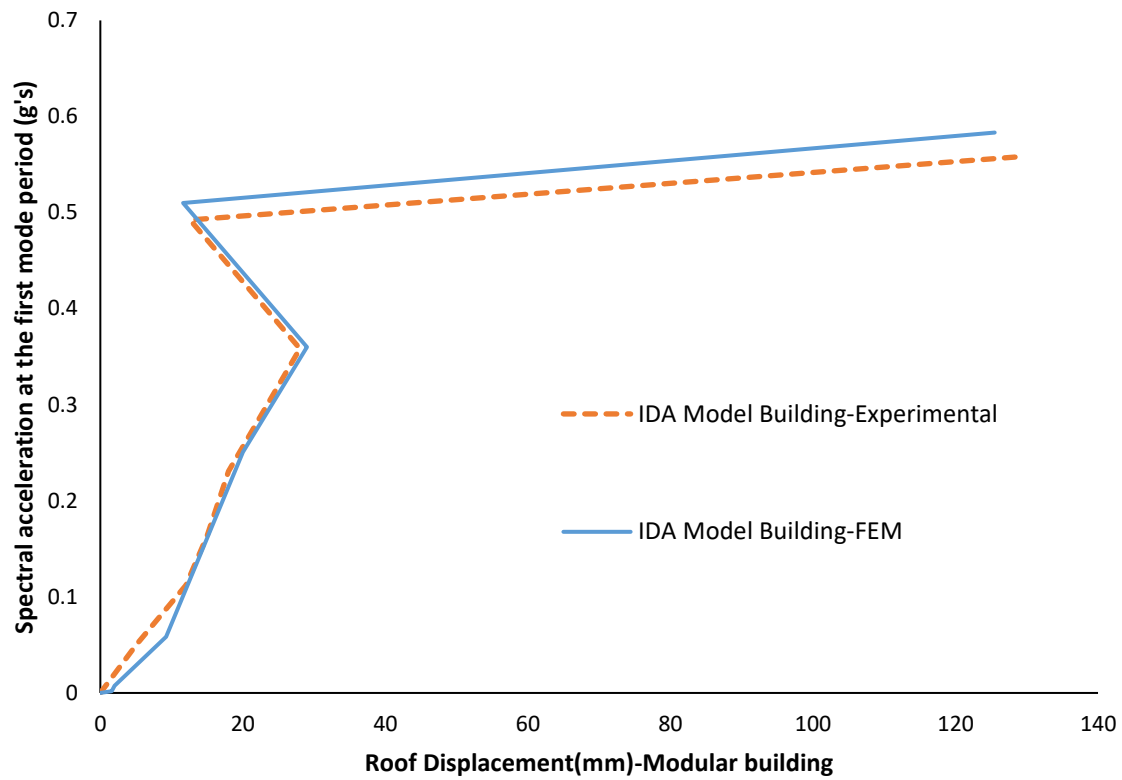
Further comparison of the plots suggests that the modular structure had little capacity after the first failure. In addition, the capacity of the structure showed a sudden drop once the connections failed. However, the conventional steel structure had substantial capacity to yield because of the large number of its elements that could undergo ductile yielding. This phenomenon was also observed during the dynamic behaviour of the structure. A more in-depth comparison of the dynamic behaviour of the modular and conventional steel buildings is presented in the following section.

5.4.3 Comparison of Behaviour Under Intensifying Dynamic Loading

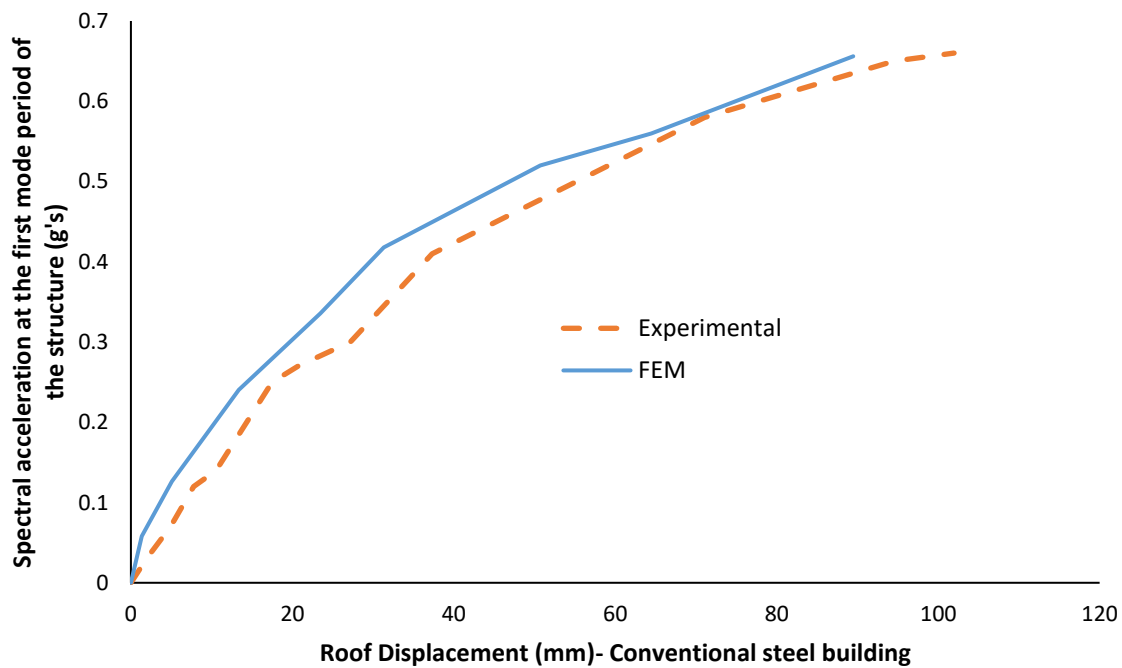
The IDA curves for the modular and conventional steel buildings obtained using numerical and experimental methods are shown in Figure 5-23 (a) and (b) respectively. One of the most apparent differences between the two curves is the sudden progression of damage observed only in the IDA curve for the modular building. This was due to the failure of connections in the most critical storey (first storey).

Connections in the modular buildings formed a primary part of the structural configuration. Here, the word ‘primary’ implies that the loss of this component will have a significant effect on structure serviceability even during normal operations; that is, as a structure standing freely under gravity without any lateral loadings on it. This is because the connections also serve in gravity load transmission. In contrast, bracings or any other in-built LFRS in a conventional steel structure do not fall into the category of the aforementioned ‘primary’ components because the loss of these does not substantially hinder the normal operations of the structure.

Moreover, in a modular building, the connections are weaker than the modules, so are vulnerable to failure, as observed during this study. When the connections fail due to their vulnerability during a dynamic event, the structure experiences major instability resulting in a sudden increase in damage. However, in conventional steel buildings, there are no such primary components in the structure that are vulnerable and have the potential to create a major instability in the structure if they fail. Thus, it is components such as braces inside the structure that fails, which protects the primary components of the structure. Even though bracings or other lateral force-resisting components can be built into a module, they will only result in further strengthening of the modules, and hence relatively weakening the connections. Moreover, whatever loading comes to the module is finally transferred to the connections, making the connections the location for a concentration of inelasticity.



(a)



(b)

Figure 5-23 Comparison of IDA curves for experimental and numerical models of the (a) modular and (b) conventional steel buildings

Further, as shown in Figure 5-11–Figure 5-14, if one critical connection fails in a modular building, this causes a stress redistribution among the other connections of the same storey and other storeys. However, these redistributed stresses are very high in the most critical storey because they have already lost one connection that could have taken some of the stress. This may lead to failure of the remaining connections in the most critical storey. This failure of all connections in the same storey results in failure of the structure across that storey, in a structural unzipping manner.

In comparison, the braces in a conventional steel building take up the excess lateral loading and yield to protect the structure. Even though yield starts initially with the critical storey, which is the ground storey in most situations, once the braces (the LFRS in the structure) of the critical storey yield, the criticality passes to the next storey, yielding the braces (LFRS) of that storey. In this way, the failure keeps progressing throughout the structure rather than being constrained to a single storey, as is the case for a modular building. Once all the braces fail and if the seismic loading is still strong enough to affect the structure, then the failure passes onto other components such as columns, resulting in them yielding. All these components, such as brace and column elements, have very high yielding capacity (ductility), resulting in greater ductility of the structure as a whole. This failure sequence for the conventional steel structure in this study managed to produce a smooth variation in damage to the structure with an increase in seismic intensity, as opposed to the sudden increase of damage observed during the experimental observations of the modular structure.

Further, it was observed during the shaking table experiment that once all the connections in the modular structure failed, the superstructure component above the first storey level, which was formed from a stack of modules, experienced sliding while rocking, and then overturned. A collapse of this nature is only possible for a modular building because of the type of failure that occurs in such buildings (unzipping of a storey). As failure in a modular building occurs across a storey, it leaves the structure with a definite failure plane, as shown in Figure 5-24. This cannot occur during the collapse of a conventional structure because of the distributed nature of failure in such structures. The availability of a definite failure plane allows superstructure component to slide, rock and overturn, leading to catastrophic failure as observed during the shaking table experiment. This stage of failure, which involves sliding, rocking and overturning is discussed in more detail in Chapter 6.

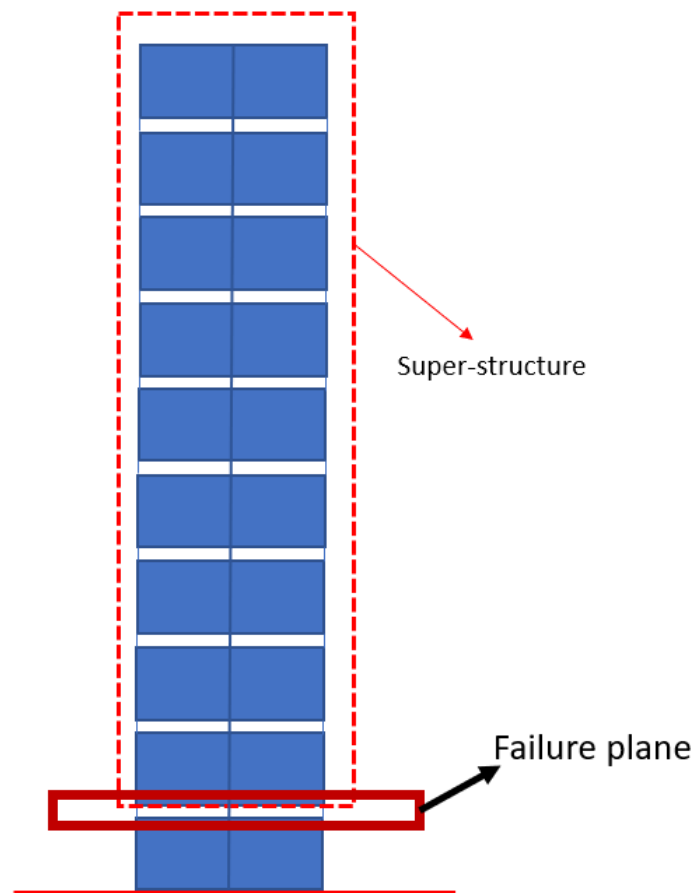


Figure 5-24 Defining of the failure plane and free-standing superstructure component of the Modular building

5.5 Comparison of Prototype and Scaled-down Model Results

This section compares the results obtained from the experimental models with those obtained using the prototype models. Although certain parameters require a constant value obtained using scaling relationships to compare the results, a comparison is essential to verify the modelling assumptions made during the development of the experimental models.

5.5.1 Comparison of Modal Analysis Results

As per scaling laws, the fundamental period for the prototype model is $\sqrt{\lambda}$ times that for the scaled-down model. Result comparison is summarised in Figure 5-25. It is observed that there is an excellent agreement between the natural periods of the prototype and scaled models.

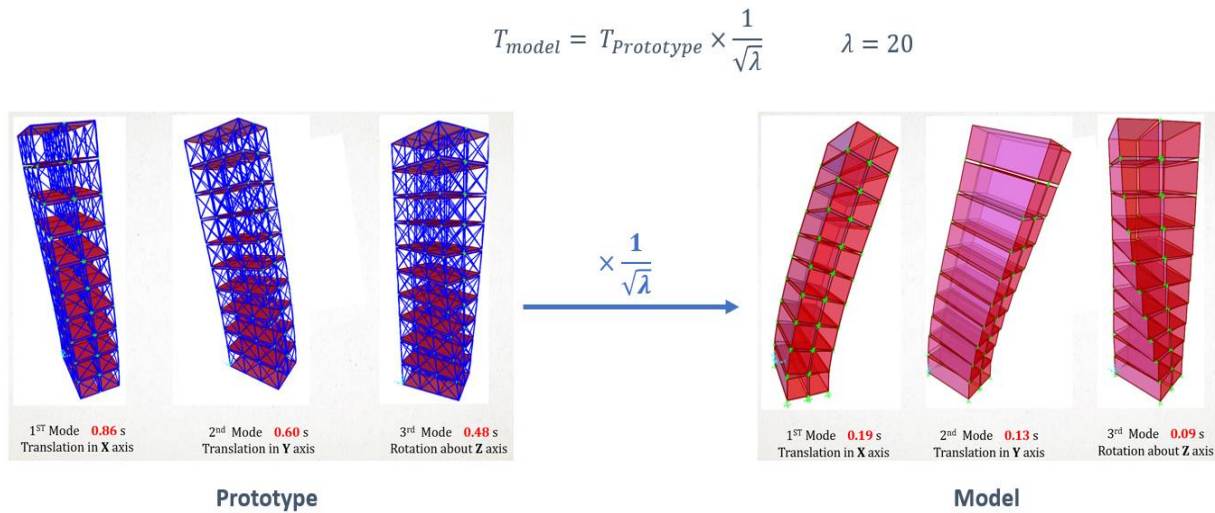


Figure 5-25 Relationship between the modal periods of prototype and scaled modular buildings

5.5.2 Comparison of Maximum Elastic Drift

The maximum elastic drifts experienced by the prototype and scaled-down models were computed for each of the analysis methods. Section 3.2.4.2.2 and 3.2.4.2.3 report that the maximum elastic drift in the prototype model was 1%. In Section 4.3.3.3, it was shown that the maximum elastic drift experienced by the experimental model was also 1%, as was the maximum elastic drift obtained using the FE models of the scaled-down models (see Section 5.2.3.5). Since the drift percentage is a ratio, it should be the same value for the prototype and experimental models.

Figure 5-26 compares the roof displacement time histories recorded for the prototype and experimental models.

In addition to comparing the drift percentages, a comparison was made of the times at which the first yield in the structures occurred (Elastic limit-Seismic intensity at which elastic response cease). The first yield in the prototype model occurred between the 25 and 30th seconds. If the scaling laws are applied, this converts to a time interval of 5.5–6.7 s based on a scale factor of $\frac{1}{\sqrt{\lambda}}$. This was cross-checked with the time at which the first yield occurred in the roof displacement time history for the scaled model of the modular building, which indicated a time interval of 5–6 s, verifying the prototype model results.

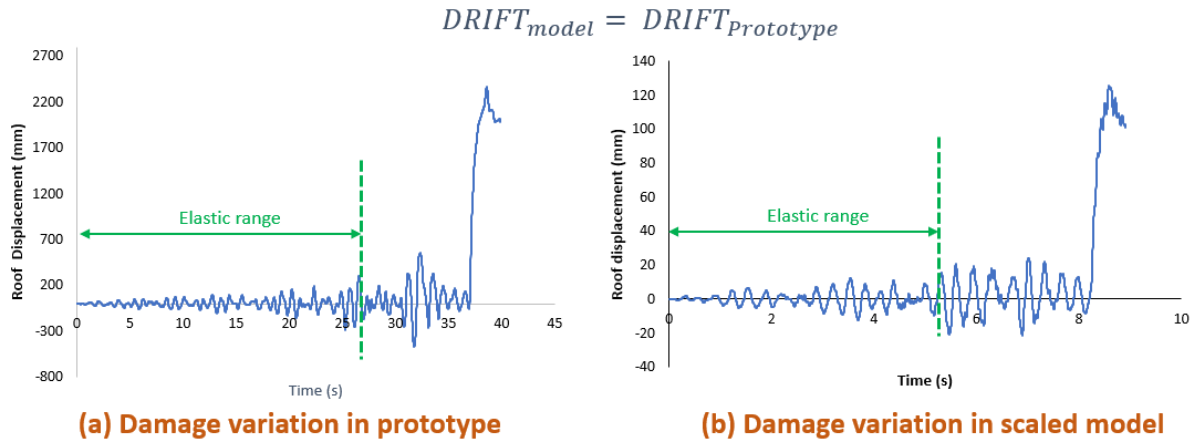


Figure 5-26 Comparison of maximum elastic drifts obtained using the (a) prototype and (b) experimental models

5.5.3 Comparison of Nonlinear Dynamic Response

Figure 5-27 presents comparative plots of variation in damage (roof displacement) of the structure in response to seismic intensity for the prototype modular building and the scaled-down model of the modular building. As per the scaling laws, the acceleration was not scaled for the scaled-down model, which is reflected in the IDA plots. It can be seen that both the scaled and prototype models follow a similar pattern of variation in their IDA curves. Moreover, if displacements are considered, it can be observed that the displacements in the scaled-down model have scaled down by a factor of $\lambda = 20$, closely matching those in the prototype model.

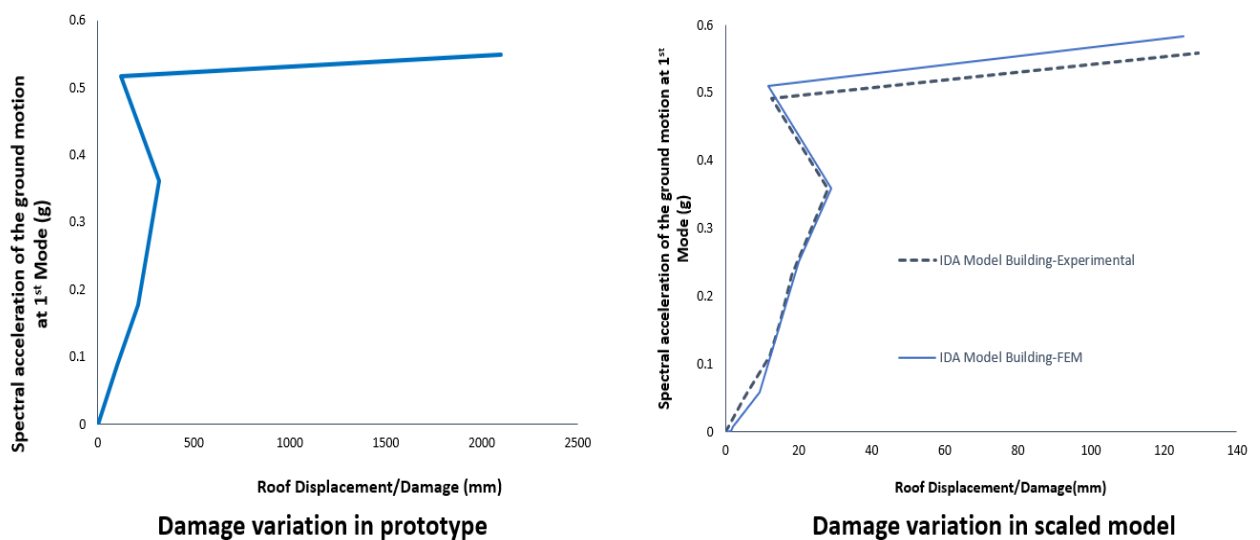


Figure 5-27 Comparison of IDA curves obtained for the prototype and scaled-down models of the modular building

All the above comparisons between the prototype and scaled model of the modular building indicate that the scaled model was capable of representing the prototype model to a great extent with outstanding accuracy. This serves as a verification of the modelling assumptions made in Section 4.2.2 when developing the scaled-down model of the modular building.

5.6 Summary and Conclusions

A summary and major conclusions that can be drawn from this chapter are presented below.

- This chapter focussed on the development of FE models of the experimental models of modular and conventional steel buildings. These models were employed to carry out further analysis to understand the first phase of collapse (up to failure of all the connections and formation of a free-standing superstructure).
- FE models of both structures were validated by experimental results, which showed excellent agreement in terms of modal, pushover and nonlinear dynamic THA results.
- Validated FE models were employed to understand and compare the failure progression behaviour of modular and conventional structures.
- Connection failure time recorded for the FE model of the modular building indicated that the time between failure of the first and last connections in the structure was minimal (Figure 5-9). This indicates that just one connection failure has a significant effect on the abrupt failure of other connections in the same storey. However, in the conventional steel frame, the failure progressed over a long period, during which the braces along the structure height yielded progressively.
- It was observed that the damage (roof displacement) to the modular structure plunged after the first connection failure. There was no such abrupt increase in damage observed due to failure initiation and progression in the conventional steel frame (Figure 5-24).
- In terms of the connections of the most critical storey of the modular building, it was observed that the edge connections initiated failure. These behaved according to an axial force dominant pattern, suggesting that the axial pull-out forces in the connections contributed most to failure initiation (Figure 5-11 and Figure 5-14).
- In contrast to the dominant pattern of axial forces followed by edge connections, the interior connections of the most critical storey behaved in the exact opposite way: they experienced higher shear than axial forces throughout the time history (Figure 5-12 and 5-13).

- Comparison of the forces in the interior and edge connections showed that the forces in the interior connections magnified abruptly as soon as the edge connections failed (compare Figure 5-11 and 5-12).
- The forces in a connection in the storey immediately above the most critical storey (second storey) were studied to understand the effect of the failure of connections in the most critical storey on the forces in second-storey connections. It was observed that the forces in these connections also amplified. However, the connections in the most critical storey were the first to fail.
- Comparison of the modal period values for the modular and conventional steel frames obtained experimentally and numerically with those calculated using code-recommended equations for calculating the fundamental natural period (based on corresponding factors for different structural types) indicated perfect agreement.
- The response behaviour of both the modular and conventional structures was satisfactory under seismic events with around a 500-year hazard return period. However, the modular structure failed abruptly, resulting in overturning of a massive block of the superstructure when the seismic intensity was that of an earthquake with 2,500-year hazard return period. This brings into doubt the safety of modular structures in terms of adhering to conventional design codes. Such codes may guarantee a safe failure response in conventional steel buildings, but do not guarantee safe performance for a modular building under intensities beyond the design limit.
- Various assumptions were made in the development of the experimental models to simplify and increase the accuracy of experimental analysis. The validity and accuracy associated with these simplifications were verified by comparing the results obtained for the scaled-down model with those for the prototype model. The results were in perfect agreement.

Chapter 6: Collapse Simulation of the Modular Building

Experimental observations presented in Chapter 4 highlighted the existence of two main phases during the collapse of the modular building. The first phase, which involved failure of the connections (collapse analysis), was studied in detail in chapter 4 and chapter 5. The FE models developed were capable of representing the experimental models with great accuracy. Hence, the FE models were employed to examine the cause of failure initiation, the abrupt increase in damage and progression of failure (analyse the collapse).

However, the validity of the FE models changes once all connections fail, and a free-standing breakaway building tower is formed above the plane of failure of the bolted connections. Moreover, this phase of the collapse which was observed during the shaking table experiment involved, sliding while rocking and then overturning. Hence, it is imperative to model the behaviour of the superstructure using numerical simulations, to determine if overturning would actually occur and to identify factors that exacerbate the risk of overturning.

Further, results of the shaking table experiments involving the dynamic testing of the free-standing breakaway tower (section 4.4) were employed to elaborate the overturning process and to validate the numerical simulations.

6.1 Experimental Observations

6.1.1 Different Phases of Building Behaviour Observed During the Shaking Table Experiment

The experimental observations on the shaking table could be categorised into three main stages of behaviour of the structure under the applied ETEF. The stages thus defined are summarised in Figure 6-1.

Stage 2 and 3 correspond to the first and second phases of the collapse response of the structure, as mentioned in the previous chapters. Stage 1 and 2 are examined in detail in previous chapters. This chapter deals with collapse simulations of the behaviour of the damaged building in stage 3.

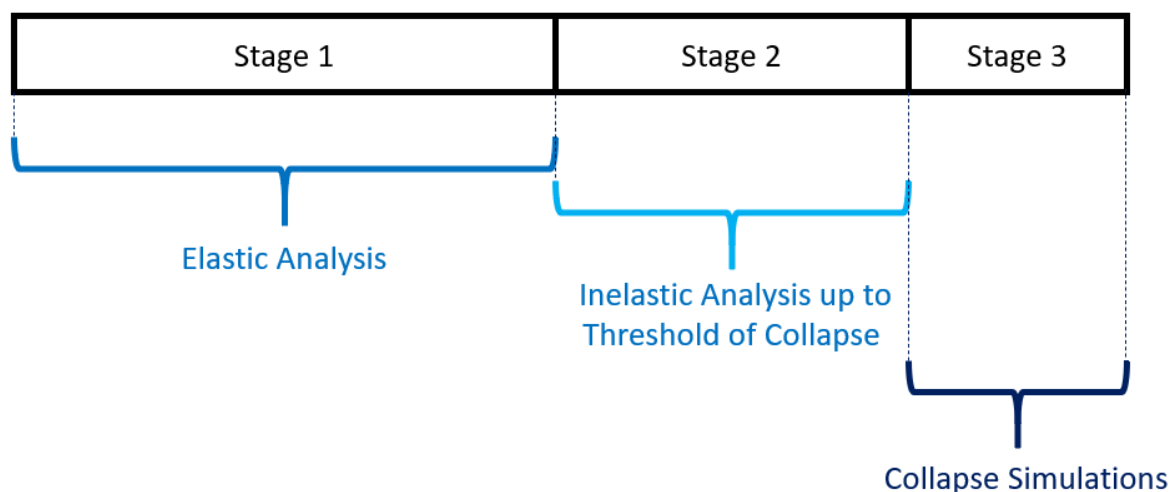


Figure 6-1 Main stages of the structural response of the modular building under the ETEF

It was observed during the shaking table experiment that as soon as all the connections failed, the structure underwent sliding together with rocking, and then underwent pure rocking to overturn. These observations can be viewed in a video of the experiment available at <https://youtu.be/WasqvY18>. To provide a visual summary of the observations, some combined screenshots of the response at different times are shown in Figure 6-2.

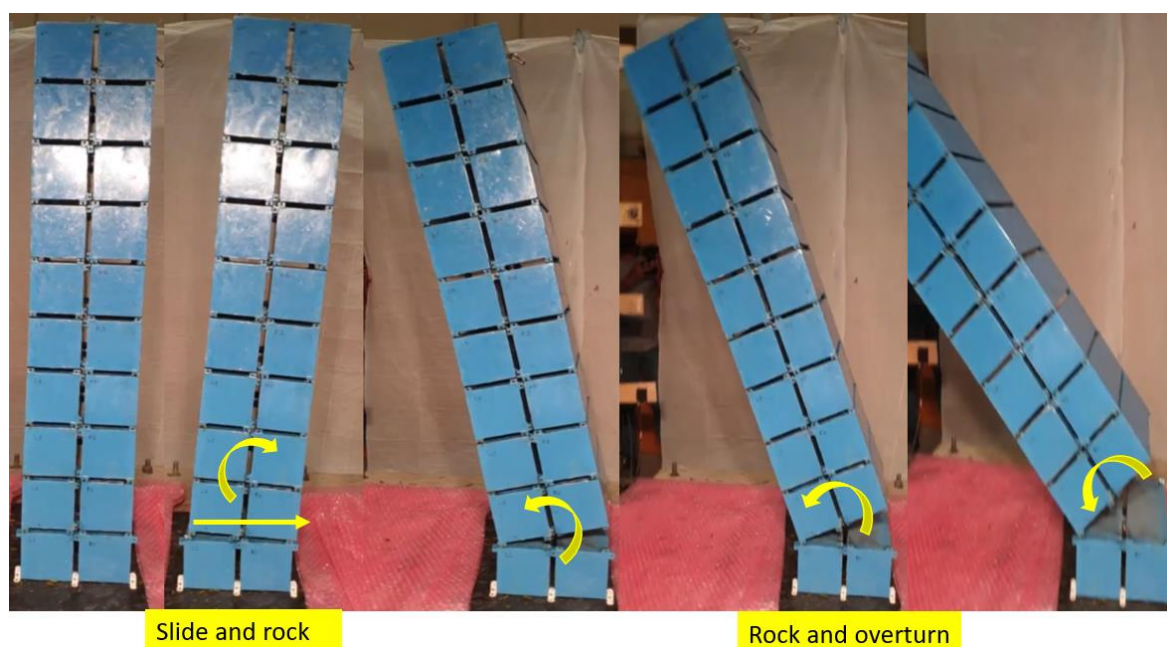


Figure 6-2 Observations during stage 3 of the modular structure response

The experimental observations showed that the superstructure component (stack of modules of nine storeys) behaved as one entity. This entity responded almost identically to a rigid body.

Hence, this study was initially conducted treating the superstructure component that underwent sliding, rocking, and overturning as a rigid body. The latter part of the study incorporates the real flexibility of the stack of modules to study the effect on the response observed of taking into account the flexibility of the superstructure. The rigid body assumption is depicted in Figure 6-3.

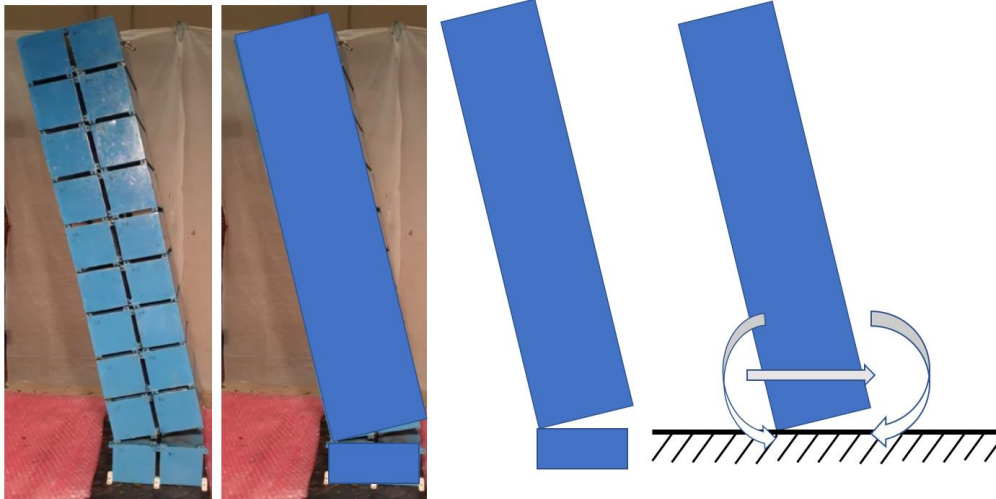


Figure 6-3 Rigid body assumption for the superstructure component

6.1.2 Conditions for Overturning

Due to the nature of construction of modular buildings, failure in this type of structure will be broadly similar to that demonstrated through the shaking table experiments. However, a collapse of this nature is not acceptable; it is catastrophic. Hence, the cause of the overturning must be examined to identify strategies to enhance the safety of the structure under seismic events, especially when seismic intensity exceeds the design limit.

The main requirement for a free-standing structure to overturn under the influence of a seismic event is displacement of the ground exceeding at least half of the width of the structure [162]. As a first step towards understanding the cause of overturning, the ground motion displacement is compared with the structure's half-width.

6.1.2.1 Quantifying the Overturning Against Ground Motion Displacement

In this section, the ground motion displacement is checked against half of the width of the structure to check whether the condition mentioned in the previous paragraph is satisfied. The displacement time history of the ground motion is illustrated in Figure 6-4.

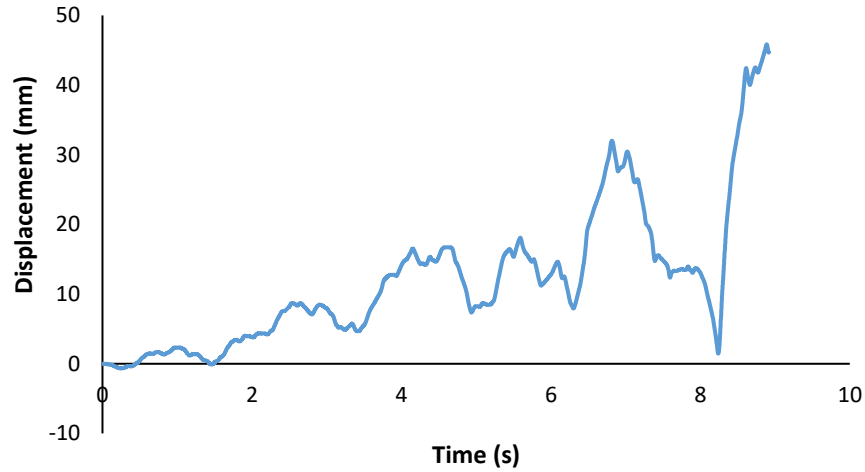


Figure 6-4 Displacement time history of the applied ground motion

In the case study as described, the peak displacement demand of the earthquake ground shaking is around 45 mm (Figure 6-4) whereas the base dimension in the direction of ground movement is 315mm (Figure 6-6). According to Ref [162], a free-standing rectangular block is safe from overturning if the peak displacement demand is within half of the base dimension of the block (Figure 6-5). Hence, the structure is much wider than the input ground motion displacement. This is an alarming result highlighting that something special has happened in the modular structure to trigger overturning of the superstructure without the necessity to satisfy one of the main requirements outlined in this section.

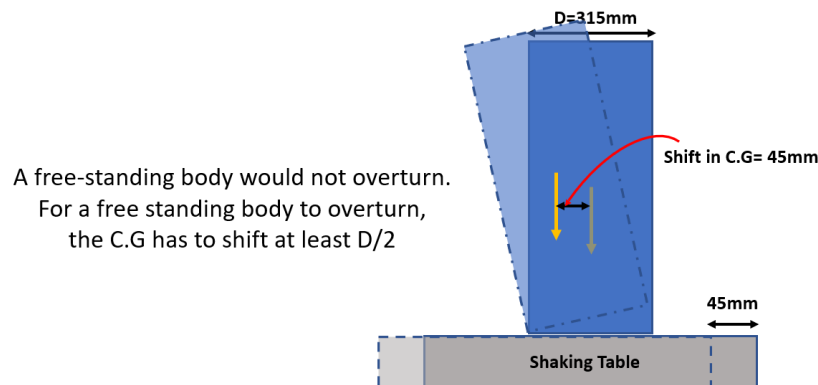


Figure 6-5 response of a free-standing body under the considered ground motion

However, the rule of the thumb as stated only applies to a free-standing object which is free to rock (rotate in the vertical plane throughout the event). For a bolt connected modular building, rocking only occurs as the connections become unzipped. In this sudden switch from a cantilever system to a free-standing block, overturning would be possible if the response

spectral velocity (V) of the building at the level of the centre of inertia exceeds the critical velocity ($V_{critical}$) regardless of the peak displacement demand. Details of this energy approach of assessing the risks of overturning can be found in section 6.1.2.2 below.

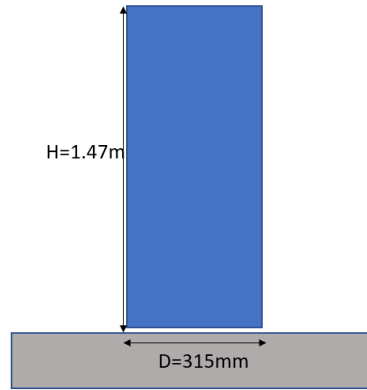


Figure 6-6 Major dimensions of the superstructure component idealised as a rigid body

6.1.2.2 An Energy Approach to Quantifying Overturning

When the structure switches to a free-standing stage, just after the failure of all its connections (transfer point from stage 2 to stage 3 as per Figure 6-1), the rock and slide motion of the superstructure starts with an initial velocity of the superstructure. This velocity obtained results in some kinetic energy in the free-standing superstructure, which converts to potential energy as a result of rocking. Considering an energy balance, it is possible to derive a critical value for the initial velocity for stage 3 that can cause the overturning of the rigid body. The derivations are described below. Figure 6-7 illustrates a free-standing rigid body subjected to an initial velocity and starting to rock. The notations used in this figure have the following meanings:

- W is the width of the rigid body
- h is the height of the rigid body
- θ is the angle of rotation of the rigid body
- β is the angle the base of the rigid body forms with its diagonal
- r is the half the length of the diagonal
- Δ is the lateral displacement of the top of the structure
- $\Delta_{C.G}$ is the vertical shift of the centroid
- V is the top velocity of the superstructure
- M is the mass of the superstructure.

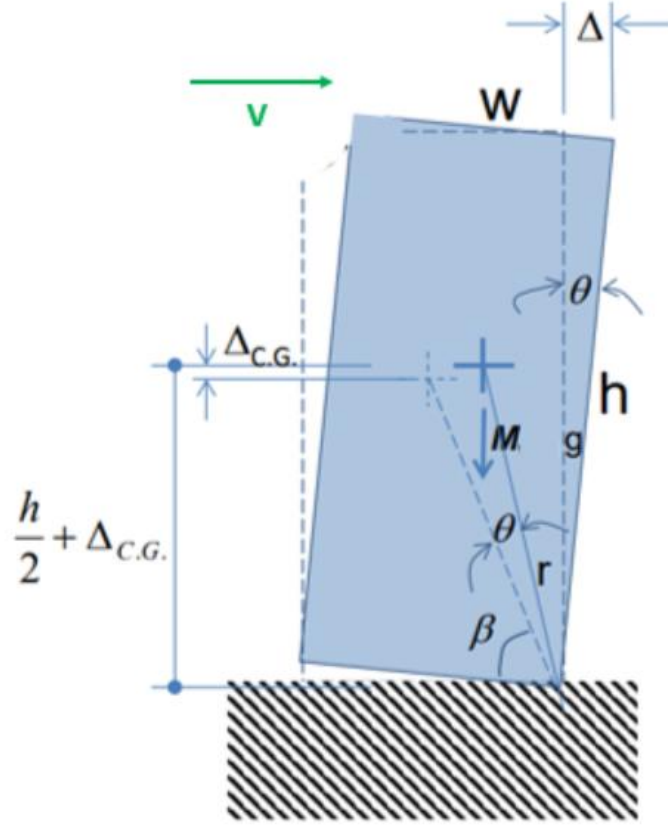


Figure 6-7 Schematic diagram of a rocking rigid body subjected to an initial velocity

Considering the energy balance:

*Kinetic energy = Potential energy obtained due
to the lift in mass of the structure at the centre of gravity (COG)*

$$\frac{1}{2}MV^2 = Mg\Delta_{C.G.} \quad \text{Eq 6-1}$$

$$V = \sqrt{2g\Delta_{C.G.}} \quad \text{Eq 6-2}$$

$$\theta = \sin^{-1} \left(\frac{\frac{h}{2} + \Delta_{C.G.}}{r} \right) - \beta \quad \text{Eq 6-3}$$

where $2r = \sqrt{h^2 + w^2}$ and $\beta = \tan^{-1} \left(\frac{h}{w} \right)$

For the critical case, where angle of rotation, $\theta = \theta_{Critical}$, the corresponding $\Delta_{C.G.}$ can be obtained using Eq 6-3 and can be substituted in Eq 6-2 to obtain the critical value for velocity, $V = V_{Critical}$. This velocity value can be compared with the initial velocity of the

superstructure component at stage 3 (just after the connections fail) to evaluate whether the structure will overturn.

Considering the rigid body in the current study, the following values are calculated:

$$r = 0.75 \text{ m}, h = 1.47 \text{ m}, W = 0.315 \text{ m}, \beta = 77.9^\circ, \theta_{\text{Critical}} = 90 - \beta = 12^\circ.$$

Then, following the steps outlined above, the V_{Critical} value is calculated as:

$$V_{\text{Critical}} = 572 \text{ mm/s}$$

The remaining task is to compute the original velocity value for the superstructure just after the connections fail. As the time at which all the connections fail is known from the experimental and FE analysis results in section 5.2.3, the velocity-time history of the roof of the superstructure is obtained to extract the velocity corresponding to the time just after all the connections fail. This process is depicted in Figure 6-8.

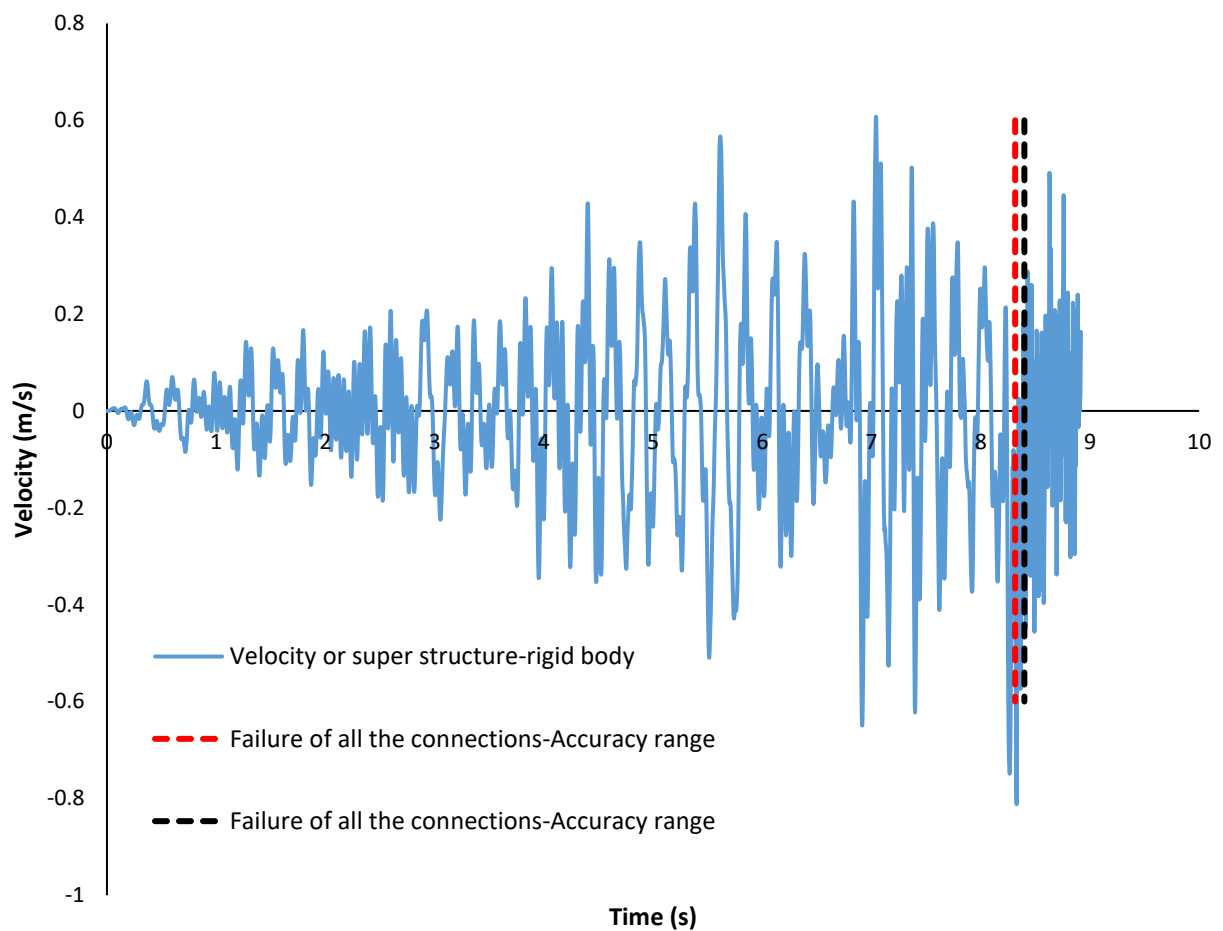


Figure 6-8 Velocity-time history of the rigid body (superstructure)

Figure 6-8 implies that the velocity of the superstructure falls in the range of 600–700 mm/s just after all the connections fail. This serves as an indication that the structure will overturn because the estimated velocity value exceeds the critical velocity required to overturn the rocking body (i.e., 572 mm/s).

However, the results obtained using this method has two major components not accounted. One is the energy input from the ground motion during the rock and slide phase. The other is the energy that would have been dissipated due to rocking, and friction from the sliding of the free-standing superstructure component.

Hence, it is essential to conduct further analysis that properly considers the structure's observed response during this stage. For this reason, a numerical model of the free-standing body, which can simulate the entire stage 3 collapse behaviour is developed in MATLAB. The numerical model is developed, taking into account the equations of motion governing the rocking, sliding, and overturning of a rigid body.

6.2 Initial Conditions for the Slide and Rock Phase

For all analyses conducted, it is essential to estimate the state of the superstructure component (rigid body) at the beginning of stage 3 (i.e. just after the failure of all the connections when the superstructure becomes free-standing). These values are obtained using the validated FE model of the scaled-down model of the modular building discussed in section 5.2.

The main parameters recorded are the angle of rotation of the rigid body (θ), angular velocity of the rigid body ($\dot{\theta}$), angular acceleration ($\ddot{\theta}$), linear displacement of the Centre of Gravity (COG) in the horizontal direction (X_C), linear horizontal velocity of the COG ($\dot{X}_C$) and linear horizontal acceleration of the COG ($\ddot{X}_C$).

Time histories corresponding to each of these parameters are shown in Figure 6-9 - Figure 6-14. These plots are employed to obtain the initial condition values for stage 3, which are summarised in Table 6-1.

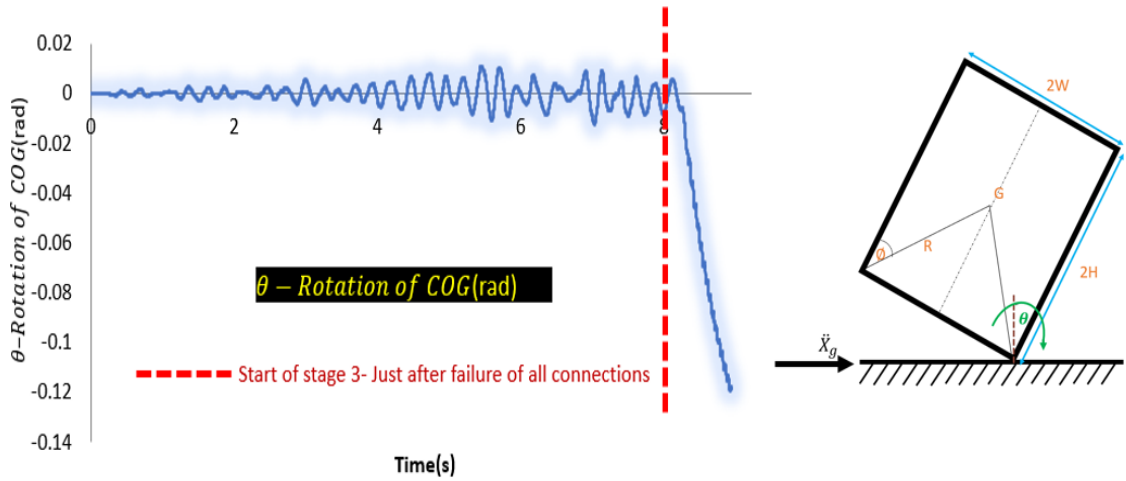


Figure 6-9 Variation in the angle of rotation (θ) of the rigid body with time

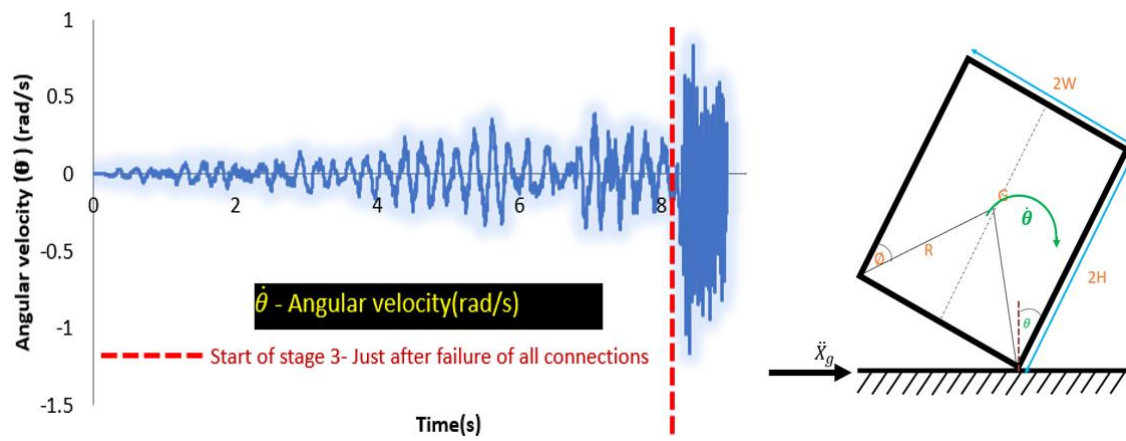


Figure 6-10 Variation in the angular velocity ($\dot{\theta}$) of the rigid body with time

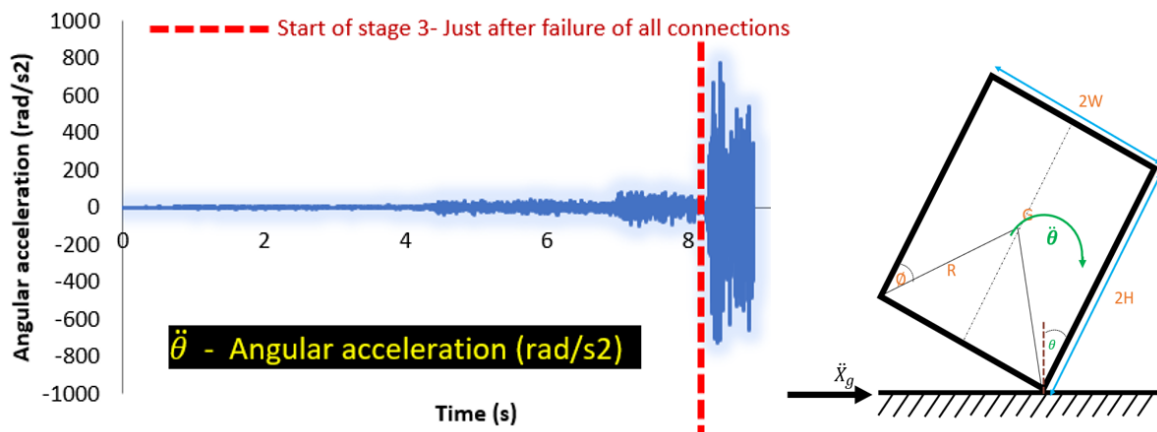


Figure 6-11 Variation in the angular acceleration ($\ddot{\theta}$) with time

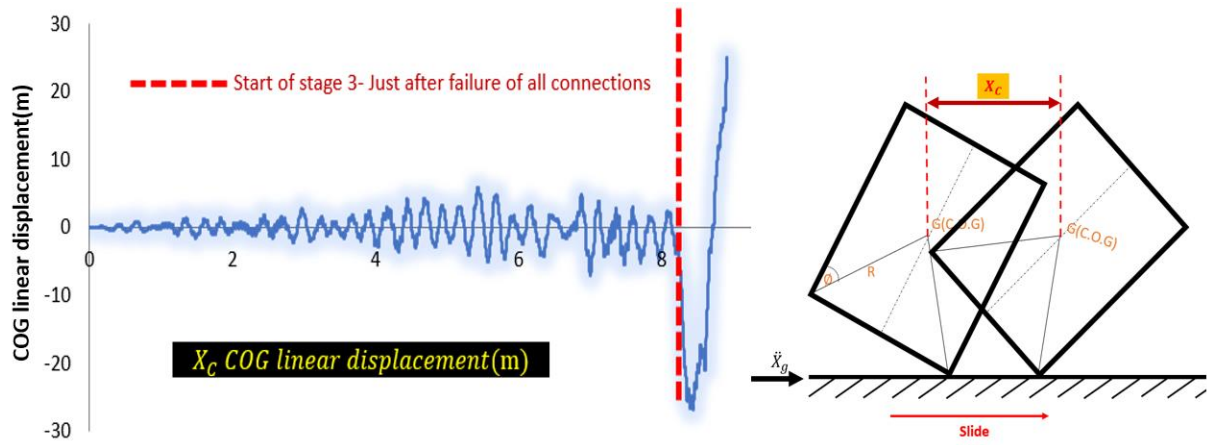


Figure 6-12 Variation in COG linear displacement (X_C) with time

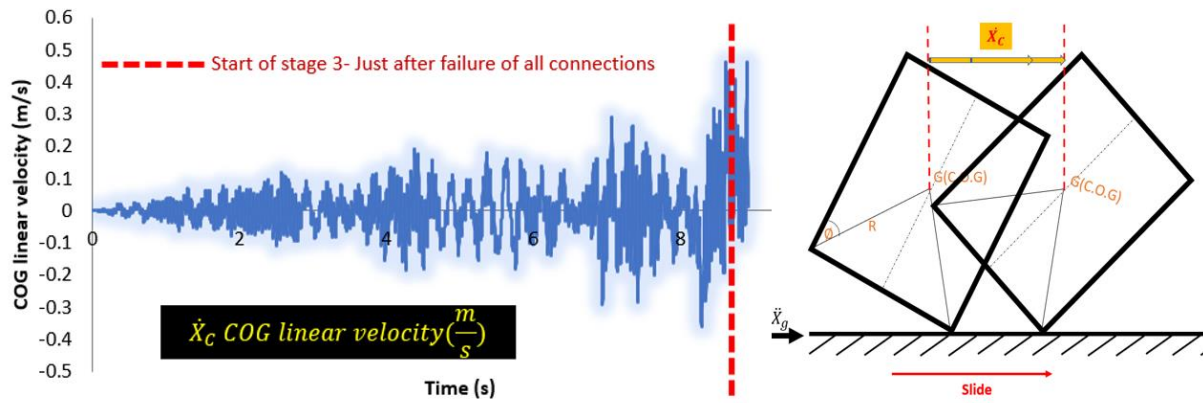


Figure 6-13 Variation in COG linear velocity variation ($\dot{X}_C$) with time

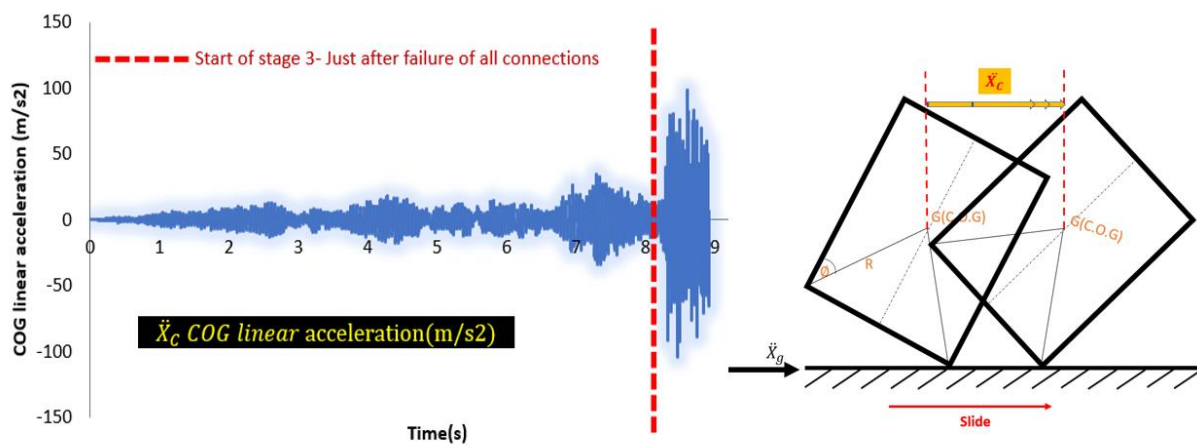


Figure 6-14 Variation in COG linear acceleration variation ($\ddot{X}_C$) with time

Table 6-1 Estimated parameter values for initial conditions (from Figure 6-9 - Figure 6-14)

Parameter	Value
θ	0.0222 rad
$\dot{\theta}$	0.6494 rad/s
$\ddot{\theta}$	81.136 rad/s ²
X_c	0.0197 m
$\dot{X}_c$	0.22081 m/s
$\ddot{X}_c$	-10.3399 m/s ²

6.3 Numerical Simulations of the Sliding and Overturning Motions

This section describes numerical modelling of the rocking, sliding, and overturning observed in the modular building superstructure part, using the equations of motions for a rocking, and sliding body. One of the main assumptions here is that the stack of modules comprising the superstructure part behaves as a rigid body, as also shown in Figure 6-3.

6.3.1 Equations for Dynamic Equilibrium

Figure 6-15 depicts the equilibrium position for a rigid body subjected to sliding and rocking under a ground motion. Equations are derived considering the equilibrium of the rigid body and are solved to obtain the parameters of interest under the time history applied.

Consider moments about ‘G’:

$$-S(\theta)NR\sin(\phi - |\theta|) + fR\cos(\phi - |\theta|) = I_G\ddot{\theta} \quad \text{Eq 6-4}$$

Considering the equilibrium in the horizontal direction:

$$f = m[\ddot{X}_g + \ddot{X}_c - S(\theta)R\dot{\theta}^2\sin(\phi - |\theta|) - R\ddot{\theta}\cos(\phi - \theta)] \quad \text{Eq 6-5}$$

$$f = -S(\dot{X}_c)\mu_k N \quad \text{Eq 6-6}$$

Considering the equilibrium in the vertical direction:

$$N - mg = mR[S(\theta)\ddot{\theta}\sin(\phi - |\theta|) - \dot{\theta}^2\cos(\phi - \theta)] \quad \text{Eq 6-7}$$

For the case of slender, rigid bodies, Eq 6-4, Eq 6-5, Eq 6-6 and Eq 6-7 can be solved to arrive at:

$$-S(\dot{X}_C)\mu_k[g + S(\theta)R \ddot{\theta}\sin(\varnothing - |\theta|)] = \ddot{X}_g + \ddot{X}_C - \ddot{\theta} \quad \text{Eq 6-8}$$

$$\left\{ \frac{I_G}{m} + R^2(\varnothing - |\theta|)^2 + S(\dot{X}_C)S(\theta)\mu_k R^2(\varnothing - |\theta|)\ddot{\theta} + [S(\dot{X}_C) R\mu_k + S(\theta)R(\varnothing - |\theta|)]g \right\} = 0 \quad \text{Eq 6-9}$$

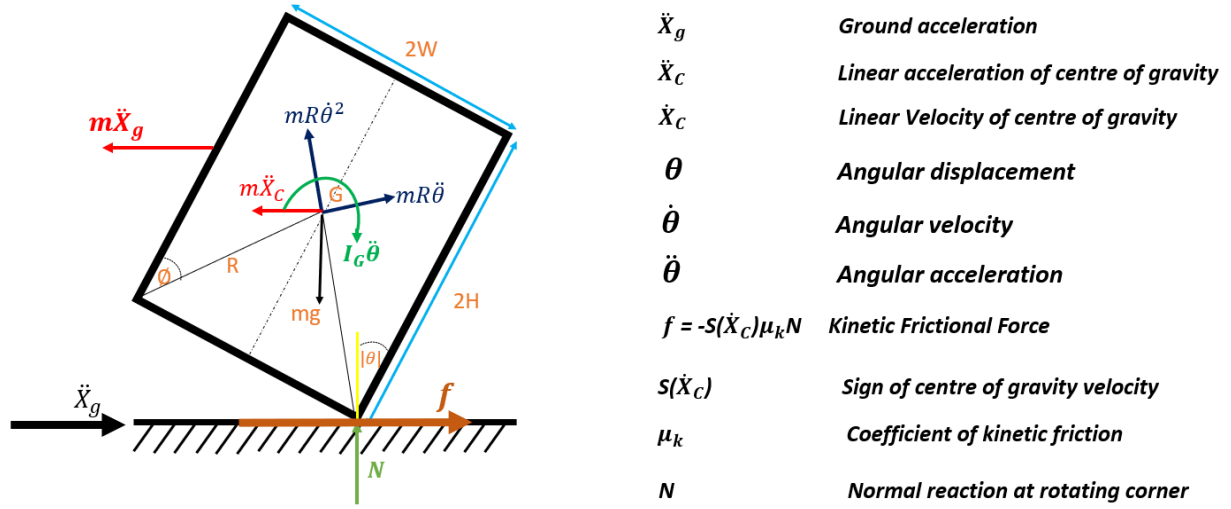


Figure 6-15 A rigid body subjected to sliding and rocking

When a rigid block experiences rocking, it pivots around one corner at a time, then switches to the corresponding corner, which causes an impact between the block and its foundation. This requires taking into account the changed velocity after impact. The new velocity can be computed using Eq 6-10 shown below,

$$\dot{\theta}_{i+1} = r\dot{\theta}_i \quad \text{Eq 6-10}$$

where r is the coefficient of restitution, which can be calculated using the principle of conservation of momentum:

$$r = (1 - \frac{3}{2}\sin(\varphi)^2) \quad \text{Eq 6-11}$$

The main equations governing the dynamics, Eq 6-8 and Eq 6-9, are solved using a MATLAB program that uses the Range–Kutte 4th-order differential equation solving method to arrive at solutions for each time step under the time history. The program is set to monitor the angle of rotation (θ) at each time step, and if θ is found to pass zero (impact occurs and rotating pivot

changes), the velocity components are modified as per Eq 6-10 to take into account the change in velocity due to the impact of pivot points. The primary dimensions chosen for the rigid block (dimensions of the superstructure part of the modular building) in this study are summarised in Figure 6-16.

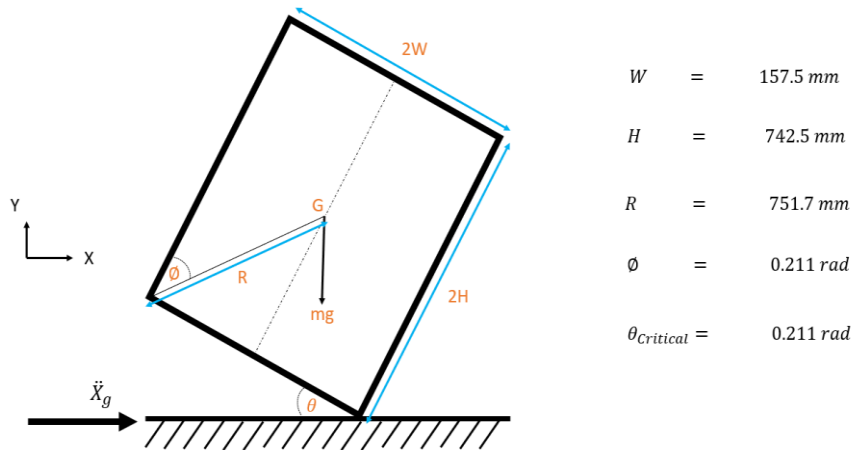


Figure 6-16 Major dimensions and parameters of the rigid body

The ground motion employed is the same ETEF as discussed in Section 4.3.3.1, but the full ground motion is not applied. Instead, the ground motion range corresponding to the free-standing phase, which is the latter part of the ETEF just after the failure of all the connections, is chosen. This is to reflect the real scenario the structure experiences just after all the connections fail. The simulation results are discussed in the following sections.

6.3.2 Angle of Rotation

The plot in Figure 6-17 can be easily understood with reference to the snapshots (see Figure 6-18) obtained from the shaking table experiment and the video of the rock and slide phase observed during the experiment on the shaking table, available at: <https://youtu.be/nOe0c0niu58>.

The rigid body begins with initial conditions at point A, just after all the connections fail, slides while rotating and reaches a peak angle of rotation in that direction of sliding, at point B. After point B, the rigid body starts rotating in the opposite direction. At point C, it reaches a zero angle of rotation phase where an impact of the pivot point, which is on air, occurs, and this point becomes the new pivot point. The rigid body starts rotating about this pivot point while crossing point D, which is the critical angle for overturning. Since the rigid body angle of rotation exceeds the critical angle for overturning, the rigid body entirely overturns, leading to a collapse.

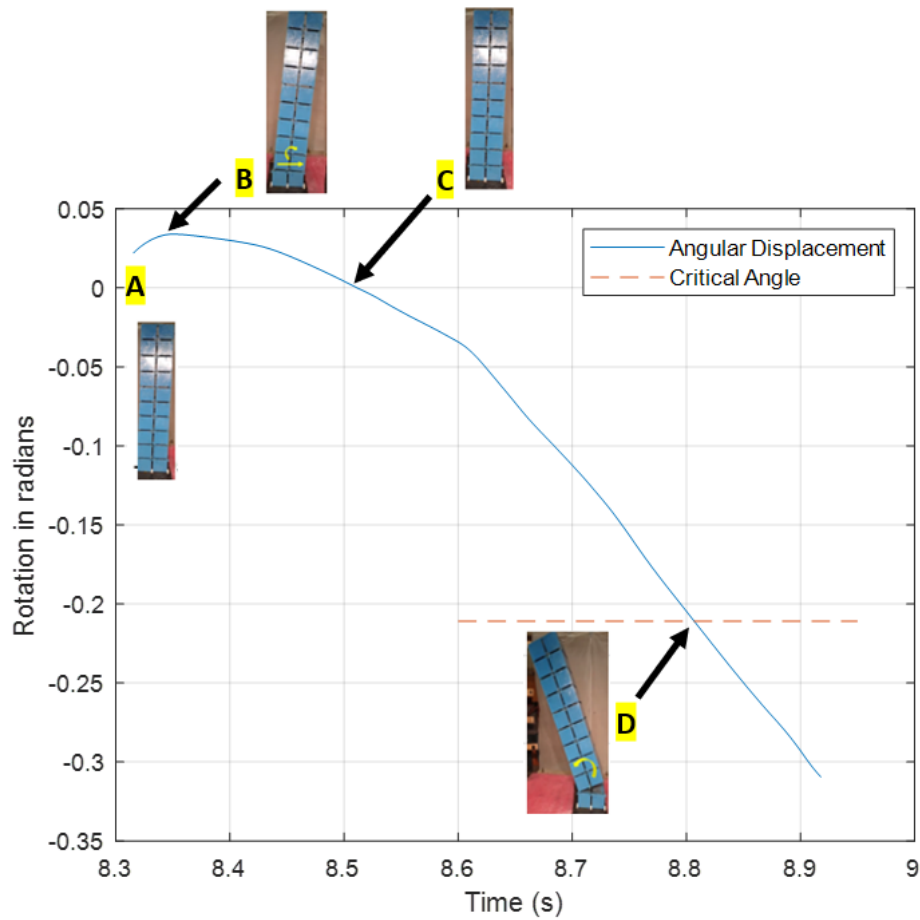


Figure 6-17 Variation in the angle of rotation with time

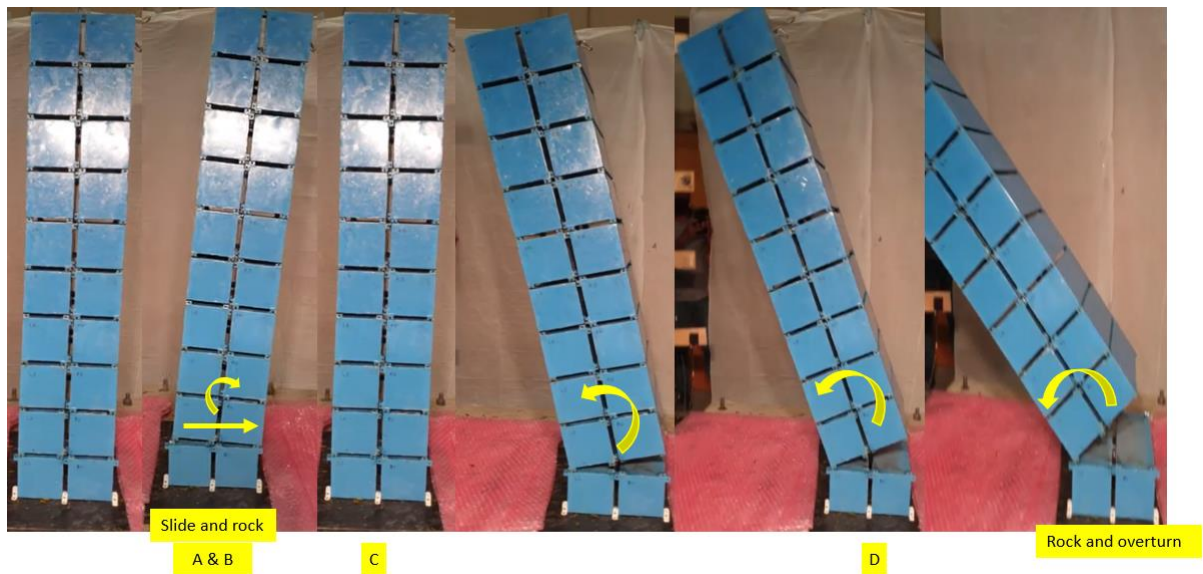


Figure 6-18 Snapshots of observations corresponding to the graphical representation in Figure 6-17

The experimental observations match very well with the numerical simulations. However, this does not necessarily validate the numerical MATLAB model. A free-standing rigid body was

thus subjected to the same ground motion on the shaking table under section 4.4. The same analysis is repeated on the numerical model developed in this section. This validation process is presented under section 6.4.1.

6.3.3 Sliding Displacement

Figure 6-19 illustrates the variation in sliding displacement with time for the rigid body subjected to ground motion. It was observed during the shaking table experiment that as soon as all the connections failed, the superstructure component slid in one direction. This sliding occurred while also rotating in that direction until a peak rotation angle was achieved. After this, rotation switched to the other direction to overturn without any sliding. Precisely representing these observations, the plot in Figure 6-19 depicts a sudden increase in sliding displacement from position A to position B (Positions A and B refer to Figure 6-18). Once the structure is at position B, the sliding stops, and the structure rocks without any sliding. The flat line in Figure 6-19 reflects the no-sliding phase.

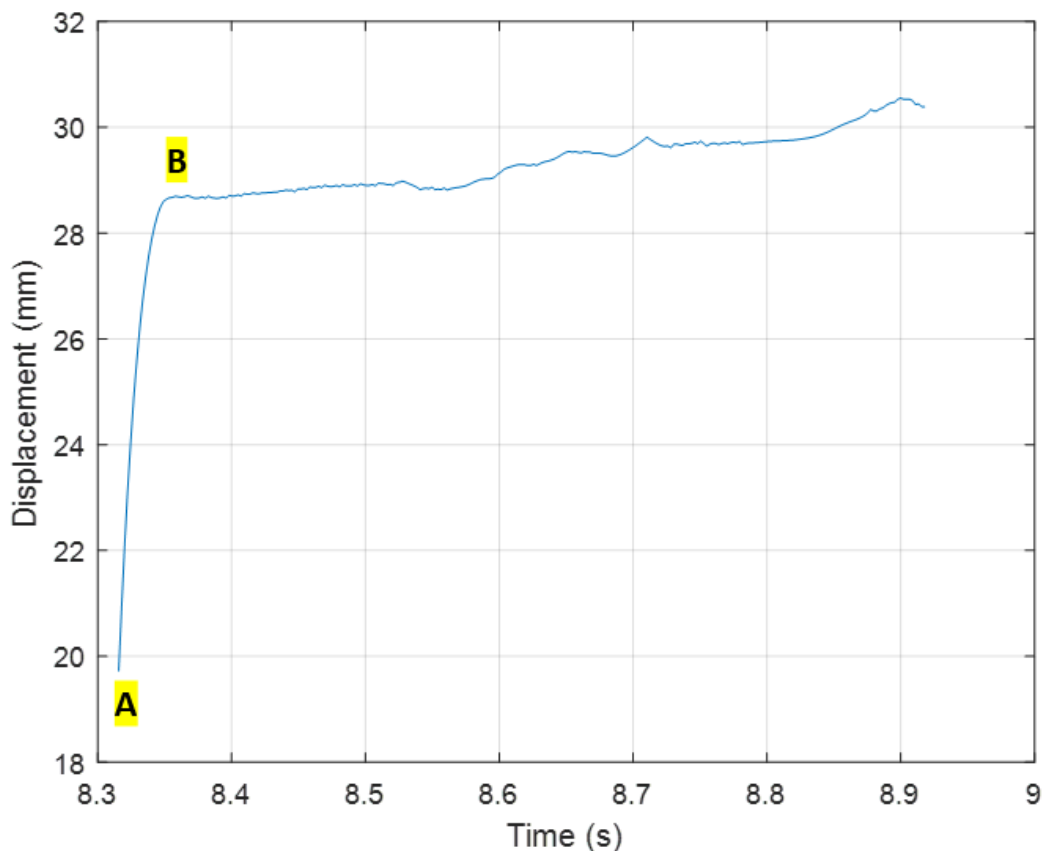


Figure 6-19 Variation of sliding displacement of the rigid body with time

6.3.4 Sliding Velocity

Figure 6-20 illustrates the variation in sliding velocity with time. As explained in Section 6.3.3, the sliding starts from the beginning and ends as soon as it reaches position B. Accordingly, the rigid body that slides starts with a higher velocity and then slows down to become stationary in terms of linear sliding (zero velocity) as depicted in Figure 6-20.

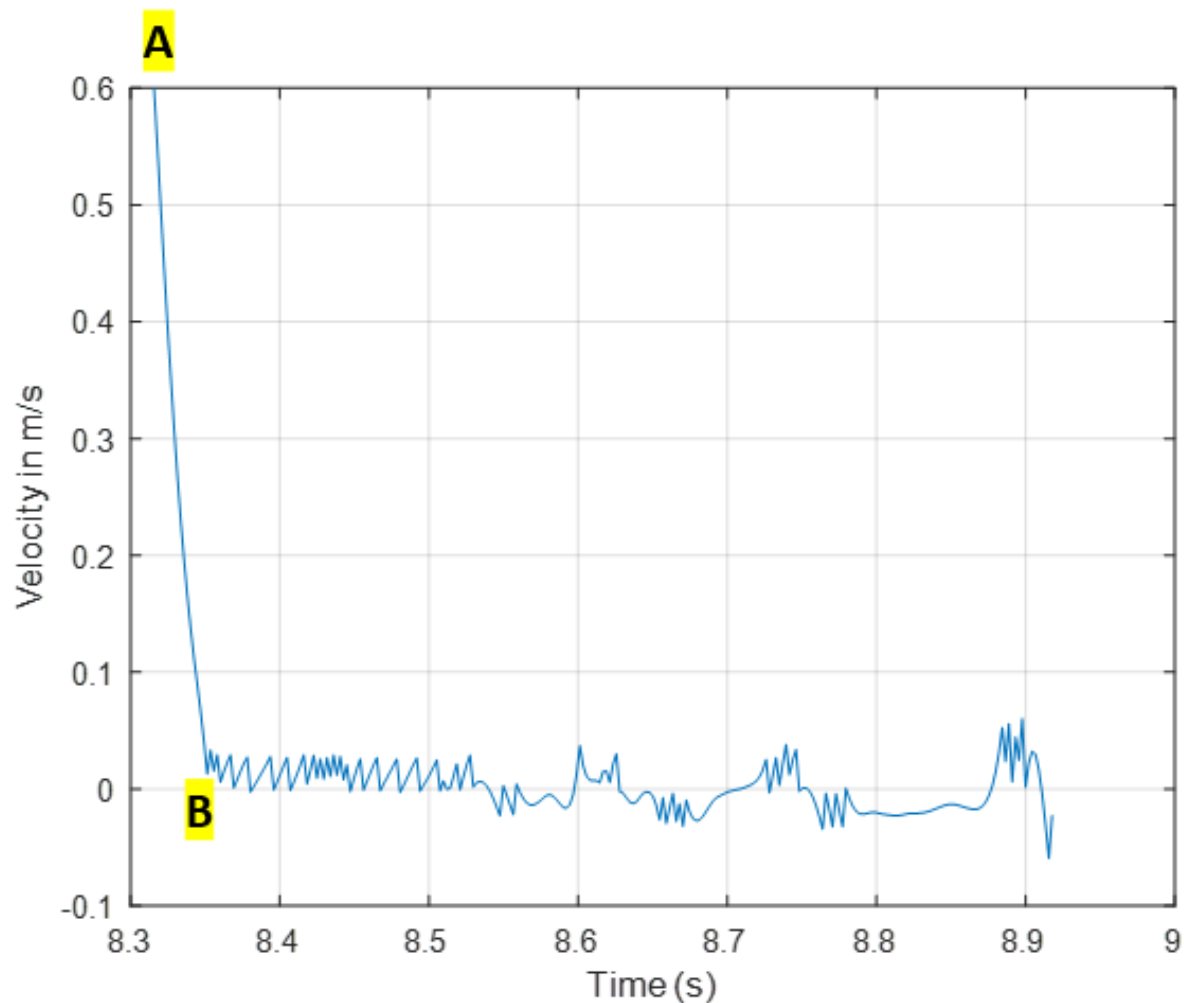


Figure 6-20 Variation in the sliding velocity of the rigid body with time

6.4 Connection Failure Triggering Overturning

As discussed in Section 6.1.2, one of the main conditions required for overturning, that the ground displacement exceeds the half-width of the rocking object, is not satisfied in this study. However, the structure experiences overturning under the ground motion under consideration.

As the overturning cannot be related to ground motion displacement, the study extends on that in Section 6.1.2.2 to evaluate whether the strain energy released from the failure of all connections contribute to the overturning of the superstructure component.

The study results based on energy balance provide evidence in support of this hypothesis. However, the energy approach incorporated is not sufficiently accurate to base on the results. Nonetheless, the shaking table experiment results presented under section 4.4 are in support of the conclusions made based on the energy approach. The shaking table experiments conducted under section 4.4 was on a control model exactly representing the numerical model of the free-standing superstructure component developed under section 6.3.

Hence, in this section of the study, the numerical model developed in section 6.3 is employed to repeat the same analysis that was conducted under section 4.4 on the shaking table; that is to study the response of the numerical model of the free-standing rigid body under the ground motion stretch corresponding to stage 3 (as per Figure 6-1)(ground motion corresponding to the rock, slide and overturn phase), but, without considering the initial conditions gained from stage 2 (as per Figure 6-1)(due to failure of connections).

6.4.1 Numerical Analysis of a Free-standing Model of the Superstructure

This section of the study involves simulating the shaking table observations (section 4.4) using the numerical model developed in Section 6.3 (MATLAB model of the superstructure component sliding and rocking). The original code developed is suitable for any condition that the structure undergoes.

However, this part of the study aims to simulate the response of the superstructure component when there are no initial conditions for stage 3; that is, starting from rest. Thus, the initial conditions gained by the superstructure through failure of the connections that occur just before the start of free-standing motion are set to zero. The same ground motion, as applied during the shaking table experiment in Section 4.4, is applied to the numerical model in MATLAB.

Figure 6-21 illustrates a comparison of the numerical and experimental results (obtained under section 4.4) obtained. Figure 6-22 is similar to Figure 6-21, except it includes more of the free oscillation of the free-standing body as recorded on the shaking table. The results from the shaking table experiment and numerical simulation are identical.

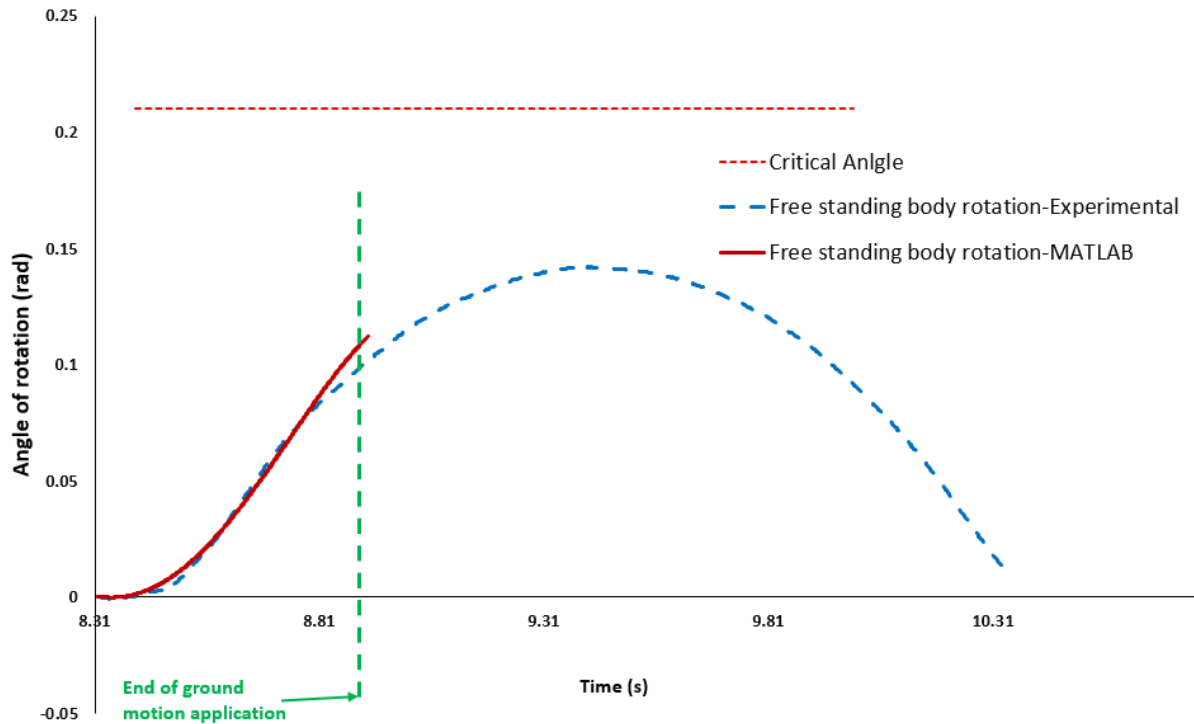


Figure 6-21 Angle of rotation obtained using experimental and numerical analyses

Based on these results, it can be concluded that there is a substantial effect on overturning from the initial conditions gained by the superstructure component at the beginning of the free-standing motion. These initial conditions, which came into the free-standing body in the form of initial velocity, were gained from the strain energy released from the failure of the connections. Further, it was highlighted under section 5.2.3.3 that all these connections failed in a matter of no time, which is a lot of energy released in a very short time. This sudden release of large energy can be related to the overturning of the free-standing superstructure.

This indicates that the energy required for the overturning of the superstructure component is gained mostly from the strain energy released from the failure of the connections, which elevates the kinetic energy of the superstructure.

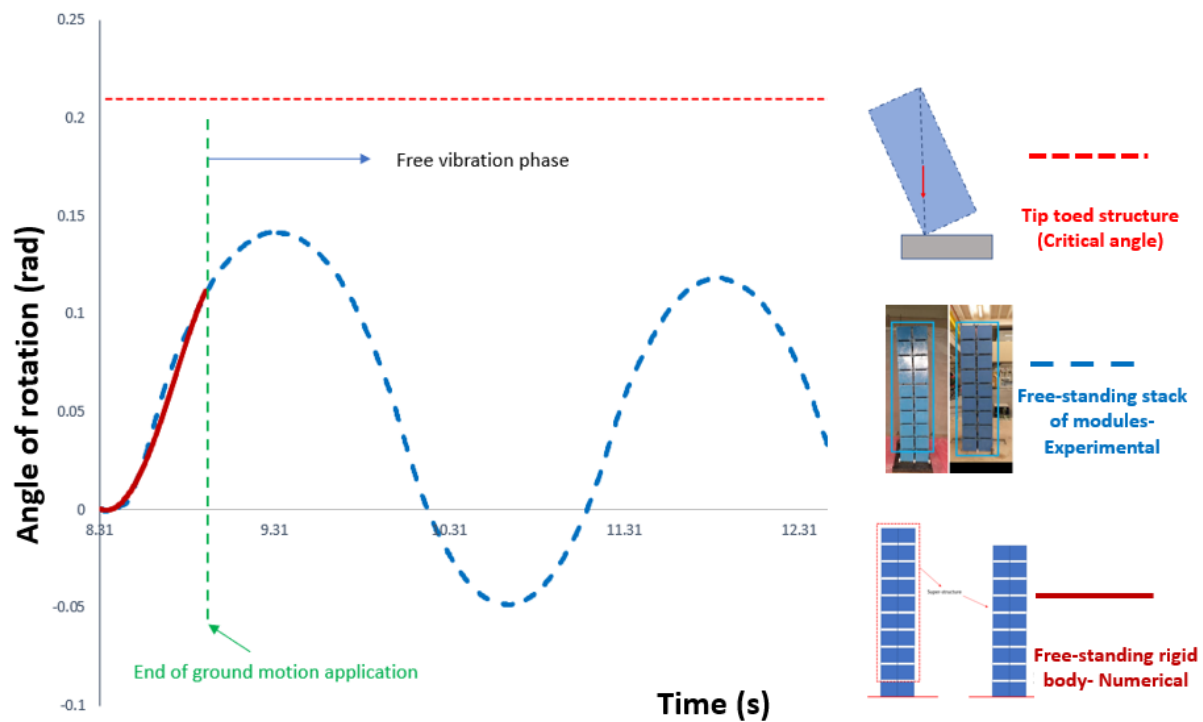


Figure 6-22 Angle of rotation obtained using experimental and numerical analyses (including free oscillation)

Figure 6-23 duplicates Figure 6-22 but is superimposed on Figure 6-17 for the case where the free-standing body started with the initial conditions gained through failure of the connections. Hence, Figure 6-23 compares the variation in the angle of rotation of a rigid body starting from rest (experimental and numerical results) with that starting with the initial conditions (the rock and slide case, which led to overturning).

This comparison conveys a more precise impression of the effect on overturning of the initial conditions gained through failure of the connections. A video comparing this observed response with the original modular building collapse can be viewed at: https://youtu.be/uc-XLpri1_0.

Aside from this main conclusion, the excellent match between the experimental and numerical results serves as a validation of the numerical model developed. Hence, the numerical model developed in MATLAB can be utilised to conduct further studies on the overturning of the superstructure component of the modular building.

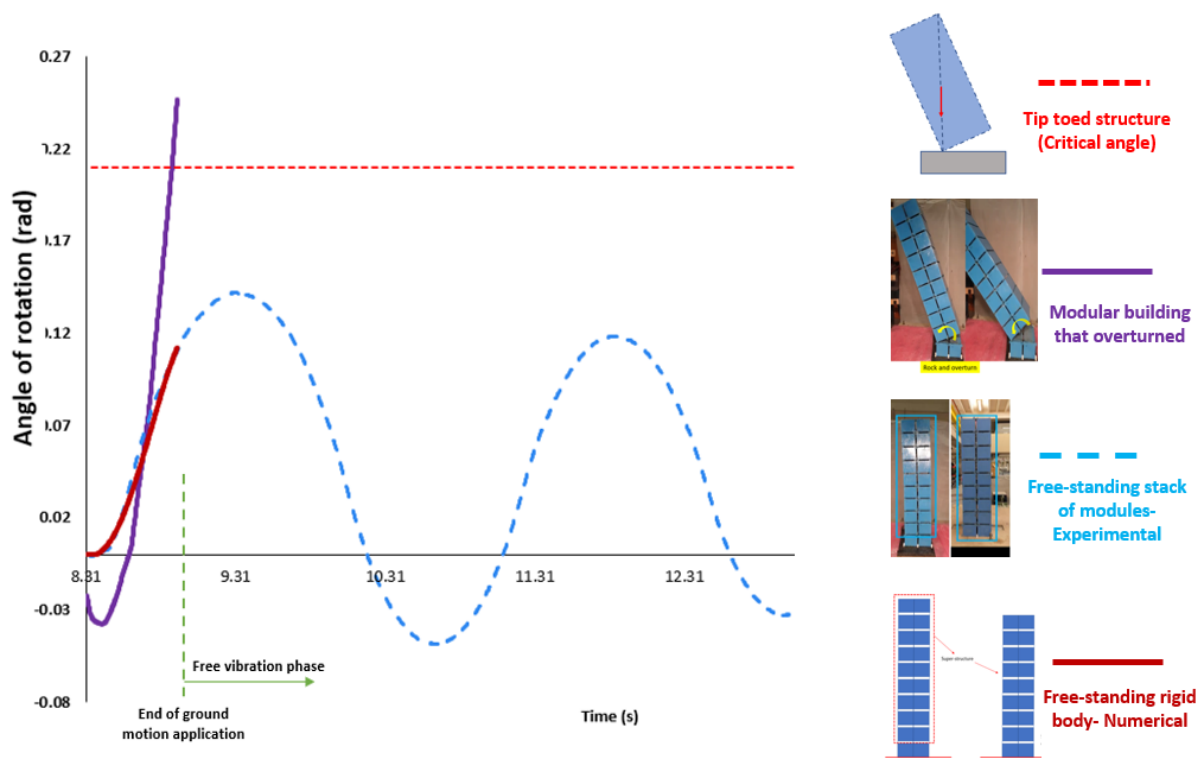


Figure 6-23 Angle of rotation of the free-standing body with and without initial conditions.

6.4.2 Effect of the Strength of the Connections on Overturning

It is concluded from the results in Section 4.4 and 6.4.1 that the sudden failure of connections results in the availability of excess energy for the superstructure part of the modular building and causes it to experience overturning. Thus, the amount of energy released can be related to the strength of the connection. For this reason, this study is further extended to investigate the effect of reduced connection strength on the overturning.

An analysis is conducted on the numerical model of the modular building developed in section 5.1.1, with the connection strength halved. The results from this model under the ETEF are employed to extract the initial conditions for the superstructure part (similar to Section 6.2), as soon as all the connections fail. These initial conditions are fed into the MATLAB model to study the rock and slide motion of the rigid body (superstructure). The resulting variation in the angle of rotation of the superstructure, compared with that of the structure that had the original connection strength, is shown in Figure 6-24. The structure with weaker connections does not overturn. This can be attributed to the lower strain energy that these weaker connections would release, compared with the building with original connection strength.

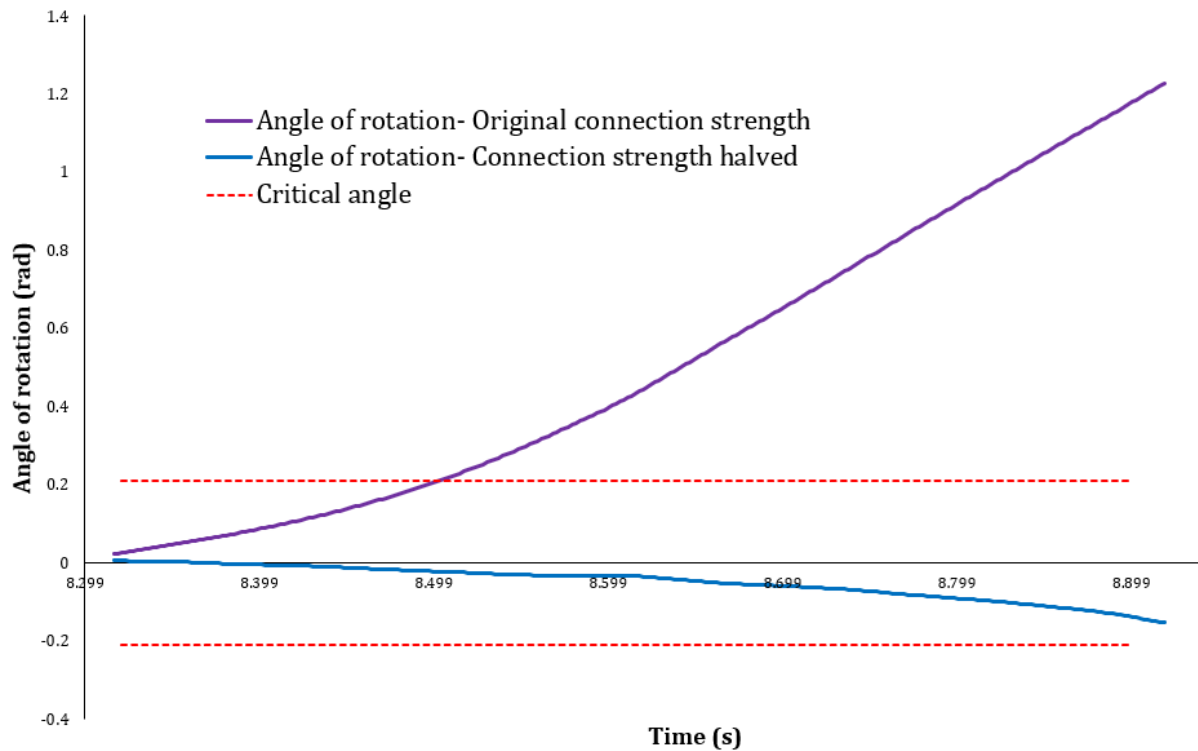


Figure 6-24 Effect of connection strength on the angle of rotation of free-standing body

6.5 Implications of Sliding Motion to Risk of Overturning

Sliding of the superstructure component after failure is not common. It is only possible in a modular structure because of the unique nature of failure involved in such buildings. This failure involves a definite failure surface, which may allow the separated superstructure component to slide while rocking on the substructure.

However, there may be situations in which the superstructure component does not undergo any sliding, counter to what was observed during the shaking table experiment. The sliding may be restricted by any roughness created between the superstructure and the surface of the substructure by the failure of connections.

This section examines the response of the free-standing superstructure under this sort of a scenario to provide an idea about the contribution of sliding to overturning; that is, whether it exacerbates or mitigates overturning.

6.5.1 Behaviour of a Rigid Body With and Without Accounting for Sliding

As discussed above, to study the effect of sliding on overturning, another MATLAB code is developed without taking into account the sliding the structure experiences. For this, the

parameters linear slide displacement (X_C), linear slide velocity ($\dot{X}_C$) and linear sliding acceleration ($\ddot{X}_C$) are set to zero to simulate a no-sliding condition. The angle of rotation of the rigid body thus obtained is compared with the angle of rotation of the rigid body when the sliding is taken into account, in Figure 6-25.

It is evident from Figure 6-25 that the rigid body that is not allowed to slide reaches the critical angle for overturning much sooner than the rigid body that is allowed to slide while rocking.

When sliding is allowed while rocking, some of the energy is dissipated through friction, leaving less energy for the rocking phase. Hence, the structure that is not allowed to slide at all has a higher energy capacity to be used for the rocking phase, resulting in early overturning.

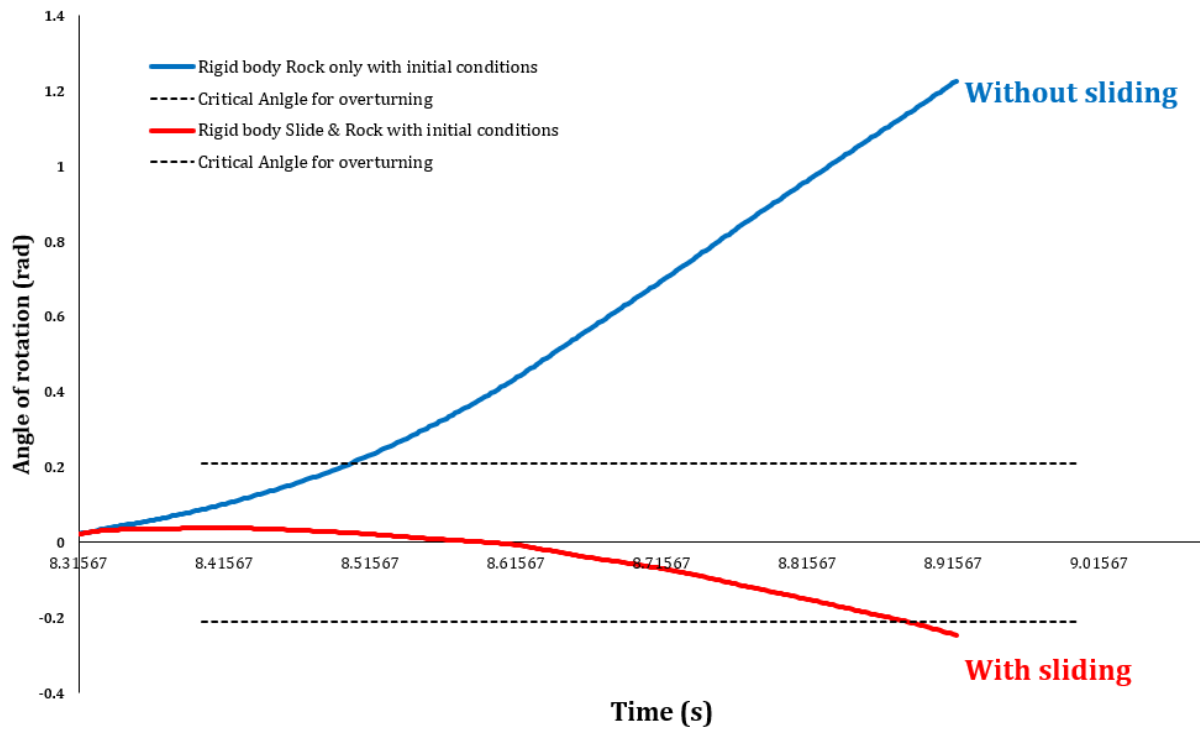


Figure 6-25 Response of free-standing body with and without accounting for sliding

6.5.2 Effect of Friction on Overturning Potential

In Section 6.5.1, it is clear that sliding is beneficial in terms of delaying overturning, and leads to overturning that just manages to exceed the critical angle of rotation for overturning. This triggers the requirement for a parametric study of the effect of variation in sliding capability on overturning. The ability to slide is governed by the coefficient of friction between the superstructure and substructure. This part of the study varies the coefficient of friction to assess variation in the response of the rigid body (superstructure) under the ground motion.

The results in Figure 6-26 show that with a decrease in the coefficient of friction (decrease in friction leading to increased sliding capability), the overturning potential reduces. When the coefficient of friction increases, it becomes closer to the no-sliding case discussed in Section 6.5.1. With more sliding capability, much of the energy is dissipated through friction, leaving substantially less energy to be used for rocking, and hence less energy left for overturning.

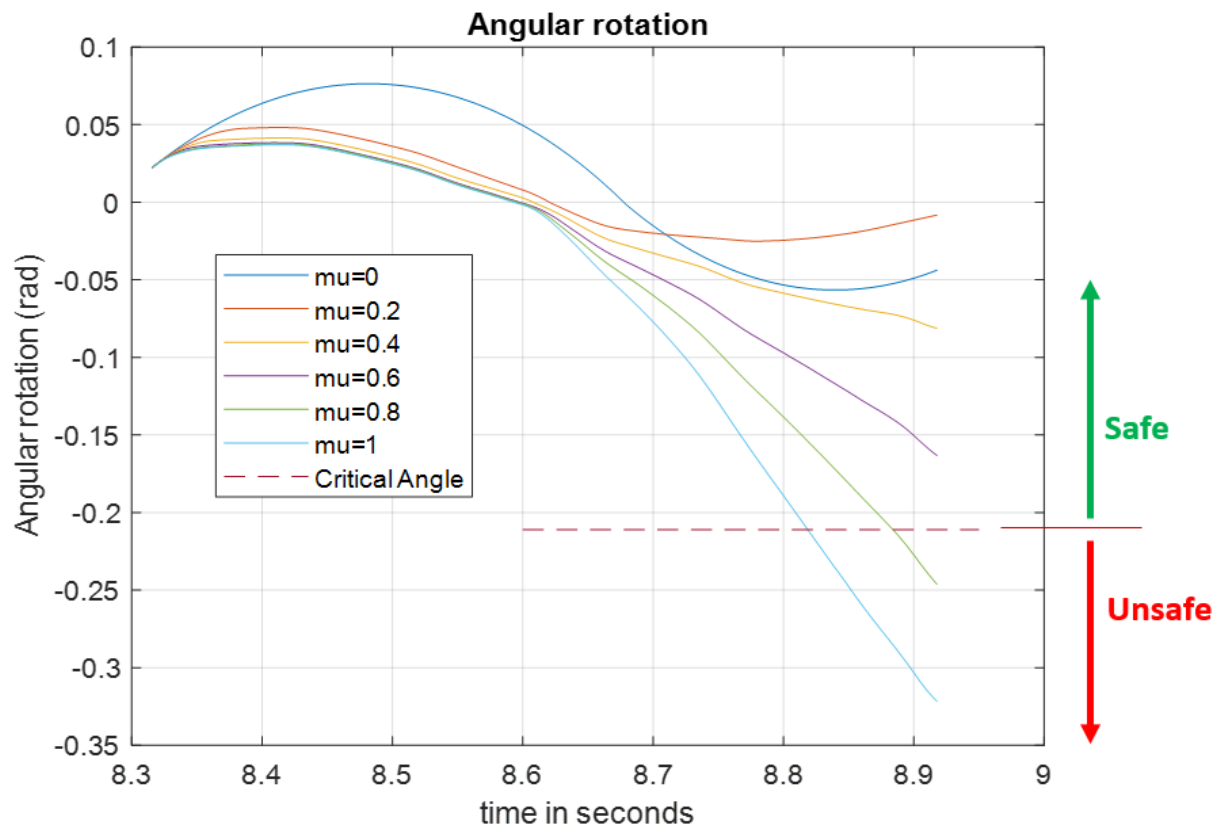


Figure 6-26 Variation of overturning potential with the coefficient of friction

6.5.3 Limitations of the Effect of Sliding on Overturning Potential

Despite the benefits from sliding in terms of reducing overturning potential, having a superstructure continue to slide along the slide plane may not always be possible. The nature of the slide plane constrains continuous sliding.

Figure 6-27 presents a simplified representation of the free-standing superstructure component on the base-fixed substructure component of the modular building, as observed during the shaking table experiment. The free-standing superstructure component, which is considered a rigid body, forms by separation from the first storey of the modular building. During the shaking table experiment discussed in Section 4.3.3, rocking and sliding of the superstructure component took place on the first storey modules, which were firmly fixed to the ground.

Hence, the first storey acted as the sliding and rocking surface for the rigid body, in contrast to the general case of a rigid body sliding and rocking on a surface of infinite length.

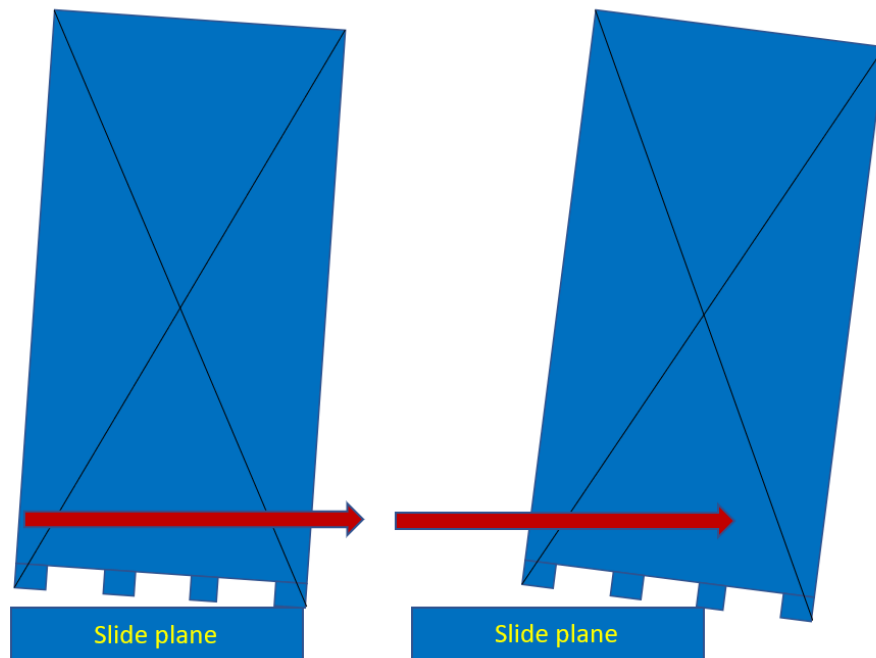


Figure 6-27 Limited slide plane available for superstructure sliding

The danger involved with this sort of a limited-length slide plane is that the superstructure may slide out of the slide plane and prevent one of the pivot points from touching the slide plane. If this sort of sliding happens excessively, it might easily overturn the rigid body due to moving out of the sliding plane.

Further, because of the sliding out of the plane, the lever arm for the restoring moment may reduce in some instances. This is further explained using sketches, as shown in Figure 6-28 and Figure 6-29. Considering Figure 6-28 and 6-29, it can be seen that there is a reduction in the restoring moment in the case of rocking with sliding, while the overturning moment remains the same in both cases. This reduction in the restoring moment may exacerbate the overturning potential of the superstructure.

However, this might not always be the case; in particular, analysis of the shaking table experiment observations from Section 4.5.3, shows that sliding was favourable. As illustrated in Figure 6-30, the rocking body did not overturn in the direction which it slid. Hence, the pivot for the rocking rigid body was still within the slide plane during the overturning phase.

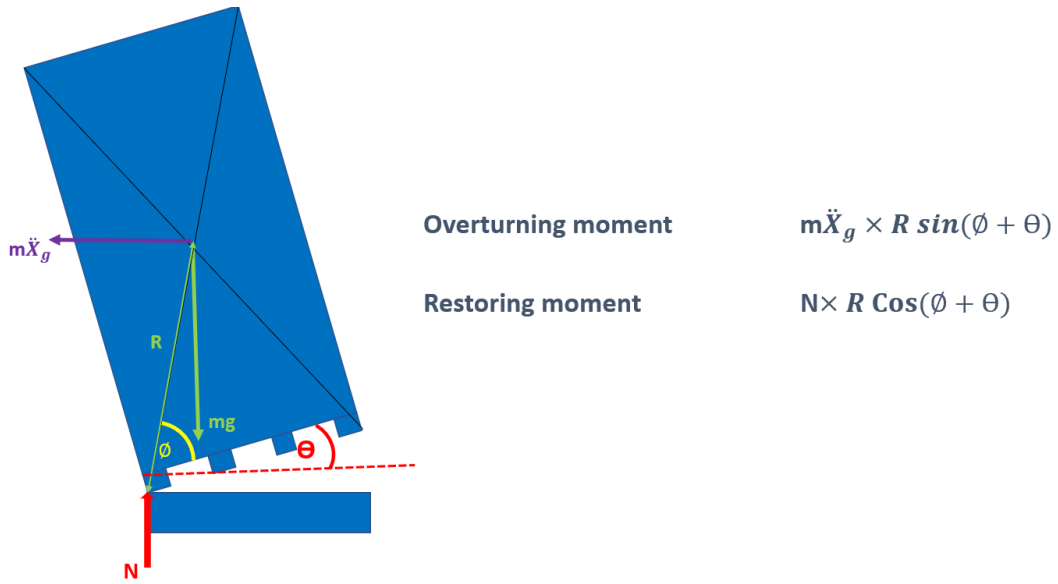


Figure 6-28 Case of rocking without sliding-all pivots inside the slide plane

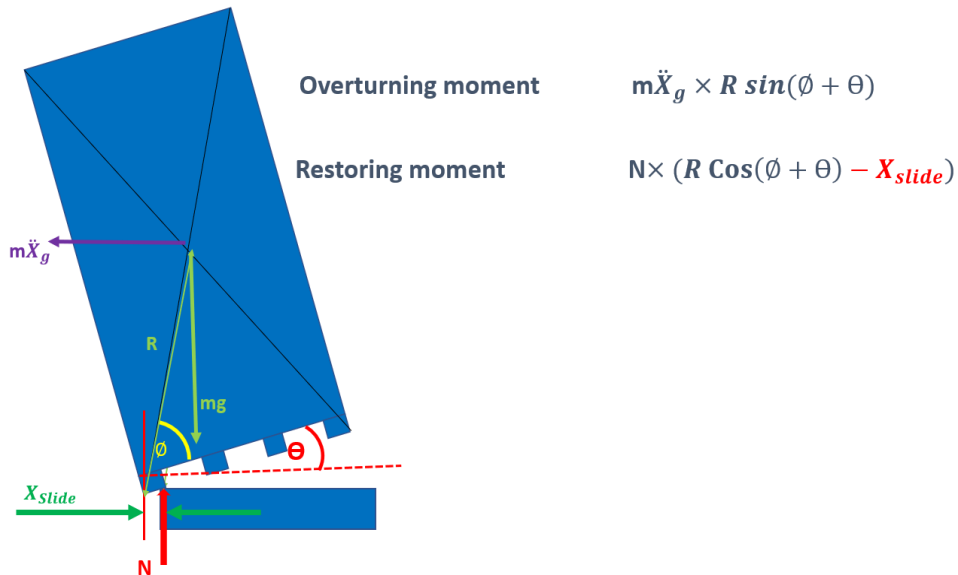


Figure 6-29 Case of rocking with sliding-moving one pivot slightly out of the slide plane

Moreover, at the time of overturning, the pivot had moved more inwards from the end of the slide plane (into the slide plane) (due to sliding; Figure 6-31). Hence, there was no reduction in the lever arm during the shaking table experiment discussed in Section 4.4.3. This implies that the structure in this study experienced no adverse effect from sliding. If the overturning was in the same direction as sliding, the sliding might make a contribution in reducing the restoring moment and causing early overturning. Although this problem was not a concern in the current study, it may be of significant concern in some other structures that might experience different ground motion or other conditions.

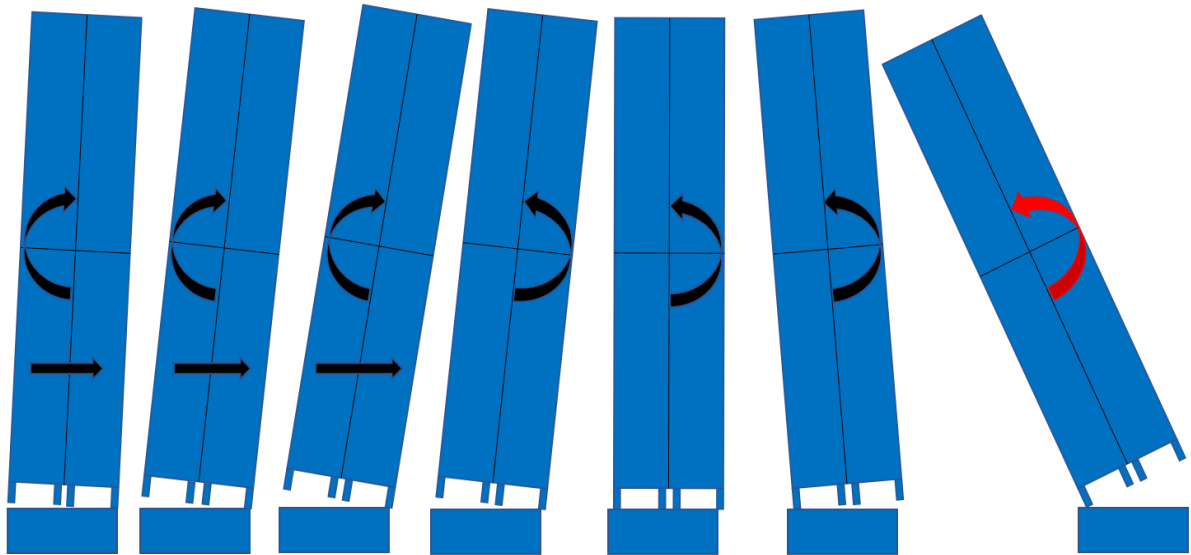


Figure 6-30 Observations during the shaking table experiment discussed in Section 4.4.3

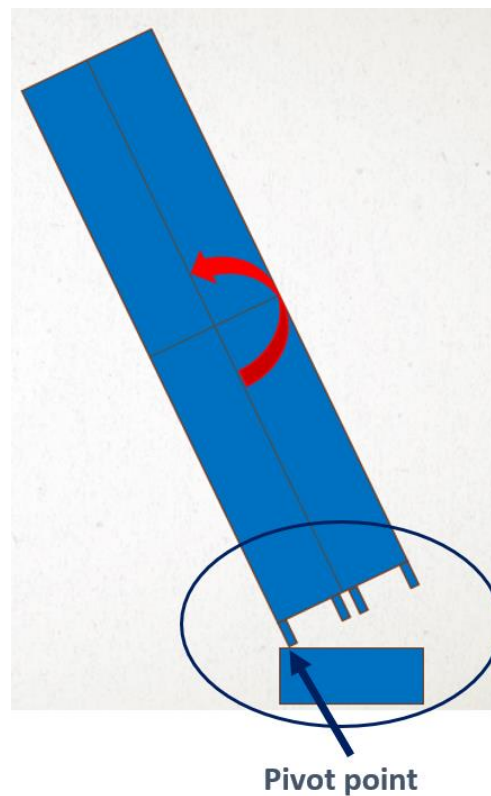


Figure 6-31 Pivot point at the time of overturning

6.5.4 Study on the Effect of a Halt During Sliding

A failure along a plane as observed during the experimental analysis may not always be smooth. It might be sufficiently smooth to allow some sliding. However, it is highly likely that there will be some roughness to the sliding surface, formed as a result of the failure of the connection along the plane, which may disrupt the sliding motion. This may have an adverse effect on the

rotational motion of the rigid body, acting as a sort of tripping and leading to unforeseen overturning. This can be considered a halt during the sliding motion. This section describes quantification of the effect on the overturning of the superstructure of a potential halt during the slide motion.

A parametric study on a rigid body experiencing rocking and sliding under varying coefficient of friction values is presented in Section 6.5.2. That study explores the overturning potential of the free-standing superstructure component under different coefficient of friction values, to check whether they would overturn. For this part of the study, a case corresponding to $\mu = 0.3$ is considered from the study conducted in Section 6.5.2.

It can be seen from Figure 6-26 that this value of μ results in safe performance of the structure in that the rigid body does not experience overturning under the ground motion applied (assuming there is an infinite sliding plane). Hence, this study evaluates whether the occurrence of a halt along the slide path affects the rigid body's motion.

Figure 6-32 illustrates the angle of rotation time history for the case under consideration. Figure 6-33 shows the corresponding linear slide velocity time history of the rigid body. It can be seen from Figure 6-32 that the rigid body rotation angle does not reach the critical angle for overturning. Moreover, Figure 6-33 indicates that the rigid body slides for some distance and then stops sliding to transform into a pure rocking motion.

In the first part of the analysis, a halt is placed as soon as the sliding begins (at the location as marked in Figure 6-34).

In the MATLAB program used to model the presence of a halt, the governing equations of motions up to the halt are considered based on a slide-rock model. The governing equations of motion just after the halt are modelled for a pure rocking case using the initial conditions for the motion from the parameters recorded at the end of the stage just before the halt.

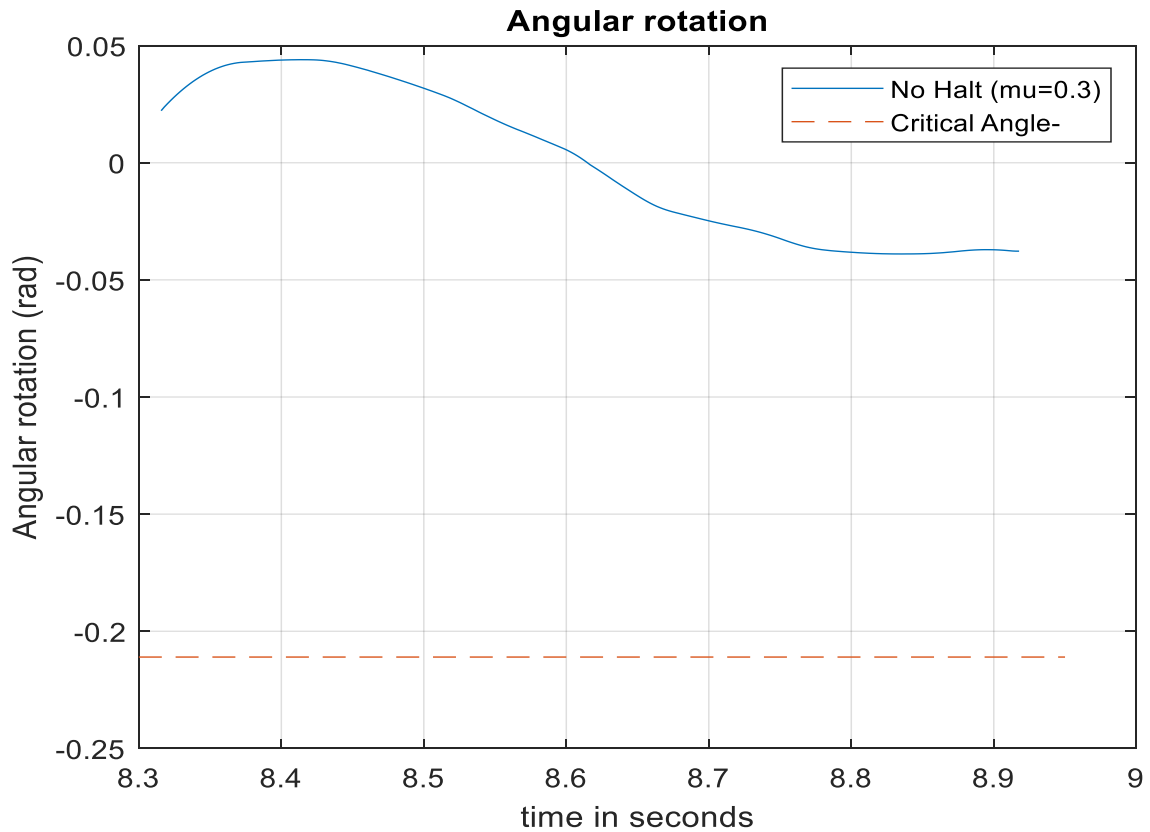


Figure 6-32 Angular rotation time history of the rigid body when $\mu = 0.3$

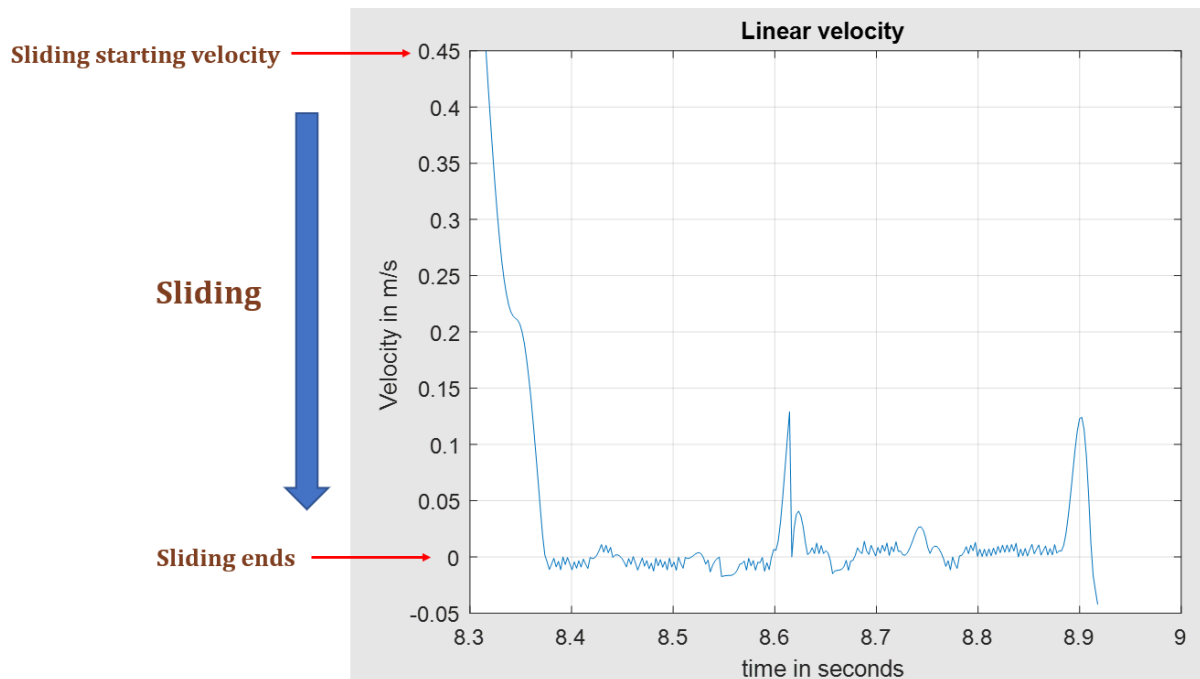


Figure 6-33 Sliding velocity variation corresponding to the $\mu = 0.3$ case

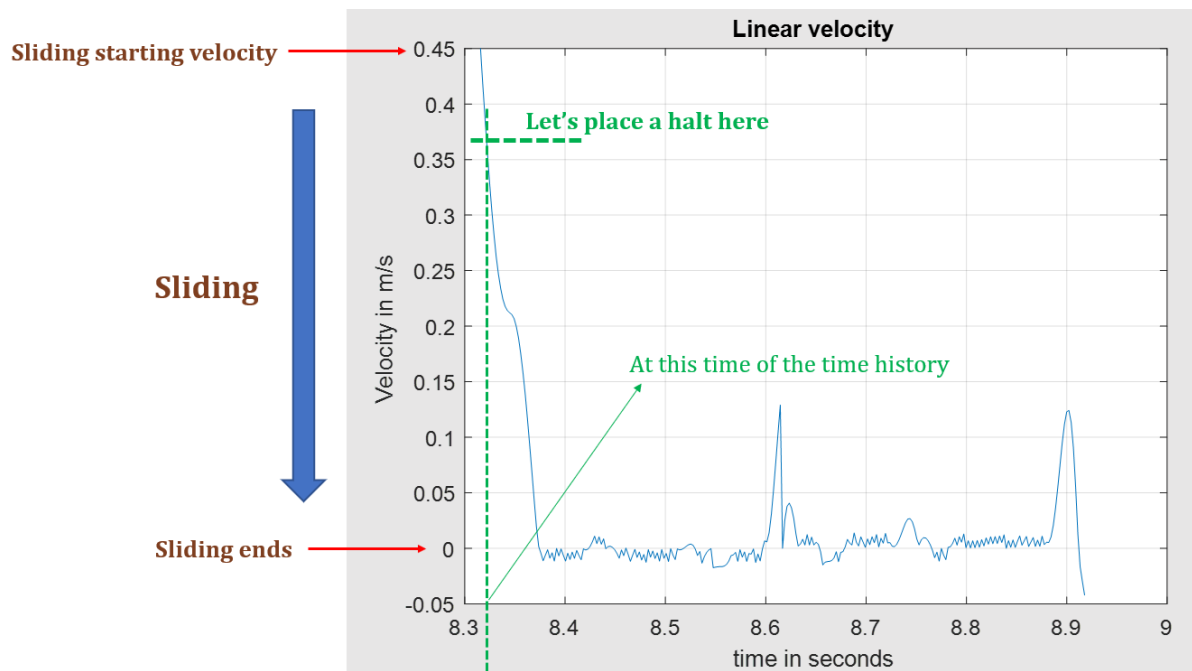


Figure 6-34 First halt location marked in the linear slide velocity time history

Once the halt is inserted, the linear sliding velocity drops to zero at the halt location. The resulting angular rotation time history of the rigid body is illustrated in Figure 6-35, which shows that the structure overturns very easily with less effort. Moreover, this plot is very similar to that in Figure 6-28 for the case of pure rocking. Nevertheless, this scenario is more like a pure rocking case because not much sliding is allowed from the beginning of the motion. Figure 6-36 compares the angle of rotation time history for the rigid body in cases with and without a halt.

Since these results represent a special case that is very similar to a pure rocking case, the study is further extended by adding additional halts at the locations shown in Figure 6-37. The three halts are marked as A, B and C in different locations of the sliding phase in descending order of sliding velocity.

Analysis is conducted for each halt location to evaluate whether overturning has occurred. The final results thus obtained are summarised and compared with a no-halt condition, in the plot shown in Figure 6-38.

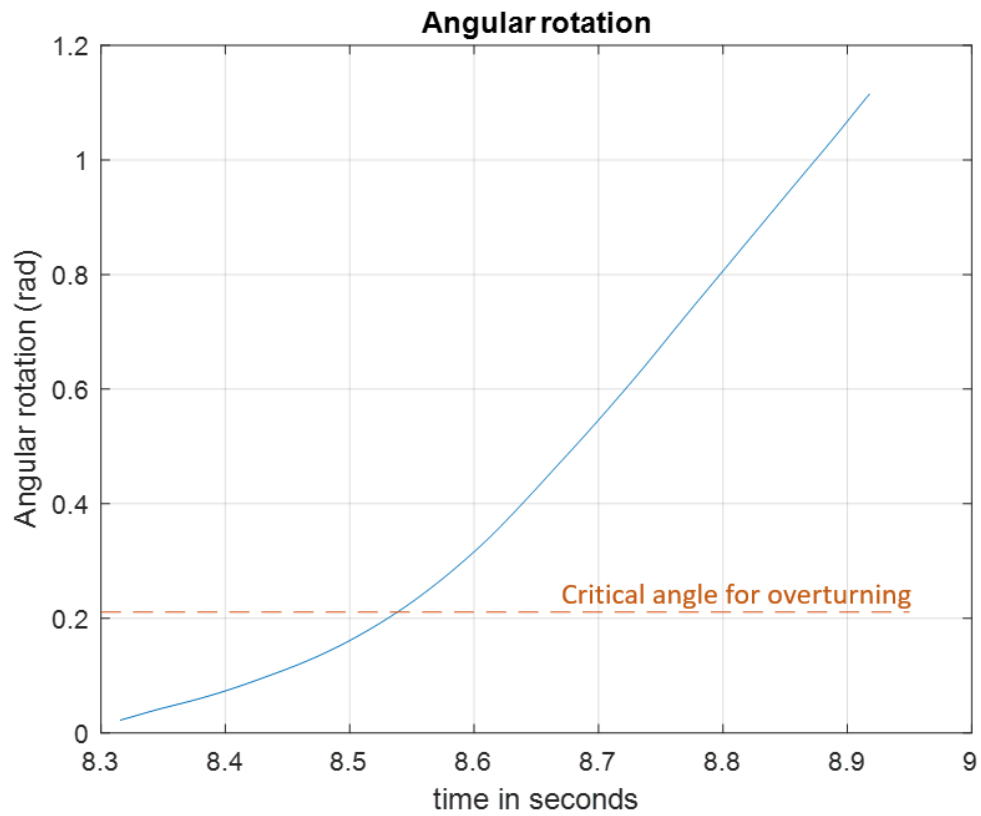


Figure 6-35 Angle of the rotation time history under the effect of first halt

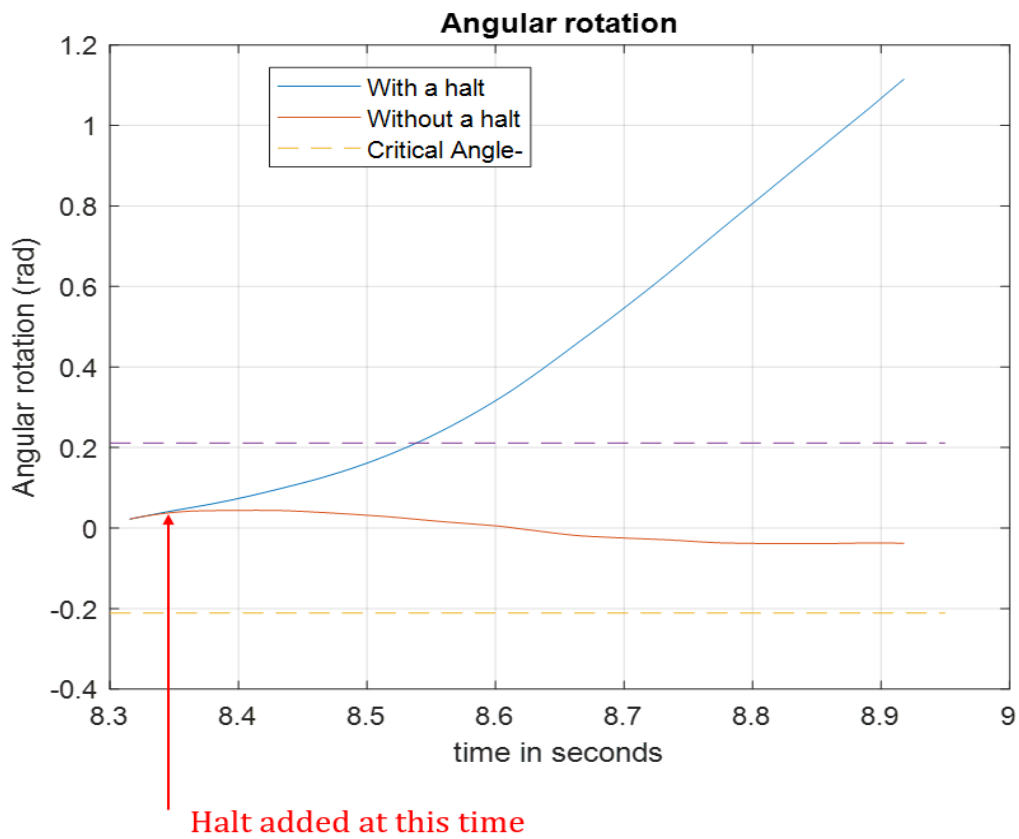


Figure 6-36 Angle of rotation time histories with and without the effect of first halt

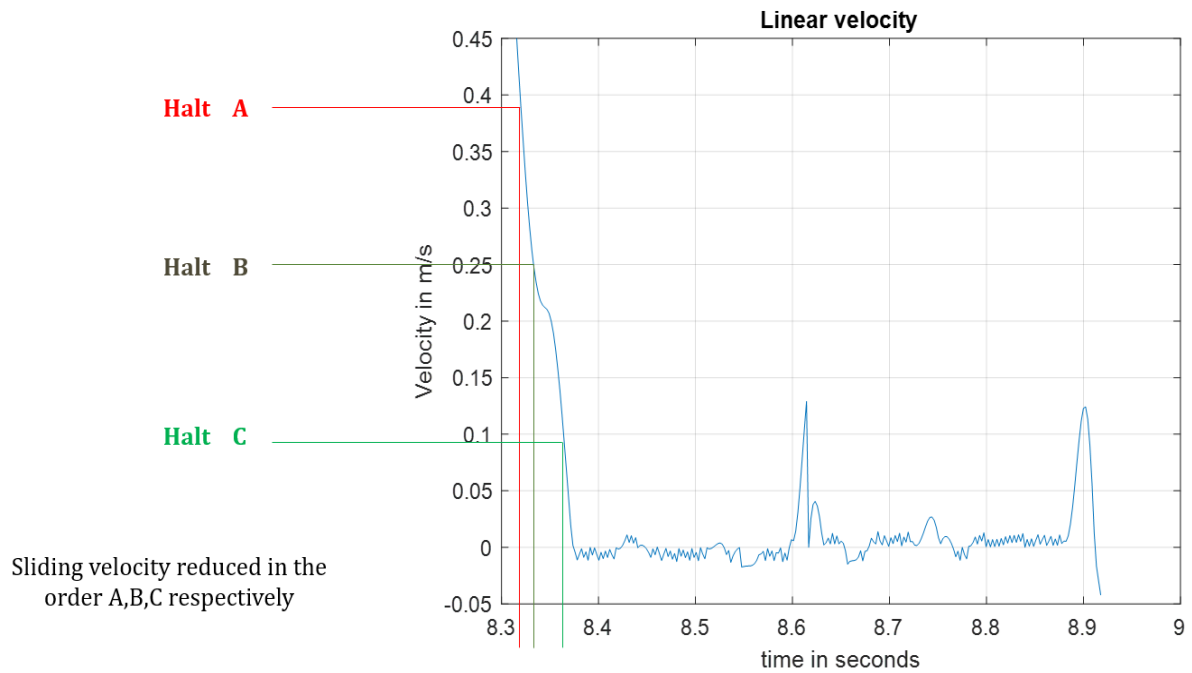


Figure 6-37 Halt locations marked in the velocity time history for the no-halt condition

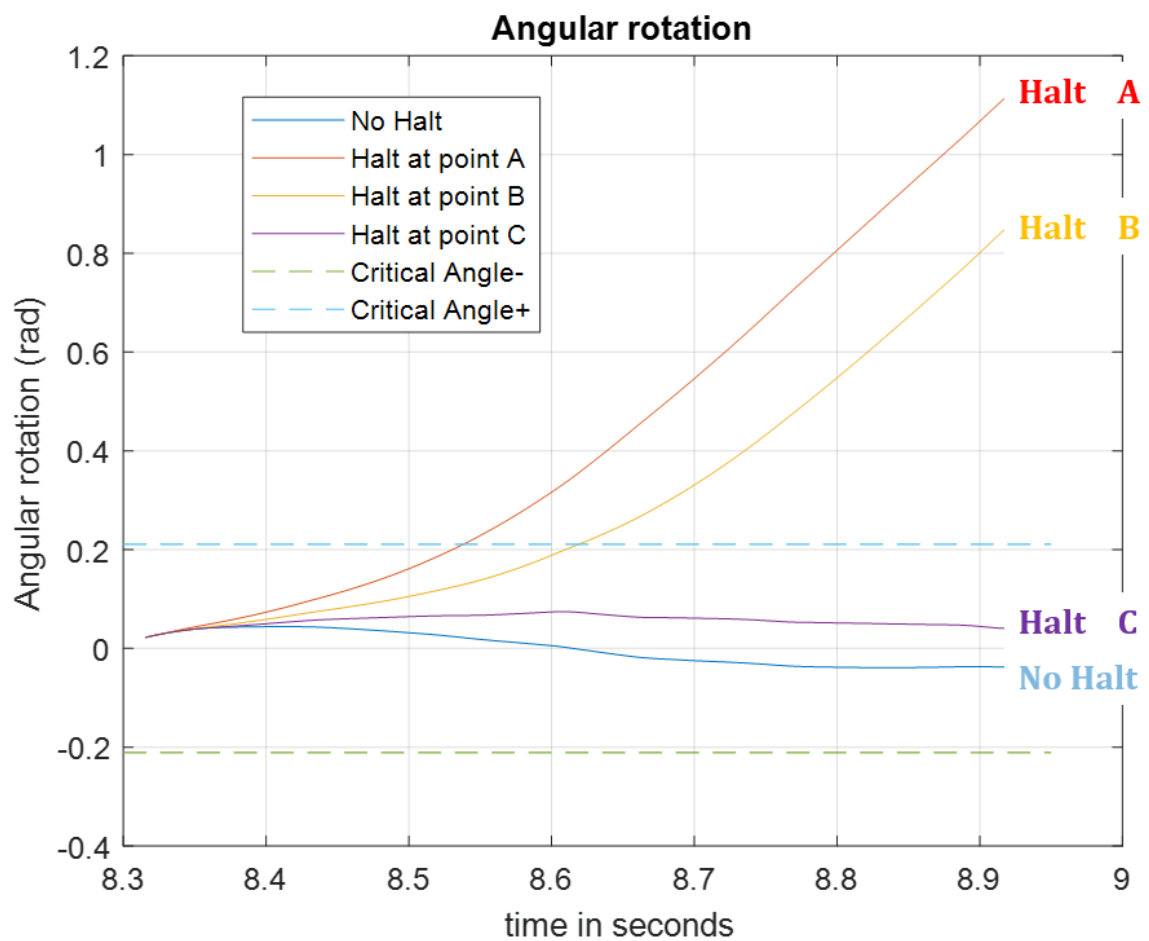


Figure 6-38 Variation in angle of rotation with halt location

Figure 6-38 indicates that halts placed very close to the beginning of the sliding results in exceedance of the critical angle for overturning, while, halt C, which is placed more towards the end of the sliding phase, does not result in overturning. This can be related to energy dissipation through friction. Halt A and B lead to minimum energy being dissipated through friction because they experienced less slide displacement before encountering the halt. Presence of Halt C close to the end of the sliding phase causes more energy to be dissipated through friction, compared with the other two halts. Hence, encountering halt C does not result in sufficient reserved energy to convert to potential energy by overturning.

The placement of halt C results in variation in the angle of rotation, which is similar to the no-halt case because the halt is closer to the point where the structure actually comes to rest due to friction in the case of no halt at all. The placement of halt A and B results in variation in the angle of rotation plots similar to the pure rocking case. This shows that the risk of the structure overturning may be exacerbated by interruptions to its sliding motion, depending on the location of the interruption.

6.6 Effect of Rigidity of the Superstructure on Overturning

This research so far has assumed that the superstructure component of a modular building comprising a stack of nine modules behaves as a rigid body with equivalent dimensions. However, in reality, the stack of modules may not behave like a full rigid body. The inter-modular connections and the modules themselves may deform during rock and slide motion, adding some flexibility to the free-standing superstructure. The flexibility thus added to the structure may or may not exacerbate the risk of overturning of the structure.

This section incorporates into the numerical model the flexibility in the stack of modules forming the superstructure component, as would be present in reality. The modified numerical model is employed to study the rocking response under the particular range of ground motion.

Many approaches are discussed in the literature for modelling the rocking of flexible structures [163-168]. The most common approach used is to model the rocking block as a deformable rocking column characterised by a uniformly distributed mass (m_c) and stiffness (EI). In this study, the approach presented in Ref. [168] is followed in modelling the rocking of the modular building superstructure to account for its flexibility. A detailed derivation of the equation of motion for the rocking motion of a flexible structure is presented in Appendix C.

Preliminary analysis is conducted on the stack of modules to compute required properties such as the stiffness and moment of inertia of a stack of modules idealised as a column section with equivalent computed stiffness and moment of inertia.

A MATLAB program is developed to solve for the equations of motion under each time step of the time history. The ground motion stretch used throughout this chapter is employed here as well (ground motion corresponding to stage 3 of the structure response as per Figure 6-1).

6.6.1 Comparison of the Response of a Free-standing Flexible and Rigid Body

The discussion in Section 6.5 implies that preventing a rigid body from sliding will result in a much earlier overturning of the superstructure component because of the pure rocking nature the structure experiences. Hence, the first part of this study compares the performance of rigid and flexible superstructures undergoing pure rocking and considering the initial values gained through failure of connections at the beginning of free-standing motion. The comparison plot is shown in Figure 6-39.

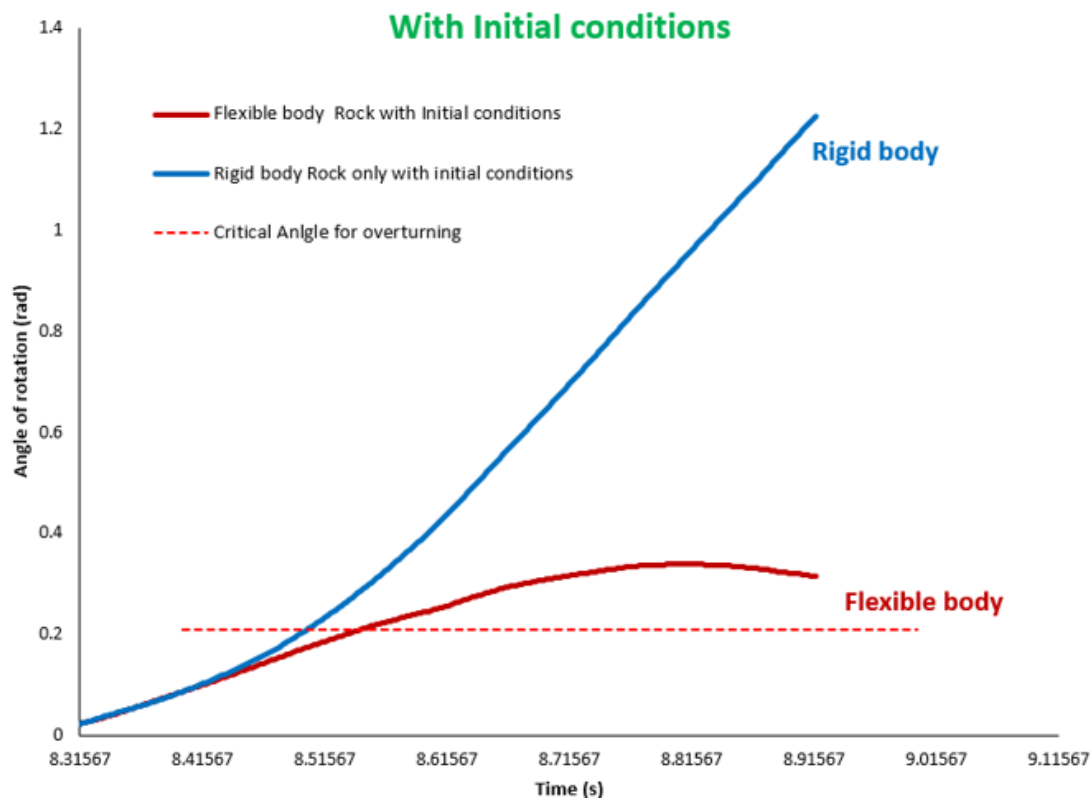


Figure 6-39 Effect of superstructure rigidity on rocking response (with initial conditions)

Figure 6-39 shows that accounting for flexibility of the superstructure is advantageous in terms of mitigating overturning to a certain extent. This is because some of the energy gained from the failure of the connections is dissipated through the flexing of the superstructure component during rocking.

Nonetheless, the flexibility incorporated model exceeds the critical angle for overturning, leading to collapse. This indicates that there is no harm in ignoring the flexibility of the superstructure component, as done in this study.

A parametric study is also conducted varying the rigidity values of the superstructure component to understanding the trend in variation in the angle of rotation with time.

Figure 6-40 presents the variation in the angle of rotation curves under different rigidity conditions. It can be seen that an increase in rigidity of the superstructure component makes it more vulnerable to overturning.

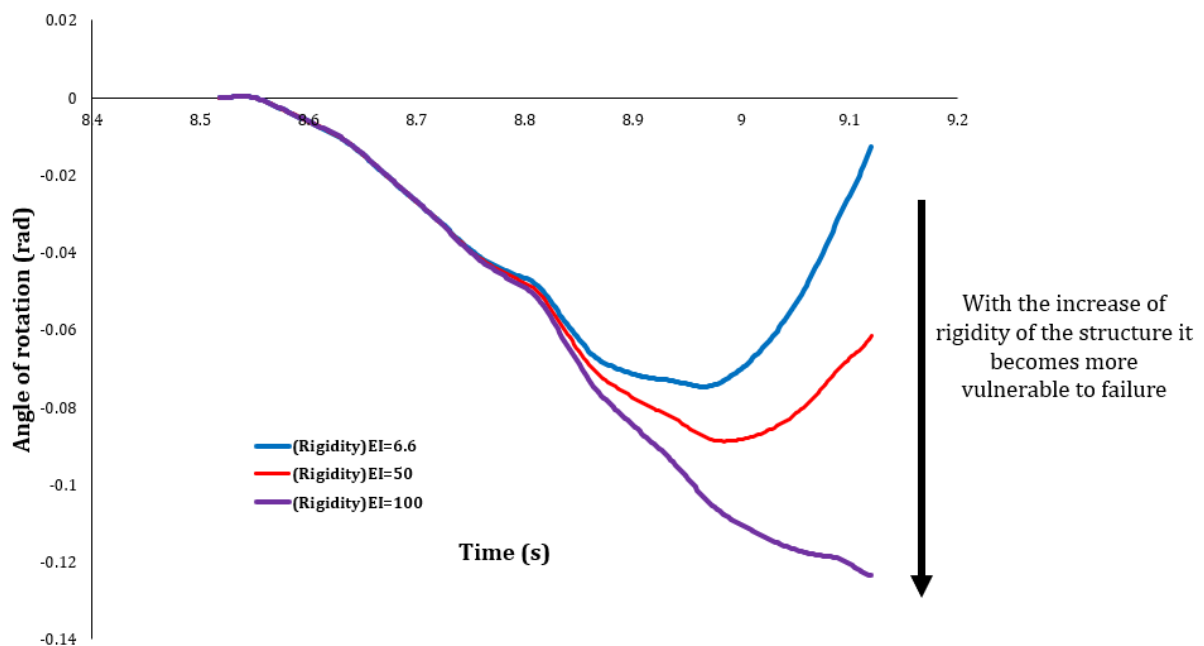


Figure 6-40 Variation of rocking response with superstructure rigidity

Figure 6-41 compares the variation in the angle of rotation obtained using the MATLAB model for a fully rigid superstructure not accounting for initial conditions (Section 6.4.1), and that when a higher rigidity value is applied in the model developed for a flexible superstructure body in this part of the study. These results resemble a closer variation, serving as a validation

of the MATLAB model developed to simulate the response of flexible bodies undergoing rocking.

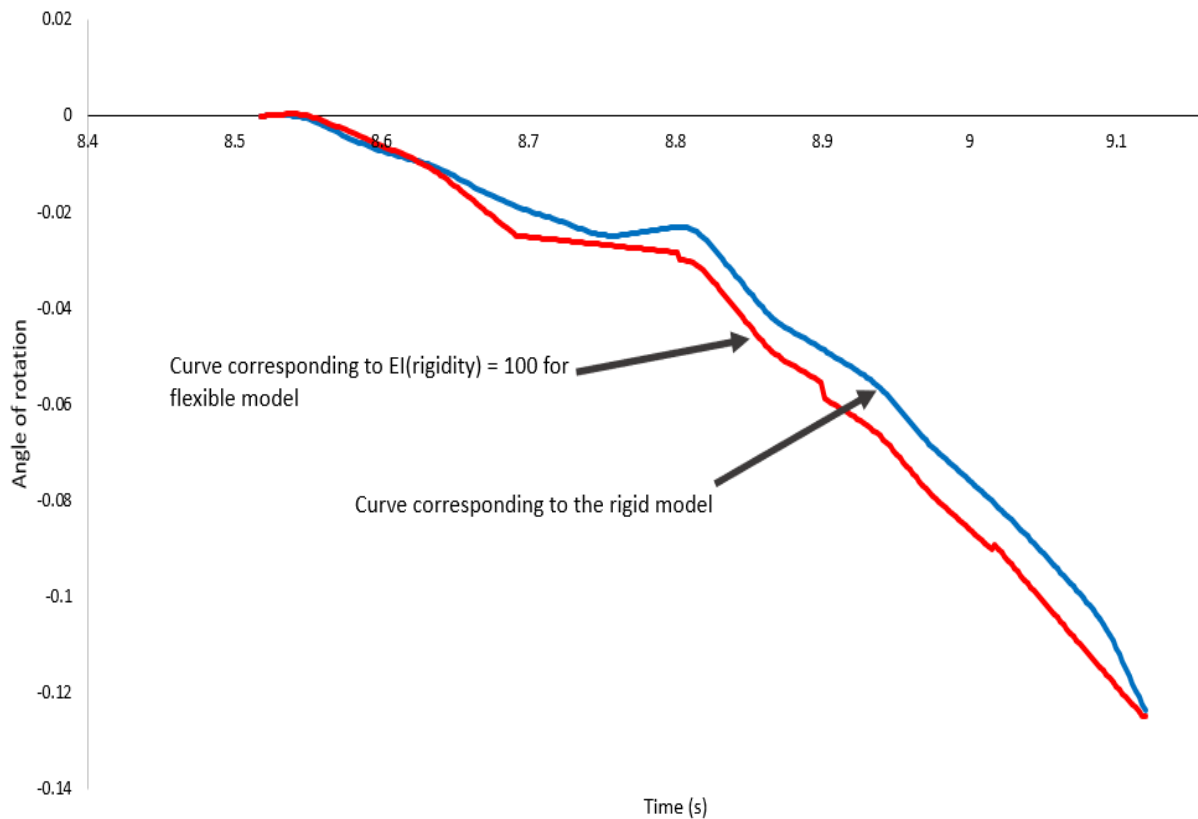


Figure 6-41 Comparison of rocking response using the flexible and fully rigid model

6.7 Investigation on the Seismic Intensity Responsible for Collapse

The destructive nature of the collapse observed and discussed in the previous chapters raises significant concerns over the seismic intensity that would lead to a collapse of this nature. Seismic intensity can also be examined based on the source-site distance. With reduction in the distance between the source and the site, the seismic intensity increases.

In this study, a ground motion modelling tool called the Component Attenuation Model (CAM) [169] is employed to convert seismic intensity values to their corresponding source-site distance values, assuming a seismic event of moment magnitude 5.5 (i.e. M5.5) for Melbourne ground conditions. The resulting plot is shown in Figure 6-42, which indicates that when the site is closer to the source, there is a sudden increase in damage at some point. This is due to the higher seismic intensity experienced by the structure resulting in a collapse as demonstrated through previous chapters in this study.

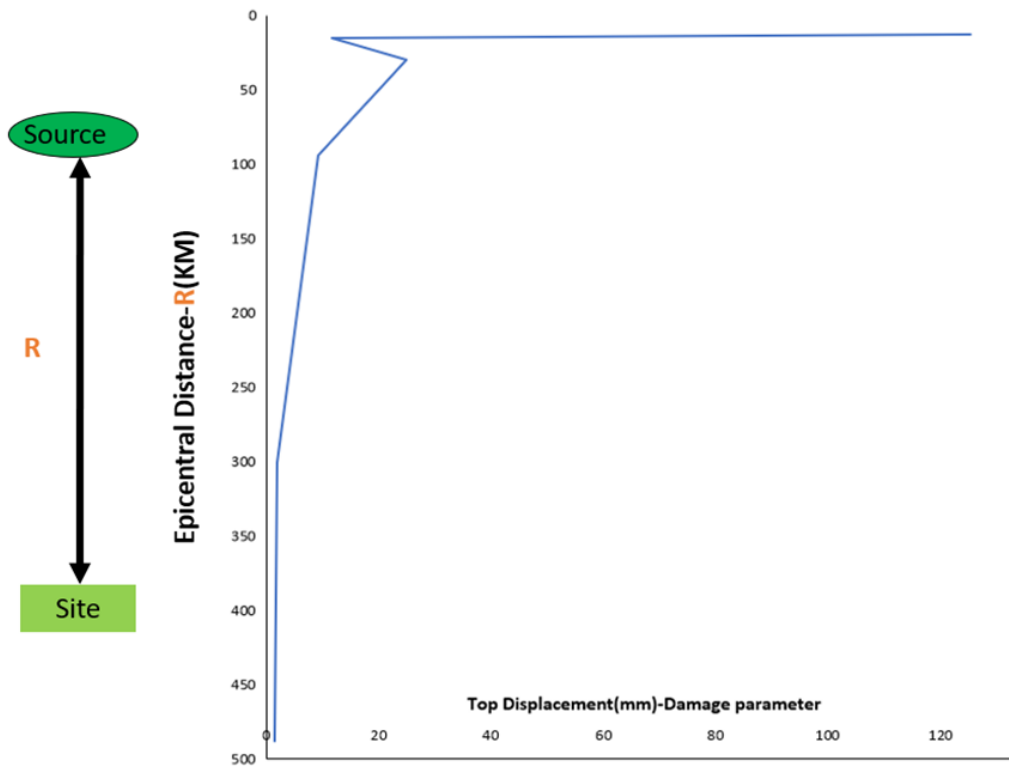


Figure 6-42 Variation in damage to the modular building with source-site distance

Considering the RSA value that causes overturning, the same CAM is used to estimate a corresponding seismic event (the intensity that causes overturning) for south-east Australian ground conditions.

Accordingly, the intensity that causes the sudden increase in damage is computed to be equivalent to that for a structure experiencing an M5.5 earthquake 10 km away from source, anywhere in south-east Australia.

This implies that any modular structure within the 300-km² area with the seismic source at its centre is likely to collapse in a manner similar to that observed during the shaking table experiment. This is illustrated more clearly in Figure 6-43.

This intensity of ground shaking that causes the collapse of the building is consistent with an MMI value of VIII-IX, as shown in Figure 6-44. An MMI of IX was observed during both the 1968 Meckering (Western Australia, Australia) earthquake and the 1989 Newcastle (New South Wales, Australia) earthquake. Both these earthquakes resulted in massive damage in urban areas. Hence, if a modular building similar to that studied here was standing in one of

these areas at the time of the earthquakes, it would have collapsed in a manner demonstrated through the shaking table experiments in section 4.3.

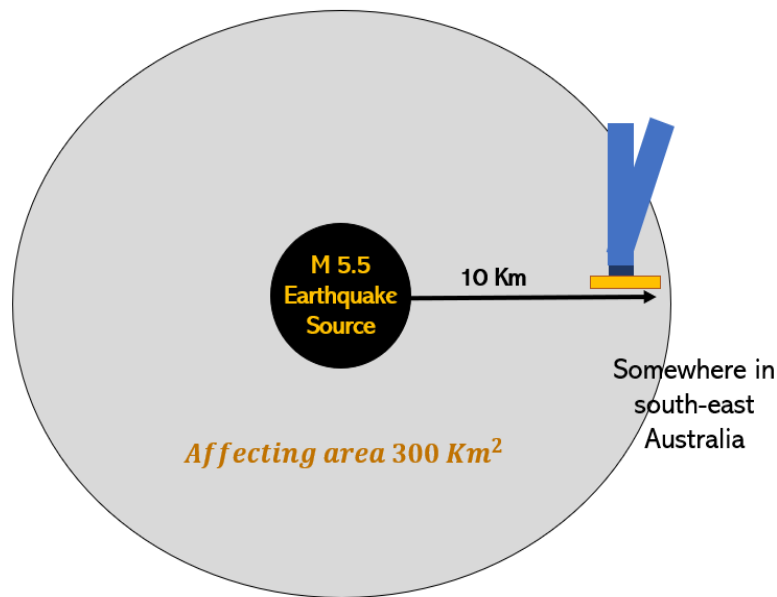


Figure 6-43 Graphical interpretation of the potential risk area

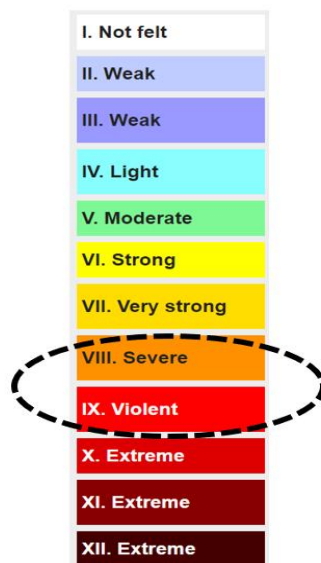


Figure 6-44 Locating the corresponding intensity on the MMI scale

6.8 Summary and Conclusions

- In this part of the study, experimental and numerical analyses were conducted to simulate and study the rock, slide, and overturn phase of the free-standing

superstructure component (last stage of collapse) formed as a result of the failure of the connections.

- A comparison between the ground displacement and the width of the structure revealed that the ground motion displacement was well below half the width of the structure. Hence, the ground motion displacement was ruled out as the cause of overturning, and the need to investigate the real cause of overturning was highlighted.
- As the next step, a simple preliminary energy balance was considered to check whether the kinetic energy of the superstructure gained from the failure of the connections was responsible for the overturning. It was deemed from the energy balance that the velocity of the superstructure right at the beginning of the free-standing phase exceeded the critical velocity required to overturn the structure.
- The energy balance method was considered to have provided a preliminary evaluation as it did not account for some other energy inputs and losses, which makes the results both conservative and non-conservative. Hence, further analysis was carried out to investigate this.
- A numerical model for the response of the superstructure component, when treated as a rigid body, was developed in MATLAB. The numerical model took into account the rocking and sliding behaviour of the rigid body under a seismic event.
- The numerical model was subjected to the ground motion stretch corresponding to the free-standing phase observed on the shaking table. Observations on the simulated angle of rotation, sliding displacement, and sliding velocity mirrored the shaking table observations on the rock, slide, and overturning phase with greater and more precise accuracy.
- The numerical model for the free-standing rigid body developed in MATLAB was subjected to the ground motion stretch corresponding to the free-standing motion, without any initial conditions from the previous phase (starting from rest). The numerical model predicted that the rigid body would not overturn. Its predictions were in excellent agreement with the results from the shaking table experiment obtained in section 4.4. These matching results served as a validation of the numerical model in MATLAB.
- It can be concluded from the experimental and numerical outcomes that the overturning of the superstructure component of the modular building occurred because of the initial

conditions established by failure of the connections. The energy required for the overturning phase was provided from the strain energy released from these failures.

- Reducing the strength of the connection to half the original strength and repeating the analysis resulted in non-overturning of the free-standing superstructure component formed as a result of connection failure. This implies that the connections released less energy when they failed than did the stronger connections in the original study.
- Sliding was found to be advantageous in terms of retarding the potential to overturn. More energy is dissipated from friction when sliding is allowed. Hence, less energy is left to be used for overturning.
- The overturning potential was found to decline with a decrease in the coefficient of friction between the sliding surfaces. Based on a parametric study, a safer margin for the coefficient of friction could be expressed to avoid overturning.
- Unlike in a general case of slide and rock, the superstructure component formed in this study did not have an infinite slide plane. The sliding and rocking took place on top of the first-storey modules. Hence, excessive sliding may result in the superstructure overturning because it moves out of the slide plane. Further, excessive sliding (out of the slide plane) may result in a reduction of the lever arm for restoring moment, and hence the restoring moment, resulting in premature overturning.
- The presence of roughness along the slide plane, which might cause a sudden halt to the sliding motion, was found to be disadvantageous in terms of exacerbating the overturning potential. The location of the halt from the start of sliding motion also had an effect on the overturning potential.
- The free-standing superstructure component was assumed to be rigid throughout all analyses. The final part of the study incorporated real-life flexibility into the superstructure component, and it was concluded that the incorporation of realistic flexibility into the free-standing body was favourable in terms of controlling overturning to some extent. However, the potential did not differ significantly from that under the assumption of a fully rigid body. Hence, the results from analysis assuming a rigid body can be considered conservative.
- The intensity of ground motion that caused the collapse was expressed in terms of probable earthquake scenarios for south-east Australia, using the ground motion modelling CAM. Further, some real ground motions that have occurred across Australia at an intensity capable of causing similar damage were highlighted.

Chapter 7: Recommendations for Safer Pure Modular Systems

Pure modular structures are the future of modular construction. Therefore, this study aimed to highlight the danger that may be involved in moving towards pure modular construction by merely adhering to current practices and code guidelines for conventional structures. The research showed that the pure modular structure under consideration would satisfy all the code requirements and perform in a very safe manner within the design limits.

However, the structure's performance during dynamic events exceeding the design limits was shown to be highly catastrophic. Generally, the code requirements do consider the safety of a structure during events exceeding the design limits, as observed for the conventional steel building analysed during this study. Beyond the design limits, the conventional steel frame dissipated damage in a progressive manner, while the modular structure collapsed in a concerning dangerous manner, overturning as a large block. Hence, this thesis concludes that mere adherence to available code provisions for conventional structures, and adoption of techniques commonly used in conventional construction may not necessarily guarantee safe performance of a modular structure, especially under dynamic events exceeding the design limits. All these concerns arise as a result of the nature of construction involved in modular buildings compared with conventional structures, most importantly, the alternate arrangement of connections and modules.

This chapter provides some recommendations for avoiding the hazardous nature of the failure observed during the experimental and numerical analyses in previous chapters.

Based on the experimental and numerical observations, the main contribution to the catastrophic failure observed in the modular structure can be attributed to two main unprecedented events that occurred during the dynamic time history:

1. Failure of the first-storey edge connections, leading to a failure of all other connections in the same storey in a structural unzipping manner
2. Sliding, rocking, and overturning of the superstructure component about the first storey

It is noteworthy that the second phenomenon was entirely dependent on the first. The failure of the connections in the first storey in an unzipping manner paved the way for a definite failure plane, on which the superstructure component could rock, slide and overturn. Avoiding the

first event may prevent the overall failure while avoiding at least the second will protect the structure itself and its surrounding structures. This is because if the overturning is avoided, the structure might just rock and settle without creating massive damages and casualties to the interior and exterior properties and occupants. As highlighted in section 6.4, the factor contributing most to the overturning was the energy released as a result of the failure of connections. This energy resulted in an elevation of the kinetic energy of the free-standing superstructure. Hence, employing methods for dissipating the energy released as soon as the connections fail may protect the structure from overturning. If a modular structure similar to that is considered in this study is already built, the failure nature of connections, as highlighted during this study, is inevitable. Still, the overturning could be avoided by arranging a mechanism to dissipate the energy released from the failure of the connections, in case if the structure experience a ground motion intensity that can cause so. The gap between the modules can be utilized for this purpose of any special installations to take care of the energy absorption.

Considering the nature of the failure of connections, such a failure which is restricted to a single failure plane (single storey) should be avoided. Even though overturning may be avoided as discussed above by incorporating energy dissipating mechanisms to absorb the energy released by the failure of connections, the formation of a free-standing block of superstructure itself is a risk because a sufficiently strong wind load itself might topple the free-standing superstructure formed. Furthermore, the repairing will be extremely difficult afterwards in terms of cost and effort. Hence it is vital to look into other forms of mitigation measures.

Some approaches that may help reduce the risk of hazardous nature of collapse are discussed in the following sections.

7.1 Mitigation Measure (I): Controlled Rocking of Building

Before a discussion on potential new approaches that are specific to modular structures, investigations are carried out on the adaptability of currently available seismic performance enhancement techniques that are used in the conventional structures. Controlled rocking, tuned mass dampers, Seismic Cloaking, Steel plate shear walls and lead rubber bearings are some of the methods used in practice in conventional structures. Even though there are many different approaches available in practice, not every approach will be beneficial and applicable for modular buildings due to the discrete construction nature involved in modular buildings. However, some of the techniques like controlled rocking with self-centring and energy

dissipating fuses can be very beneficial and easily applicable for modular structures than for conventional steel or concrete structures.

The practice with current design methods based on current building codes is to force the failure of specifically designed lateral force resisting elements in the structure and perform satisfactorily under seismic events while ensuring life safety. As per capacity design principals, these specifically designed lateral force resisting elements dissipate the ground motion energy input and protect other primary elements of the structure from experiencing loadings beyond design limits. Buckling restrained braced frames are an excellent example of such structural systems. However, the requirement for base rocking systems came up following two main issues with these conventional LFRS. Large residual deformations and drift concentrations were found to be jeopardizing the overall performance of the LFRS and significantly increase the repairing cost. Engineers were not satisfied with the economic impact associated with the repairs involved with the code-based design approaches, after an earthquake. Despite the satisfactory performance in terms of life-safety under the code-based design approaches, the shift to base rocking structures was considered to design structures beyond life-safety as a minimum performance level [170].

In contrast to the performance of conventional structures, under previous chapters, this study revealed that the inter-modular connections which are a primary component of the modular buildings are the most critical component and the structures collapse disproportionately due to the failure of connections. It was observed that the failure of these connections could not guarantee a life-safety performance level, whereas, under the same seismic conditions, the failure of sacrificial LFRS in the conventional steel frame could guarantee a life-safety performance of the conventional structure.

Considering the experimental and numerical observations presented in this study, one of the best methods of guaranteeing a safer seismic performance of pure modular structures is avoiding any failure occurrence in the modular structure. Allowing an uplift and rocking of the whole structure has been proven extremely beneficial in terms of limiting forces transmitted into the structure [170-174]. This way, the design strength and ductility demand required of connections can be lowered.

The lateral loading in any structure is highest at its base. This maximum loading is called the base shear. The modular structure considered in this study was analysed, considering a rigid,

fully fixed foundation. Hence, the next most critical (vulnerable) spot in terms of heavy lateral loadings would be the first storey. However, suppose the foundation of the structure is made flexible by allowing to rock. In that case, the loadings imparted on any other storey will be much lesser than when the foundation is rigid. This will protect all the connections in the structure and protect the structure from catastrophic collapse, as demonstrated through previous chapters in this study (see Figure 7-1).

It was proven through experimental (section 4.4) and numerical (section 6.4.1) analyses that the leading cause of overturning of the superstructure component formed as a result of the failure of the connections was the energy dissipated from the sudden failure of the connections. However, when the whole structure is allowed to rock, the chance of connections becoming critical and failing is avoided. Hence, a catastrophic failure, as demonstrated through the previous chapters, is avoided.

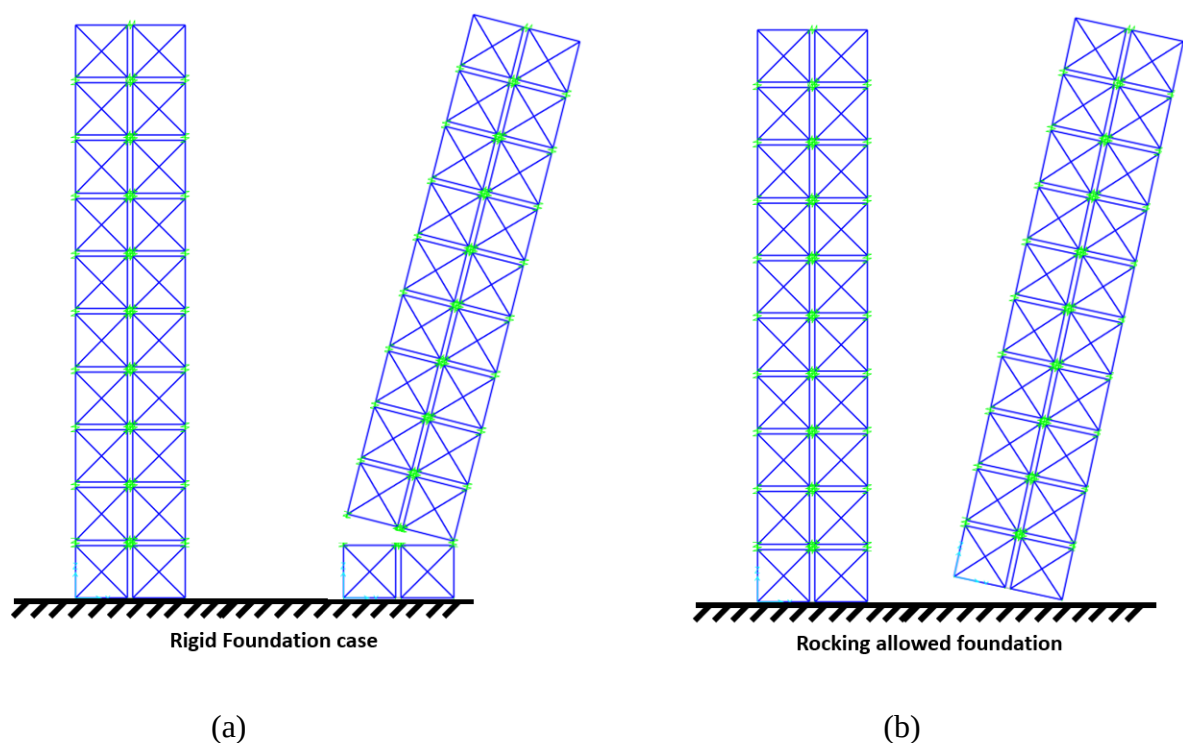


Figure 7-1 Comparison of seismic response under (a) rigid foundation (b) rocking allowed foundation case

Despite the excellent performance expected during seismic events when the full structure is left free-standing, it may not be practically possible to allow a structure to be entirely free-standing, especially considering other loadings such as wind loadings which might easily topple the structure. Hence, attention should be given to partially restraining the structure. A

partial restraint can be a limited, weak restriction of the foundation which allows some uplift of the structure even under small loadings but stops the structure from excessive uplift. Alternatively, a partial restriction can come in the form of a full restraint of the foundation up to a particular loading (can be called as a yield loading) level and allow full or partial free uplift of the structure after that specific loading.

Three main concerns need to be addressed in achieving a practically feasible controlled rocking modular structure. First and one of the most important issues is, if a partial restraint is provided up to at least some yield load level in the foundation, extreme care should be taken to design the foundation connections to not fail in a brittle manner as demonstrated through the previous chapters in this study for the inter-modular connections. If such a brittle failure is observed, it might easily overturn and collapse the structure in a manner similar to what was observed under the failure of connections. However, foundation connections are entirely different from conventional bolted connections in a structure. Further, special care could be given in designing foundation connectivity for modular structures so that a ductile energy dissipation mechanism could be incorporated to allow a smooth energy dissipation upon yield and allow a safer uplift of the structure. Special research could be carried out to come up with a *modular foundation* where special energy dissipation mechanisms are incorporated into the foundation component manufactured at controlled factory environments. The same could be designed to yield and allow a controlled uplift of the structure.

The second and third concerns in allowing controlled rocking are widely discussed and answered in many researchers. They are the potential of large rotations and damage that could be caused by impacts during rocking. Currently used techniques in conventional structures to avoid these problems could be modified and applied to modular structures. Current practices commonly involved the use of post-tensioning or specifically designed damping devices that could control excessive rotations during rocking. Some studies have considered the use of buckling restrained braces, viscous dampers, and soft pads at the rocking surface as a technique of energy dissipation and controlling the effect of impact at the phase of recentring. Most of these techniques, together with the discrete nature involved in modular constructions, can be effectively used to induce a safer seismic performance of the rocking allowed modular structures [172, 175-177].

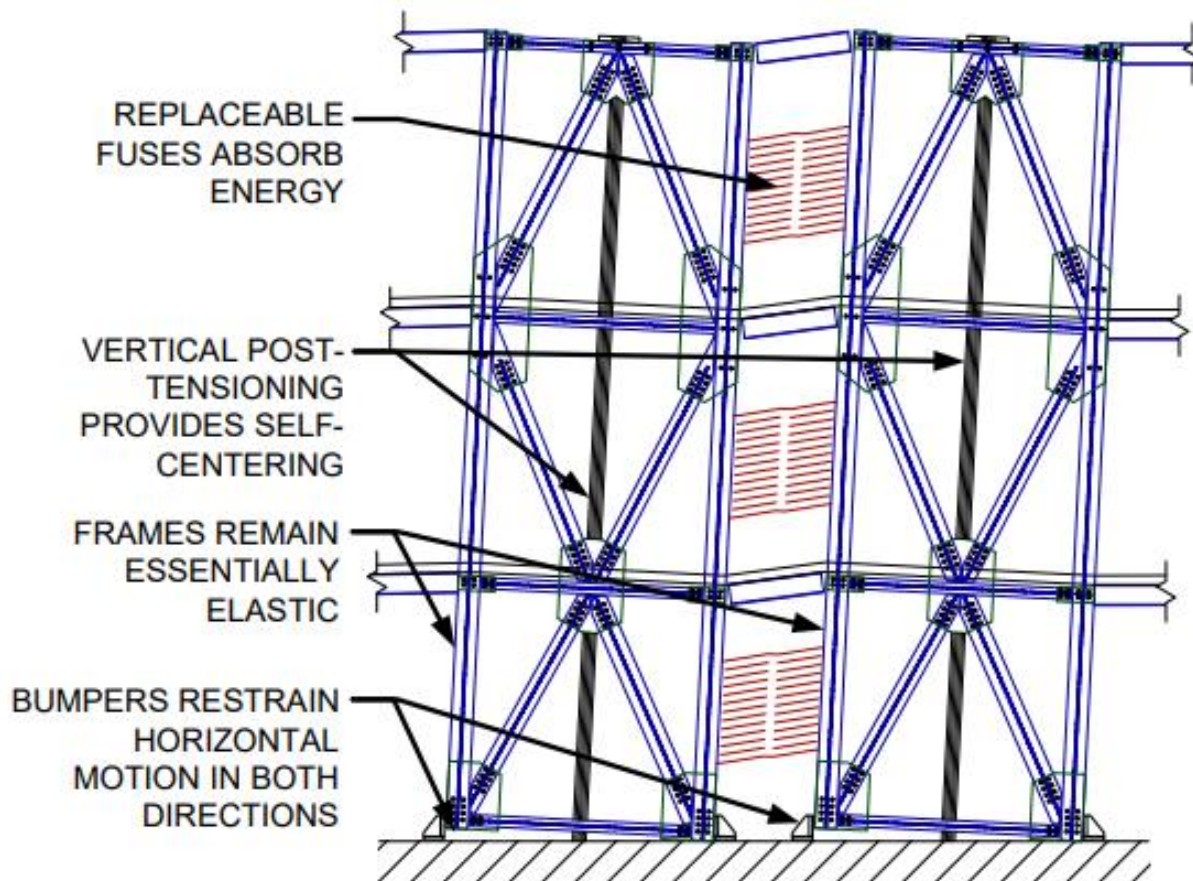


Figure 7-2 Example controlled rocking system in a conventional structure [176]

Figure 7-2 depicts a controlled rocking allowed frame system with replaceable structural fuses to allow a safer seismic performance as reported in Ref [176]. This consists of post-tensioned strands which takes part in the self-centring of the frame. The replaceable structural fuses act as energy dissipators to limit the stresses induced on the rest of the structure.

Even though the post-tensioning approach to self-centre the frame may be against the concept of pure modularity where additional work will be left to do on-site, the rest of the approaches can be easily incorporated to modular structures. As can be seen in Figure 7-2, despite the frame considered being a conventional steel frame, a gap between the two bays had to be incorporated to include the replaceable fuses between the frames. However, in modular buildings, this gap is inherently available due to its construction nature, making it more convenient for the installation of any structural fuses to dissipate energy. Further, a gap is also available in the vertical direction of the modules which can also be employed to install structural fuses and to install some mechanism to take care of the self-centring of the frame.

Moreover, it was highlighted that the impact during the rocking could be of concern in terms of damage to the whole structure. In the current practises, as mentioned before, a soft impact during rocking is ensured by using different techniques. In the modular buildings, apart from ensuring a soft impact between the foundation and the structure, viscous dampers or soft pads can be placed between each vertical modules to make sure the loads between the modules due to any impact during rocking is taken care of. All these are demonstrated, as shown in Figure 7-3.

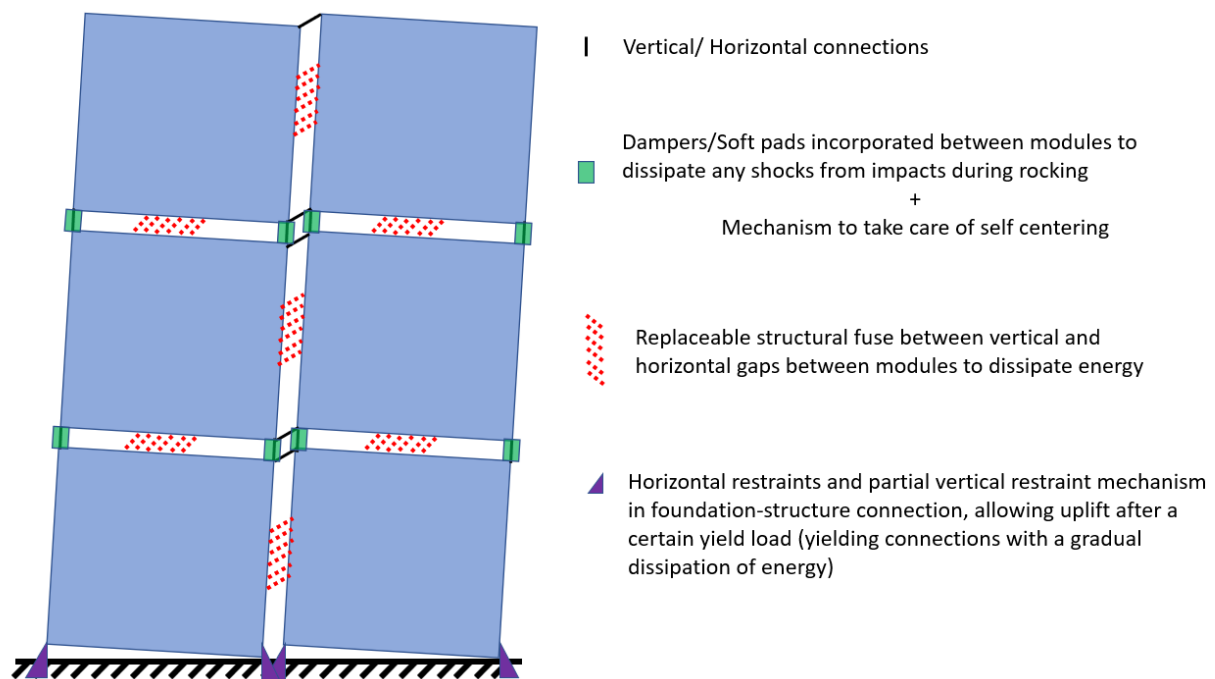


Figure 7-3 Example potential controlled rocking system in a modular building

As shown in Figure 7-3, if the two stacks of modules are allowed to rock independently rather than all six modules rocking as a single stack, energy dissipation between the two vertical stacks can be effectively activated to dissipate more energy. This is easily possible in modular construction due to the construction nature of it.

In contrast, in conventional structures, if this mechanism was to be employed, a gap between each bay has to be deliberately made during the construction of the structures. This can be further illustrated using Figure 7-4. Each of the added energy dissipation mechanisms in the order of ‘a’ to ‘c’ facilitates dissipation of much higher energy.

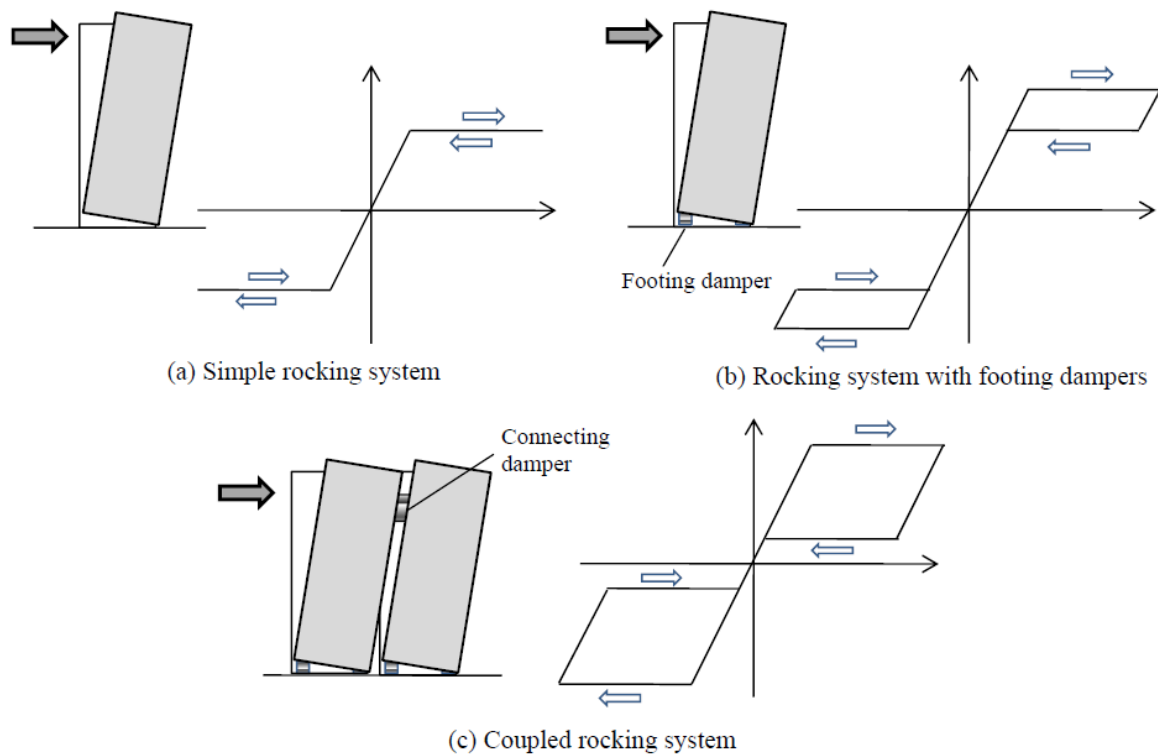


Figure 7-4 Different possible energy dissipation mechanisms [171]

Throughout the journey of modular construction, a component that received the least attention was the foundation. This was considered the only task that needs to be completed at the site. Conventional approaches of foundation, as discussed under section 2.1.7.2, were followed in achieving the module to foundation connectivity. However, adoption of uplift allowed foundation gives the flexibility to manufacture some of the foundation component off-site and move towards a *modular foundation*. This is due to the less requirement of the foundation being strongly connected to the ground and the need to incorporate some sort of mechanism for energy dissipation, damping and yielding upon exceeding particular loading into the foundation module.

Figure 7-5 illustrates a viscously damped controlled seismic rocking (VDCSR) system reported in Ref [178]. The design of the foundation facilitated column rocking and uplift over the foundation. The uplift force was controlled by the dampers mounted beside the column and dampers embedded inside this foundation. The seismic controlling action of the foundation was included inside the steel casing of the foundation, and the entire foundation component was connected to the ground using concrete anchor rods. A foundation of this nature will allow most of its components to be manufactured in off-site factory environments. Adopting a similar

approach to develop a modular foundation for modular structures may not just be beneficial in terms of seismic performance, it will also reduce the time and cost on the foundation component of modular structures, providing significant economic benefits.

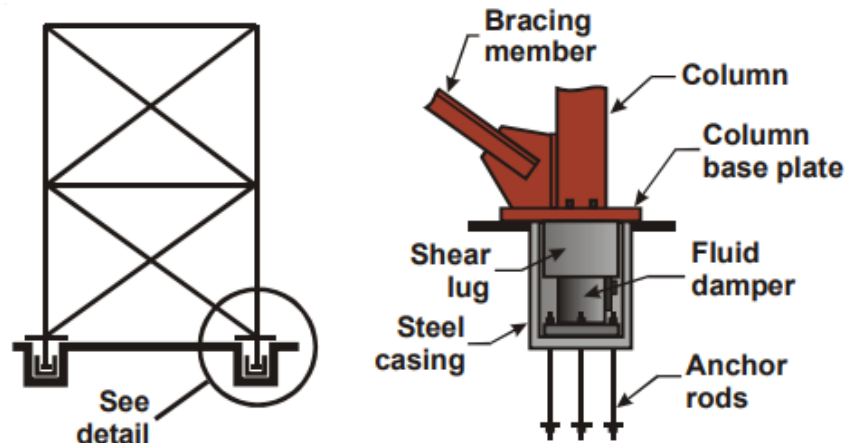


Figure 7-5 Viscously Damped Controlled Seismic Rocking (VDCSR) system [178]

Apart from the concepts derived from the studies that have already focussed on uplift and rocking allowed foundations, some additional measures that can also be incorporated can be suggested based on the main outcomes of this study under previous chapters. Based on the outcomes of this study from previous chapters, it was concluded that sliding has been beneficial in terms of mitigating overturning (section 6.5.1). It was also revealed that the sliding could be dangerous when it is occurring on top of ground modules (due to limited slide plane) or when the sliding body encounters a halt which may trip the sliding body. However, if little sliding could also be allowed to occur at the base under extreme seismic events, it might serve advantageous in term allowing a safer performance of the structure while also being protected by the allowed uplift of the structure.

7.2 Mitigation Measure (II): Connection Design

As highlighted in Section 2.1.7.3, most connections used in modular construction are bolted connections, in which bolted splice action is inevitable. In comparison with the larger frame sections, braces, concrete wall panels and all structural components in a module, the connections are weak. Moreover, these connections are at the same level, stranded between upper and lower modules and attracting heavy inelasticity due to their shorter height, especially as a storey. Some approaches that may help avoid the cause of these problems are discussed below.

The most straightforward solution that can be specified to avoid the failure of connections is to strengthen the connections. However, there are many constraints in moving towards stronger connections. For example, welded connections would be a suitable choice in terms of their strength as inter-modular connections. Still, such connections involve significant on-site work, and it may not be possible to access all connections when the modules are stacked in place. As these inefficiencies run counter to the concept of pure modular structures, welding should not be used as a mode of inter-modular connectivity.

Bolted connections, as mostly used in practice, behave like brittle connections because of the nature of their failure, which, as discussed in-depth in Section 2.6.5. Thus, although bolted connections can be made much stronger, it is challenging to make them ductile. Yet, without ductility, the connections have to be very strong indeed to resist extreme dynamic events. Achieving such high strengths may have economic as well as physical constraints in practice. Hence, it is vital to have a sufficient balance of ductility and strength in introducing new connection systems for modular structures.

Section 2.6.5 highlights that the loss of connection grip, which results in activation of the splice action, causes failure of the connections. Hence, if the splice action could be restrained to some extent, the ductility and strength of the connections could be improved.

Figure 7-6 illustrates two possible configurations that convey the basic idea behind restraining the splice action in a bolted splice connection. The thickness of the restraining part and other parameters could be designed as per strength and ductility requirements. This basic concept might be expanded to develop more complex bolted connections that can provide the required balance of strength and ductility to protect the structure even under conditions beyond design limits.

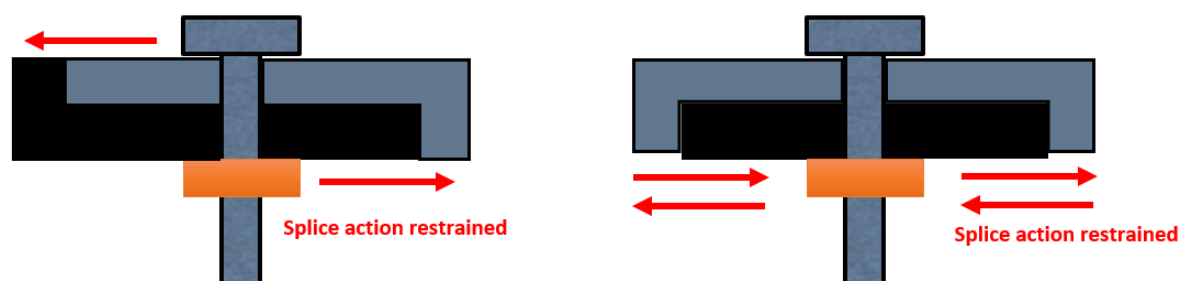


Figure 7-6 Representation of restraining splice action in a basic bolted splice connection

General practice in designing a structure is to use different element sections for different storey levels when the structure height is sufficiently tall to classify as a high-rise structure. However, for short- to mid-rise structures, the same section sizes are used throughout the structure height, considering it as a conservative approach. The same methodology is usually applied to elements such as inter-modular connections.

In the case of a conventional structure, following this approach will not have a significant effect on the elements, other than being conservative and beneficial for the structure. This is because of their continuity along the structure height, which enables them to distribute inelasticity throughout the height. However, in the case of a modular building, as highlighted in this thesis via experimental and numerical models, the inelasticity concentrations focus on connections. Moreover, the lateral loadings are the highest in the bottom storeys. Thus, if the same (same in terms of strength and stiffness) connections were employed for every storey level of the structure, the bottom-most storey connections would experience higher stresses while connections in the upper storeys (storeys other than the first storey) would experience lower stresses for the same- connection capacity. In this way, the chance of upper storey connections sharing any unforeseen lateral loadings from loadings beyond the design limits is lost. This is because of the first-storey connections giving up first as a result of their capacity being exceeded (storeys other than the first storey). This is one of the factors contributing to the inelasticity not being able to be distributed to the other connection storeys of the structure.

This problem can be addressed by using connections with different strength and stiffness in different storeys. Special care should be taken to provide higher-strength connections to the first few storeys of the structure so that they have been treated fairly for what they undergo during loadings. In this way, the failure can still be redistributed throughout most connections along the structure height. Even though the approach of using the same elements for all storeys looks like a conservative approach in conventional structures, the same can lead to unforeseen collapse mode in modular structures because of their discrete nature of force distribution. The failure can be forced to be distributed throughout all the connections in the structure height by designing connections in less critical storeys to be weaker than that of more critical storeys like the first storey.

Apart from distributing the failure across many connections in different storeys throughout the building height, another approach that could be followed is to make all the connections and the entire structure significantly stronger; that is, to design the whole structure for a 2,500-year

hazard return period instead of the conventional approach of designing for a 500-year hazard return period. This may sound uneconomical, but it may cost less than the damage if a 2,500-year hazard return period earthquake occurs. This could be studied through a cost comparison study in which the extra cost of designing the structure to withstand a 2,500-year hazard return event is computed as a percentage of the cost of designing it for a 500-year hazard return period earthquake.

In one such study, the authors conducted a cost comparison of different structural configurations in a conventional structure designed for various seismic hazard levels [179]. Table 7-1 summarises the outcome of their study: As per the study, the cost of building for a 2,500-year hazard return period was not substantially higher even for frames with many storeys. For example, the cost of building an eight-storey dual system designed to withstand a 2,500-year hazard return period earthquake was only 12% higher than for a 500-year hazard return period earthquake.

This sort of cost calculation may suggest that the extra cost of building modular structures to withstand a 2,500-year hazard return period earthquake is not as high as it seems. This is mainly because the only design component that needs extra strengthening and ductility is the connections, which may not result in substantial extra cost.

Table 7-1 Cost comparison of building to withstand different seismic hazard levels [175]

Structural System	10% probability of exceedance in 50 years		2% probability of exceedance in 50 years		Unit Cost Difference (TL/m ²)
	Total Cost (TL)	Unit Cost TL/m ²	Total Cost (TL)	Unit Cost TL/m ²	
3 story frame	24,886.1	79.00	24,960	79.24	0.23
3 story dual system	28,407.4	90.18	29,351.1	93.18	3.00
5 story frame	41,607.6	79.25	43,189.5	82.27	3.01
5 story dual system	47,936.8	91.31	51,635.1	98.35	7.04
8 story frame	69,815.9	83.11	115,598.4	137.62	54.50
8 story dual system	82,774.2	98.54	93,471.4	111.28	12.73

TL, Turkish Lira

7.3 Mitigation Measure (III): Modular Unit Design

Designing a structure for higher seismic hazard levels using strong connections or designing the bottom-most storeys connections with higher strength are two approaches to protecting the

structure, at some additional cost. However, designers may not always opt for incurring extra cost for a project to protect the structure from an unforeseen extreme event.

Another approach to making the connections relatively strong is to deliberately make the modules weaker to allow either only the modules or both modules and connections, to yield. In this way, any failure in the structure will be more evenly distributed throughout the structure height, like in a conventional building (see Figure 7-7).

As illustrated in Figure 7-8, if the failure can be transferred to the modules, the area over which it can be spread and accommodated is much larger. However, with the current practices and generally available connections and modules used in the industry, the modules tend to be much stronger than the connections, resulting in connections giving up first as observed during the experimental and numerical observations in this thesis.

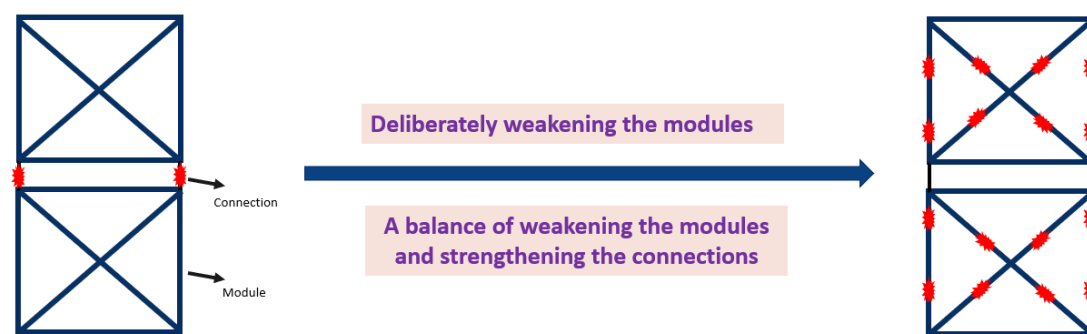


Figure 7-7 Distribution of damage throughout the structure by allowing the modules to yield

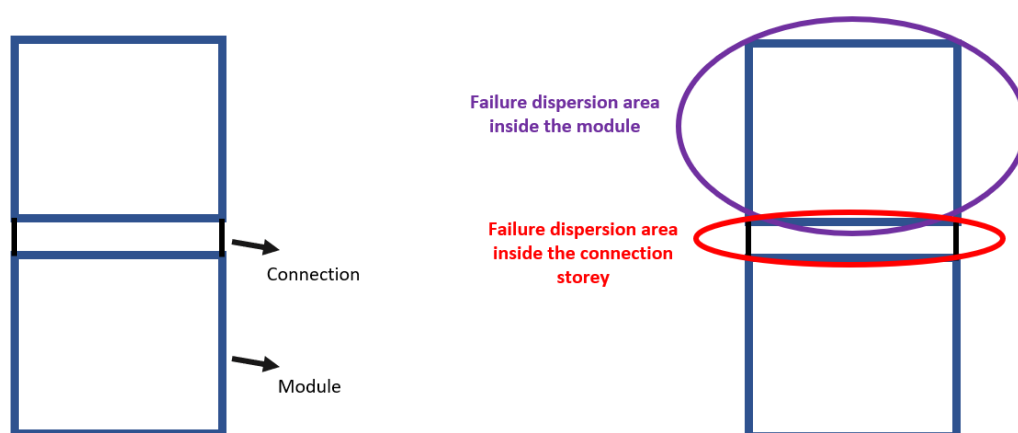


Figure 7-8 Representation of spread of yield/failure in different areas of a modular building

Transferring failure initiation and propagation to modules will require a balance between a slight increase in the strength of the connections and a decrease in the strength of the modules. Reducing the strength will require extra care to ensure that the modules are still able to resist the gravitational and lateral loadings expected within everyday operations and design limits. Numerical analysis can be carried out to optimise the section sizes for use in modules that can withstand design loadings as they are the first to give up when loadings exceed the design limits. If modules are forced to yield or fail before connections, the structure failure will be more like that of a conventional steel frame undergoing failure. The failure will be more evenly distributed throughout the structure, and there will not be any brittle failure such as that experienced when a bolted connection undergoes failure. The modules will have a higher ductile capacity due to many elements inside them and the ductility of each of these elements contributing to the complete failure to be ductile.

This approach is very promising as a way of avoiding the destructive nature of the collapse observed in pure modular systems.

7.4 Mitigation Measure (IV): Configuration Design

It is evident from previous chapters that both factors (two unprecedented events discussed at the beginning of this chapter) contributing to the failure of the modular building resulted from the failure of just one edge connection of the critical storey. This resulted in opening up of the other connections of the same storey in an unzipping manner, ultimately resulting in a defined failure surface, which is not a common occurrence in conventional structures.

A new structural configuration is proposed here in which the modules are assembled in a vertical orientation, as shown in Figure 7-9.

As highlighted in Section 5.2.3.3, axial forces were dominant in the first-storey edge connection in which the failure initiated. When the structure swayed under the ground motion, the connections at the edges were subjected to push-pull action, which triggered them to fail. However, with the new configuration shown in Figure 7-9, the number of horizontal connection planes would reduce to just one, from five in the conventional approach (in this particular case), for the same number of modules and hence the same plan area.

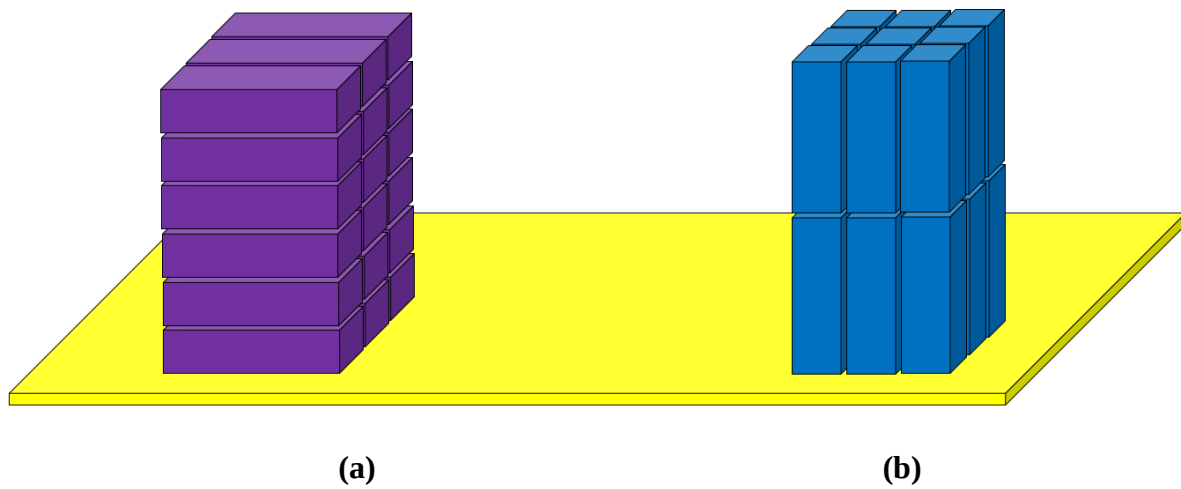


Figure 7-9 (a)Conventional arrangement (b) Proposed new arrangement

Moreover, these vertically placed modules could be manufactured with mezzanine floors with open areas that could be combined to form the same floor space that would have been available via the conventional approach (Figure 7-10).

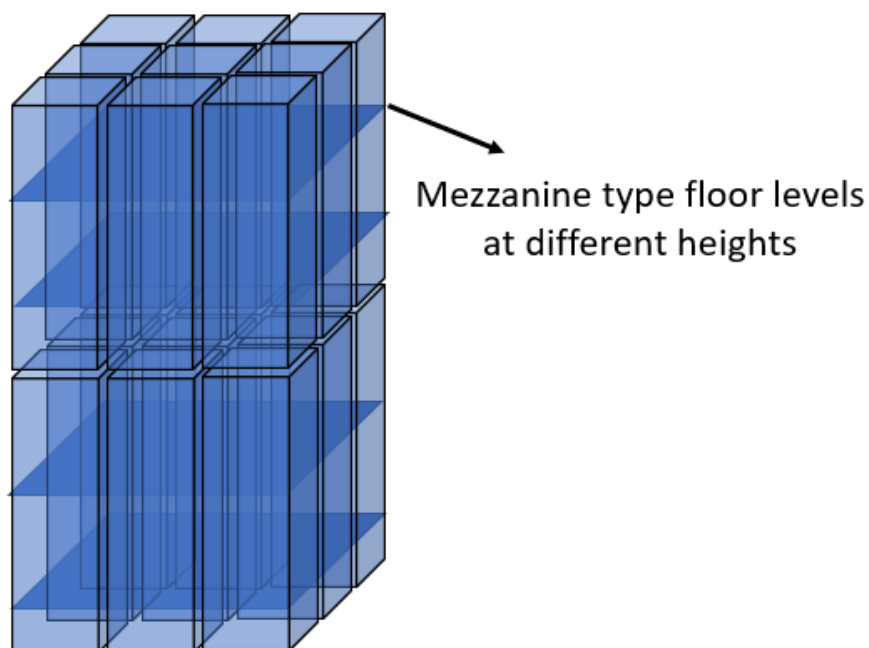


Figure 7-10 Floor levels inside vertically stacked modules

Apart from the advantages mentioned above, this arrangement would act more like the bundle-tube type (Figure 2-46) structural system employed in tall-building construction. Bundle-tube

structural systems have proven to be a very effective means of lateral load resistance, especially for tall structures where the lateral loads are critical.

A preliminary study involving two, 2D structures, one modular building constructed using the conventional approach for modular buildings; and the other constructed using the new approach suggested here, was conducted using FE models. Both structures were of 15 storeys, as shown in Figure 7-11.

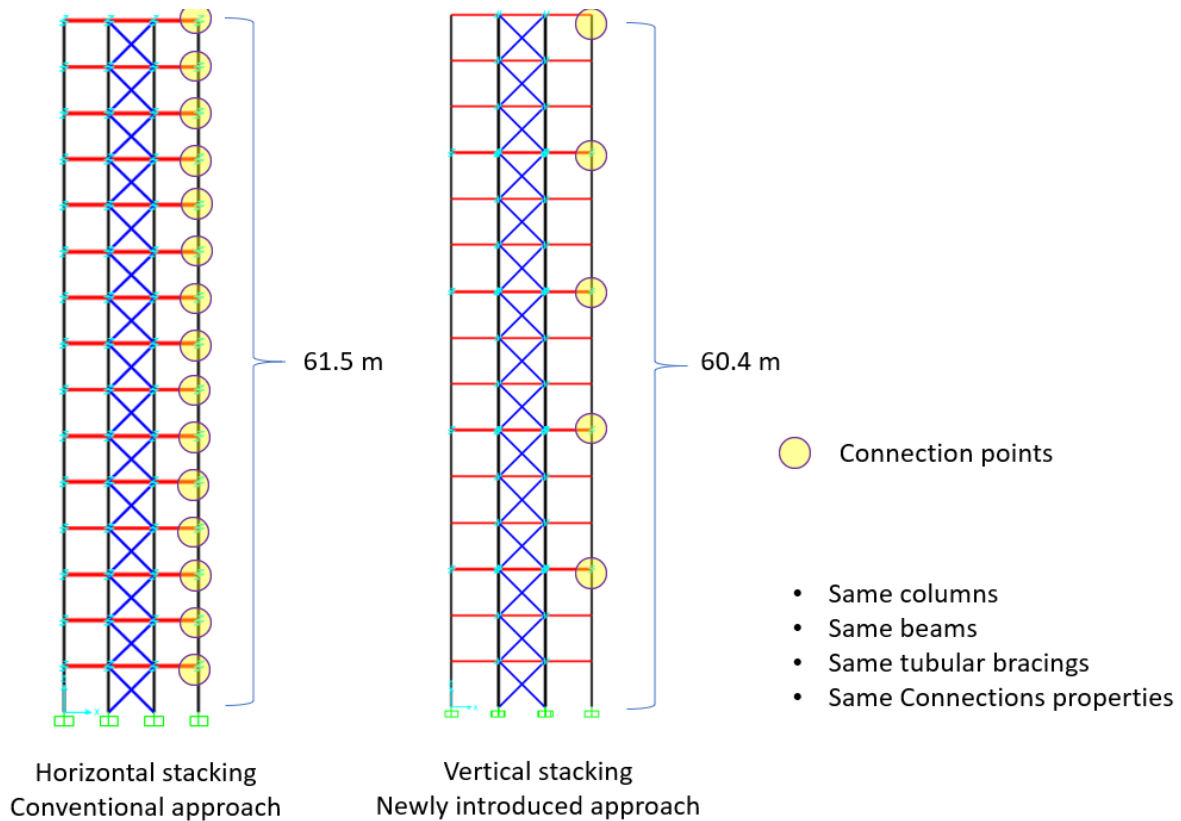


Figure 7-11 Comparison of 2D models of horizontal and vertical modular building construction approaches

The two structures were analysed and compared in relation to basic code requirements such as fundamental period, pushover analysis (capacity spectrum), mode participation and performance under a code-based response spectrum. The new structure's performance was highly satisfactory in that it adhered to all code requirements, demonstrating the potential of a structural system of this nature as an improved pure modular building. However, further analysis needs to be carried out using high-fidelity FE models to better understand the performance of the structure and the nature of collapse involved in this structure under dynamic events exceeding design limits.

Another advantage of this newly introduced structural configuration is that the vertical assembly could be done with modules of different vertical heights so that the vertical connections do not fall in the same horizontal plane (Figure 7-12). This would be beneficial in terms of avoiding structural unzipping type failure across one storey if one edge connection fail. If the unzipping type of failure across a storey can be avoided:

1. The damage is more likely to be distributed throughout the structure.
2. Forming of a definite failure plane, which may allow the superstructure component to rock, slide, and overturn, as observed for modular buildings constructed as in current practice, is avoided.

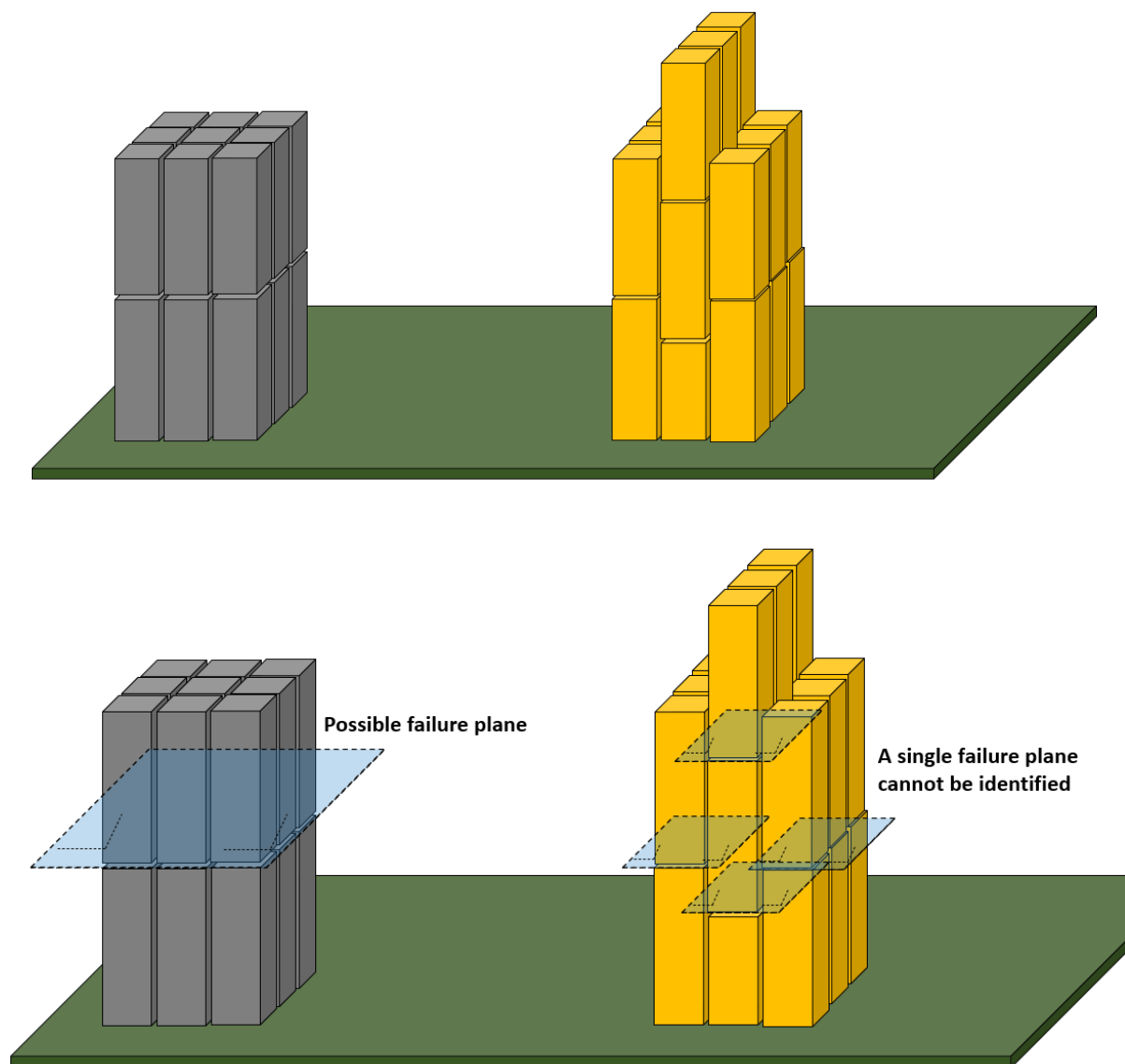


Figure 7-12 Comparison of different possible structural configurations and identification of possible failure plane

7.5 Summary and Conclusions

A summary of the important content that was highlighted in this chapter is presented below.

- In this section of the study, four mitigation measures that can be incorporated into pure modular buildings to avoid the hazardous failure involved in them (as presented under previous chapters) were presented.
- First mitigation measure involved moving the inelasticity concentration away from the superstructure by allowing the structure to undergo controlled rocking. Furthermore, the usage of energy dissipating mechanisms between modules and at the foundation during rocking was also proposed to safely dissipate the energy during dynamic events.
- Second mitigation method involved designing the connections to be stronger and ductile through avoiding the splice action (which is the cause of brittle failure nature of bolted connections, hence, the modular building) in bolted splice connections.
- Third mitigation method involved distributing the inelasticity throughout the structure in contrast to the current situation where the modules are relatively stronger than connections which give up first during dynamic events. Modules were proposed to be manufactured comparatively less strong so that the failure initiates and propagates in the modules once the design limit is exceeded.
- Fourth mitigation method involved the introduction of a novel configuration design. The proposed configuration involved installing the modules in a vertical manner to replicate bundle-tube structural systems which are highly efficient in resisting lateral loads in high-rise buildings. Furthermore, the proposed configuration design had the possibility of avoiding a definite failure plane, availability of which led to sliding, rocking, and overturning of conventional pure modular structures.

Chapter 8: Conclusions and Recommendations

8.1 Conclusions

This study was aimed at highlighting the potential risk that may be involved in pure modular structures constructed without the aid of any external LFRS, when they are subjected to seismic events exceeding the design limit. Five main objectives were set in achieving this aim of the study. The following is a listing of the main findings reported in this thesis which are mapped against the research objectives presented in Chapter 1

- 1) *Development of finite element models of a prototype modular and conventional steel building analogical to each other in terms of physical dimensions. These models are to be employed to establish their expected static response and dynamic response under dynamic events beyond design limits.*

Towards achieving the first objective, an investigation was first carried out into the current practice of using external LFRS such as core walls, shear walls or mega braces for conventional low- to medium-rise buildings. The investigation revealed that based on current industry practices and previous research outcomes, small- to mid-rise modular structures can be built without the requirement for any external LFRS. Based on this investigation outcome and previous studies, it was deemed that a pure modular building and a conventional steel frame of 10-storeys could be considered in this study without any requirement for an external LFRS. In the next step of the study, the prototype building was modelled in various FE software, taking into account the code provisions for Melbourne ground conditions. The building was deemed to satisfy all the requirements that a designer would look for in terms of static and dynamic analysis, for the considered region and for a particular design earthquake return period. Hence, it was concluded that a mid-rise pure modular structure could be built to adhere to all design requirements and perform safely within the design limits.

Static and dynamic analysis of the high-fidelity FE model of the prototype modular structure indicated that when the modules and connections were adequately modelled, and the structure was subjected to a seismic event beyond the design limit, the failure in the modular structure was concentrated more in the connections. Specifically, the first-storey edge connections were critical. The modules, which were made up of many frame elements

and lateral force-resisting brace elements, did not undergo any significant failure and continued to remain safe while the connections gave up. This verifies the hypothesis that the modular construction's discrete nature would force weaker connections to fail while the modules would remain safe, even under extreme loadings. These observations on the prototype models made it convenient to model modules as rigid boxes that will never fail in experimental analyses.

- 2) *Development of scaled-down experimental models of the modular building, a control model representing a free-standing stack of modules, and the conventional steel building for analysing static and dynamic response using a shaking table.*

Experimental analysis (pushover analysis and dynamic analysis) on the modular building's scaled-down model resulted in a response very similar to what was observed under the analysis of the prototype modular building. It was observed that the failure in the modular building initiated from the first storey edge connections and then progressed rapidly to the interior connections of the same storey. The unzipping-type failure across the first storey was one of the significant discoveries of this study. This is not a standard mode of failure seen in conventional structures. Hence, bringing this issue to the fore can be considered a massive contribution of this study to the modular building research community.

Further, with the increase in the seismic intensity, the free-standing mass displayed sliding alongside rocking followed by overturning. This was one of the most important observations during the experiment, as the overturning of the superstructure is unlikely to be observed in conventional structures. This observation is unique to modular constructions because of the specific nature of their construction. Apart from being a unique mode of failure, a failure of this nature is quite catastrophic and would be dangerous for building occupants as well as the surroundings. A failure of this nature would leave no time for escape.

Following the observations on the formation of a free-standing stack of modules that experienced overturning, the same stack of modules was placed on the shaking table to stand freely and experience the same stretch of ground motion corresponding to the free-standing motion of the overturned stack of modules. It was observed that the stack of modules that started the rocking phase from rest did not overturn under the same ground

motion stretch. The experiment hinted towards the role of the initial conditions of the superstructure prior to the rocking phase in its overturning and the eventual collapse.

In contrast to the hazardous response of the modular building, the conventional steel frame performed in a much predictable manner under the applied seismic loading. The damage was not focussed on the primary components of the structure. The LFRS took much of the damage, distributing it throughout the structure height, rather than being restricted to a single storey in the structure. As well as being distributed throughout the structure, the damage occurrence was progressive with the seismic intensity. This implies that the rate of damage progression remained virtually the same regardless of whether the seismic intensity was within or above the design limit.

3) Development of numerical models of the scaled-down modular building and the conventional steel building to repeat the static and dynamic analysis conducted on their experimental models and compare the experimental and numerical results.

This objective was explicitly set to understand the experimental models' collapse response in more details while also simulating the experimental observations in the FE model. The FE models of both structures were validated by experimental results, which showed excellent agreement in terms of modal, pushover and nonlinear dynamic THA results. Furthermore, successful validation of the modular building's numerical model using experimental results served as a validation of the element removal algorithm developed in OpenSees to simulate failure/collapse in the modular building.

Results obtained from the numerical model of the modular building further clarified the experimental observations on the sequence of connections failure with their exact timings. Furthermore, the numerical models were able to output the force variations of each connection. First-storey connections were in a more critical state than any other connections. Of all the connections in the first storey, the edge connections were found to be experiencing a higher axial pull-out than shear force time histories. These edge connections initiated the failure in the storey. Under a seismic event, all the interior connections were found to be experiencing higher shear than axial forces. Results from the numerical models revealed that as soon as failure initiated (in the edge connections), there was a sudden amplification in the interior connections forces, resulting in their failure.

Notably, the connections in other storeys were intact while the entire connection plane in the first storey saw a progressive failure starting from an edge. The sharp increase in the structure's damage variation (roof displacement) coincided with the failure of connections. It was further observed during the numerical and experimental analyses that there was much lesser headway before the structure collapsed after the initiation of the failure of an edge connection. The sudden or brittle nature of the failure progression was a striking observation in the modular structure.

The response behaviours of both the modular and conventional structures were satisfactory under seismic events with around a 500-year hazard return period. However, the modular structure failed abruptly, resulting in overturning of a massive block of the superstructure when the seismic intensity was that of an earthquake with a 2,500-year hazard return period. This brings into doubt the safety of modular structures in terms of adhering to conventional design codes. Conventional building structures are able to demonstrate the ductility and reserved capacity under increasing seismic loads beyond the design limit, as per the building design codes. Current study showed a completely different failure mechanism in the modular structures beyond the design limit, rendering the reliance on building codes unsafe

4) *Development of numerical models (of control model) to simulate the modular building collapse response and investigate the factors exacerbating and mitigating the collapse behaviour.*

This objective was set to study the response of the free-standing stack of modules (assumed as a rigid body) formed. As the first step, the observed rocking sliding and overturning of the modular building superstructure was simulated precisely using numerical models. The numerical model was successfully validated with the experimental observations of the free-standing stack of modules (that started from rest and did not overturn) that was analysed on the shaking table under objective 2.

Investigation of the overturning revealed that the free-standing block of the structure (formed as a result of the failure of connections) overturned despite the ground motion not exceeding half the width of the structure, which is one of the requirements for a free-standing rocking structure to overturn under a ground motion. The possibility of a free-

standing rocking body to overturn under a ground motion without even meeting one of the primary requirements is another highly significant discovery of this study.

Experimental and numerical analysis conducted to understand this phenomenon revealed that the overturning occurred because of the initial conditions attained by the free-standing body right at the beginning of the free-standing motion. These initial conditions were a result of the failure of connections. Specifically, the kinetic energy was elevated in the free-standing body by the strain energy released from the failure of the connections, and this led to the overturning of the free-standing superstructure. This important finding is not just useful for understanding the behaviour of modular structures but applies to any free-standing rocking motion under dynamic loadings. It was further verified by developing another modular structure with connections that had half the strength of the original connections. Duplicating all analysis on this new model showed no overturning of the superstructure component as the connections did not have sufficient strain energy, unlike in the original model. Hence, this study concludes that researchers and designers should look for and consider energy dissipation mechanisms during the failure of connections. Dissipating the energy effectively may protect the structure from catastrophic collapse modes like overturning.

The sliding of the superstructure observed during experimental and numerical analyses is exciting and worth highlighting; again, this phenomenon is unique to modular construction. It occurred in the modular structure because it had a definite failure plane. This is not likely to occur in a conventional structure. Hence, this study contributes to the modular research literature by identifying the potential for such a failure mode in modular structures.

Investigations on numerical models revealed that the sliding of the free-standing body was advantageous in terms of dissipating some energy through friction. However, unlike in a general case of slide and rock, the superstructure component formed in this study did not have an infinite slide plane. The sliding and rocking took place on top of the ground storey modules. Hence, excessive sliding may have resulted in the superstructure overturning due to moving out of the slide plane. Further, excessive sliding (out of the slide plane) may have caused a reduction of the lever arm for the restoring moment, hence the restoring moment, resulting in premature overturning. The effect of friction between the sliding surfaces is more related to the sliding. While a parametric analysis identified a safe range for the coefficient of friction between the sliding surfaces, a smaller coefficient of friction

may not always guarantee safety from overturning, because it might lead to too much sliding and move out of the plane to overturn.

Analysis of numerical models revealed that the presence of roughness along the slide plane, which might act as a sudden halt to the sliding motion, may be disadvantageous in terms of exacerbating the overturning potential. However, the location of the halt from the start of sliding motion also had an effect on the overturning potential.

Further investigation using numerical models disclosed that the free-standing body's flexibility might have some beneficial effect in terms of mitigating the overturning potential, although this was shown to be minor. Nonetheless, this could open up a new research direction for researchers wishing to develop seismic resilient modular systems. Studies could be carried out to incorporate more flexibility into the structure so that the failure would be distributed throughout the structure. Moreover, in the worst-case scenario, if a failure similar to that demonstrated through shaking table experiments occurred in reality, the flexibility of the structure may protect the structure from overturning.

Having summarised findings regarding the response behaviour of the modular buildings under intensifying dynamic loadings, it was imperative also to consider what sort of ground motion was relevant, and how likely it was to occur. IDA curves showed that the sudden increase in damage and the overturning type of failure was observed for a seismic intensity corresponding to a 2,500-year hazard return period. This is almost double the design seismic intensity. Further, ground motion generation models like CAM were employed to generate a likely scenario for south-east Australia, which showed that the intensity of interest is similar to an M5.5 earthquake source occurring within 10 km of the site. This implies that any modular structure within the 300-km² area surrounding the source of such an earthquake is likely to experience a collapse akin to that demonstrated in this study.

Overall, this study has highlighted the danger that may be involved with pure modular buildings (because of the discrete nature of the construction) when they undergo seismic events beyond the design limit. One of the major conclusions of the study is that mere adherence to conventional design codes may not guarantee a safe performance of a modular structure if it experienced a seismic event beyond the design limit. Although the structure would perform safely within design limits, it is a general requirement for a design code to ensure the safe performance of a structure under any event beyond the design limit. While

conventional building design codes do ensure safe performance of conventional structures beyond design limits, extending those provisions for modular buildings would be unsafe.

- 5) *Coming up with recommendations (based on the experimental and numerical results of the study) to develop safer pure modular structures.*

This study's most significant contribution is the identification of the hazardous nature of collapse involved in pure modular structures built as per conventional design requirements. More importantly, the study investigated in depth the cause of the collapse and the factors that exacerbate it. Hence, the last objective was set to propose possible mitigation measures based on the identified causes of the collapse.

This study specifically focussed on four mitigation measures. It was concluded during the study that the failure of first storey edge connections triggered the failure of all other connections of the same storey. Hence, it was proposed to avoid the failure of connections by avoiding excessive stresses (exceeding the design limits of connections) in the connections. Based on the evidence from literature and other special features of modular buildings, it was suggested to allow controlled rocking at the foundation level to dissipate any excessive energy input to the structure during dynamic events.

The study also concluded that most of the connections used in current modular building practice are bolted connections. It was also identified that bolted connections tend to behave in a brittle manner, post-elastic conditions. Hence, the study also came up with recommendations to develop bolted connections restricting the splice action. The restriction of splice action was also intended to increase the ductility and strength of the connection.

Under objective 1 of this study, it was highlighted that the modular buildings have discrete strength and stiffness variation along the height of the structure. This was identified as the main reason for the inelasticity concentration and failure of the connections. Considering this, a recommendation was put forward to attempt to distribute the strength and stiffness throughout the structure. Moreover, suppose the failure could be forced into the modules instead of the connections. In that case, a ductile failure of the whole structure could be expected due to many ductile components inside the modules undergoing failure.

As the final recommendation, a novel modular configuration was proposed. This configuration which involved vertically installed modules, aimed to possess a higher lateral load resisting capacity and avoid the failure of connections along one plane.

8.2 Limitations and Recommendations for Future Research

This study investigated in depth the possible modes of failure involved within a modular structure and the reasons for each of the stages of failure. This study thus represents a much-needed step towards a future for modular construction. The study stands as a guide for researchers in developing new innovative connections or safer pure modular structures. The study examined in detail every phase of the collapse observed, which will help future researchers overcome the problems highlighted in this study and move towards safer pure modular systems. Although it was beyond the scope of this study to come up with recommendations for safer pure modular systems, Chapter 7 is dedicated to discussing approaches that may avert disproportionate collapse, as demonstrated in this study. Future researchers are encouraged to investigate the proposed approaches to develop safer pure modular systems.

One of the main limitations of this study is that it focussed on only one particular structure. Considering the research gap regarding the collapse behaviour of modular structures under dynamic events, it is essential to thoroughly understand the collapse behaviour of modular structures; thus, given time constraints, the study focus was on a single chosen structure. However, the choice of the structure was made to enable generalisation of findings to most modular structures. A mid-rise structure was chosen because it is more likely that pure modular structures of this height will be built before they are expanded to high-rise structures. However, it is recommended that future researchers investigate variation in collapse behaviour when structure height is varied. The aspect ratio of the structure in this study was chosen to be 1:5. Intuitively, such a slender structure presents the worst-case scenario. Hence the study results may be expanded to other structures easily, although an accurate understanding of the likely response of any considered structure can only be obtained through a specific analysis of that structure.

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Appendices

Appendix A: Classification of structural systems in buildings

Table A-1 Classification of structural systems in conventional buildings [65, 66, 86]

Category	Sub-Category	Material / Configuration	Efficient Height Limit	Advantages	Disadvantages	Building Examples
Rigid Frames	—	Steel	30	Provide flexibility in floor planning. Fast construction.	Expensive moment connections. Expensive fire proofing.	860 & 880 Lake Shore Drive Apartments (Chicago, USA, 26 stories, 82 m), Business Men's Assurance Tower (Kansas City, USA, 19 stories), Seagram Building, 30th to the top floor (New York, USA, 38 stories, 157 m)
		Concrete	20	Provide flexibility in floor planning. Easily moldable.	Expensive formwork. Slow construction.	Ingalls Building (Cincinnati, USA, 16 stories, 65 m)
Braced Hinged Frames	—	Steel Shear Trusses + Steel Hinged Frames	10	Efficiently resist lateral loads by axial forces in the shear truss members. Allows shallower beams compared with the rigid frames without diagonals.	Interior planning limitations due to diagonals in the shear trusses. Expensive diagonal connections.	Low-rise buildings
Shear Wall / Hinged Frames	—	Concrete Shear Wall + Steel Hinged Frame	35	Effectively resists lateral shear by concrete shear walls.	Interior planning limitations due to shear walls.	77 West Wacker Drive (Chicago, USA, 50 stories, 203.6 m), Casselden Place (Melbourne, Australia, 43 stories, 160 m)
Shear Wall (or Shear Truss) - Frame Interaction System	Braced Rigid Frames	Steel Shear Trusses + Steel Rigid Frames	40	Effectively resists lateral loads by producing shear truss - frame interacting system.	Interior planning limitations due to shear trusses.	Empire State Building (New York, USA, 102 stories, 381 m), Seagram Building, 17th to 29th floor (New York, USA, 38 stories, 157 m)
	Shear Wall / Rigid Frames	Concrete Shear Wall + Steel Rigid Frame	60	Effectively resists lateral loads by producing shear wall - frame interacting system.	Interior planning limitations due to shear walls.	Seagram Building, up to the 17th floor (New York, USA, 38 stories, 157 m)
		Concrete Shear Wall + Concrete Frame	70	"	"	311 South Wacker Drive (Chicago, USA, 75 stories, 284 m), Cook County Administration Building, former Brunswick Building (Chicago, USA, 38 stories, 145 m)
Outrigger Structures	—	Shear Cores (Steel Trusses or Concrete Shear Walls) + Outriggers (Steel Trusses or Concrete Walls) + (Belt Trusses) + Steel or Concrete Composite (Super) Columns	150	Effectively resists bending by exterior columns connected to outriggers extended from the core.	Outrigger structure does not add shear resistance.	Taipei 101 (Taipei, Taiwan, 101 stories, 509 m), Jin Mao Building (Shanghai, China, 88 stories, 421 m)

Category	Sub Category	Material / Configuration	Efficient Height Limit	Advantages	Disadvantages	Building Examples
Tube	Framed Tube	Steel	80	Efficiently resists lateral loads by locating lateral systems at the building perimeter.	Shear lag hinders true tubular behavior. Narrow column spacing obstructs the view.	Aon Center (Chicago, USA, 83 stories, 346 m)
		Concrete	60	"	"	Water Tower Place (Chicago, USA, 74 stories, 262 m)
	Braced Tube	Steel	100 (With Interior Columns) – 150 (Without Interior Columns)	Efficiently resists lateral shear by axial forces in the diagonal members. Wider column spacing possible compared with framed tubes. Reduced shear lag.	Bracings obstruct the view.	John Hancock Center (Chicago, USA, 100 stories 344 m)
		Concrete	100	"	"	Onterie Center (Chicago, 58 stories, 174 m), 780 Third Avenue (New York, USA, 50 stories, 174 m)
	Bundled Tube	Steel	110	Reduced shear lag.	Interior planning limitations due to the bundled tube configuration.	Sears Tower (Chicago, USA, 108 stories, 442 m)
		Concrete	110	"	"	Carnegie Hall Tower (New York, USA, 62 stories, 230.7 m)
	Tube in Tube	Ext. Framed Tube (Steel or Concrete) + Int. Core Tube (Steel or Concrete)	80	Effectively resists lateral loads by producing interior shear core - exterior framed tube interacting system.	Interior planning limitations due to shear core.	181 West Madison Street (Chicago, USA, 50 stories, 207 m)
Diagrid	–	Steel	100	Efficiently resists lateral shear by axial forces in the diagonal members.	Complicated joints.	Hearst Building (New York, USA, 42 stories, 182 m), 30 St Mary Axe, also known as Swiss Re Building (London, UK, 41 stories, 181 m)
		Concrete	60	"	Expensive formwork. Slow construction.	O-14 Building (Dubai)
Space Truss Structures	–	Steel	150	Efficiently resists lateral shear by axial forces in the space truss members.	Obstruct the view. May obstruct the view.	Bank of China (Hong Kong, China, 72 stories, 367 m)
Superframes	–	Steel	160	Could produce supertall buildings.	Building form depends to a great degree on the structural system.	Chicago World Trade Center (Chicago, USA, 168 stories, Unbuilt)
		Concrete	100	"	"	Parque Central Tower (Caracas, Venezuela, 56 stories, 221 m)
Exo-skeleton	–	Steel	100	Interior floor is never obstructed by perimeter columns.	Thermal expansion / contraction. Systemic thermal bridges.	Hotel de las Artes (Barcelona, Spain, 43 stories, 137 m)

Appendix B: Numerical methods of evaluation of dynamic response

Time stepping methods

Consider a nonlinear equation to be solved,

$$m\ddot{u} + c\dot{u} + ku = -m\ddot{u}_g \quad \text{Eq 2}$$

Following conditions at the initial state,

$$\begin{aligned} u_0 &= u(0) \\ \dot{u}_0 &= \dot{u}(0) \end{aligned} \quad \text{Eq 3}$$

The applied force $F(t)$ is specified at discrete time intervals as shown in Figure 2-71 and the time increment

$$\Delta t_i = t_{i+1} - t_i \quad \text{Eq 4}$$

can be assumed as constant. If the response is determined at the time t_i , is called i^{th} step displacement; velocity and acceleration at the i^{th} step are denoted by $u_i, \dot{u}_i, \ddot{u}_i$ respectively. The displacement, velocity and acceleration are assumed to known, satisfying Eq 2-8 as

$$m\ddot{u}_i + c\dot{u}_i + ku_i = m\ddot{u}_{g_i} \quad \text{Eq 5}$$

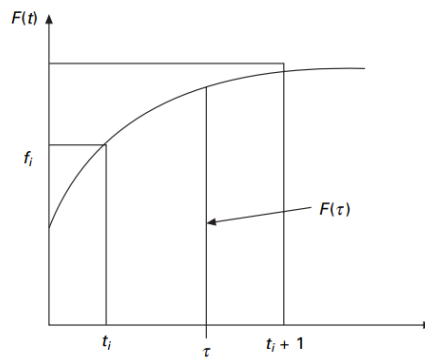


Figure B-1 Piecewise linear interpolation of forcing function [180]

The " ku_i " term on the left-hand side has solutions giving the resisting force at time t_i as f_{si} , for linear elastic systems. But, for inelastic systems, it would depend on the previous displacement history and velocity at the time 'i'. The response quantities $u_{i+1}, \dot{u}_{i+1}, \ddot{u}_{i+1}$ at $i+1^{\text{th}}$ step could

be determined using a numerical procedure. The dynamic equilibrium equation at $i+1^{\text{th}}$ step is written as

$$m(\ddot{u}_{i+1}) + c(\dot{u}_{i+1}) + k(u_{i+1}) = m\ddot{u}_{g_{i+1}} \quad \text{Eq 6}$$

The time-stepping procedure could be employed to obtain the responses at each time step by applying the numerical procedure successively with $i = 0, 1, 2, \dots$, starting from the initial conditions u_0 and $\dot{u}_0$. The time-stepping method cannot be considered as an exact method. Convergence is essential in any numerical method to obtain results for the full-time history, and such a converging numerical method will have the following properties:

- Convergence –The procedure is expected to converge to an exact solution when the time step is recorded.
- When numerical rounding errors are encountered, the procedure is expected to be stable.
- The time-stepping procedure followed should not produce results deviating too much from that of the exact solution.

Central difference method

In this subcategory of the time-step method, the equation of motion is directly integrated following a step by step method. This procedure relies on a finite difference approximation of the time derivatives of displacement (i.e., velocity and acceleration) [89]. Considering constant time intervals, $\Delta t_i = \Delta t$, the velocity and acceleration for time step ‘i’ can be expressed in central difference expression format as,

$$\dot{u}_i = \frac{u_{i+1} - u_{i-1}}{2\Delta t} \quad \ddot{u}_i = \frac{u_{i+1} - 2u_i + u_{i-1}}{(\Delta t)^2} \quad \text{Eq 7}$$

Replacing the corresponding terms in Eq 2-8 with Eq 2-10, give

$$m\left(\frac{u_{i+1} - 2u_i + u_{i-1}}{(\Delta t)^2}\right) + c\left(\frac{u_{i+1} - u_{i-1}}{2\Delta t}\right) + ku_i = m\ddot{u}_{g_i} \quad \text{Eq 8}$$

In Eq 2-11, u_i and u_{i-1} are considered known. Known displacements u_i and u_{i-1} are used to compute u_{i+1} . Thus u_0 and u_{-1} are required to determine u_1 ; the specified initial displacement u_0 is known. These steps are repeated at each time increment for each $\ddot{u}_{g_i}$ value to calculate the resulting displacement value at each time step.

Runge-Kutta method

Runge-Kutta method is a single step method since it requires the value of only x_i to evaluate x_{i+1} . There have been many orders in the evolution of the Runge-Kutta method, of which the 4th order method is the most commonly used. This method is one of the most stable methods compared to the central difference method and many other methods introduced. Runge-Kutta method is explained below for the case of a single degree of freedom model under dynamic excitation [89].

$$m\ddot{u} + c\dot{u} + ku = -m\ddot{u}_g \quad \text{Eq 9}$$

Assume $v = \dot{u}$, therefore Eq 2-12 can be expressed in terms of two 1st order equations as:

$$\dot{v} = \ddot{u} = \frac{[-m\ddot{u}_g - c\dot{u} - ku]}{m} \quad \text{Eq 10}$$

$$\dot{u} = v \quad \text{Eq 11}$$

The Runge-Kutta recurrence formulae for u_{i+1} and v_{i+1} respectively are given as

$$u_{i+1} = u_i + \frac{\Delta t}{6} (\dot{u}_1 + 2\dot{u}_2 + 2\dot{u}_3 + \dot{u}_4)$$

$$u_{i+1} = u_i + \frac{\Delta t}{6} (v_1 + 2v_2 + 2v_3 + v_4) \quad \text{Eq 12}$$

$$v_{i+1} = v_i + \frac{\Delta t}{6} (\dot{v}_1 + 2\dot{v}_2 + 2\dot{v}_3 + \dot{v}_4) \quad \text{Eq 13}$$

Eq 2-15 & Eq 2-16 are the equations for the average of velocity and acceleration, respectively, within the time interval Δt .

Appendix C: Derivation of equation of motion for rocking of a flexible body under dynamic equations

The flexible rocking block is assumed to behave as a column element with a consistently distributed mass (m_c) and stiffness (EI) of the column as illustrated in Figure C-1 below. Two other lumped masses are placed at the top (m_t) and base (m_b), while the moment of inertia (I_b) is considered around the centre of mass. Total base width is defined as $2B$. likewise, to rigid bodies, the slenderness is: $\alpha_{sl} = a \tan(\frac{B}{H})$, and the surface is considered to be rigid. The flexural body under rocking can be considered as a dynamic system of two degrees of freedom (DOFs). Base rotation (θ) is considered the first DOF and the horizontal displacement of the top relative to the bottom of the column (u) is considered as the second DOF.

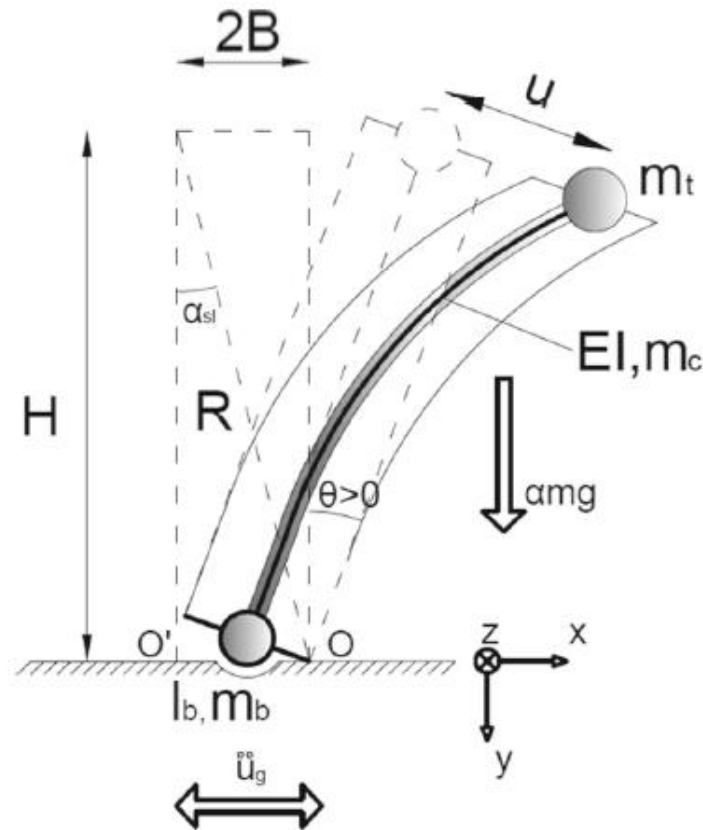


Figure C-1 Flexible rocking block model

In the presence of uplift, the rocking behaviour is described by two DOFs (θ, u). The equations of motion shown below can be derived by applying Lagrangian formulation.

$$\begin{aligned}
& \left(I_B + m_c \left(B^2 + \frac{H^2}{3} \right) + \frac{33}{140} m_c u^2 - \text{sign}(\theta) \frac{3}{4} B u m_c + m(B^2 + H^2) - \text{sign}(\theta) 2 B u m + m u^2 \right) \ddot{\theta} \\
& = - \left(\frac{33}{70} m_c + 2m \right) u \dot{u} \dot{\theta} - \left(\frac{11}{40} m_c + m \right) H \ddot{U} + \text{sign}(\theta) \left(\frac{3}{4} m_c + 2m \right) B \dot{U} \dot{\theta} \\
& + \ddot{U}_g \left(-\text{sign}(\theta) (m + m_c + m_b) B \sin(\theta) - \left(\frac{m_c}{2} + m \right) H \cos(\theta) + \left(\frac{3}{8} m_c + m \right) u \sin(\theta) \right) - - - - - (\mathbf{A}) \\
& + g \left(-\text{sign}(\theta) (m + m_c + m_b) B \cos(\theta) + \left(\frac{m_c}{2} + m \right) H \sin(\theta) + \left(\frac{3}{8} m_c + m \right) u \cos(\theta) \right) \\
& \left(\frac{33}{140} m_c + m \right) \ddot{U} + \left(\frac{11}{40} m_c + m \right) H \ddot{\theta} = \\
& - \frac{3EI}{H^3} U - C \dot{U} + \left(\frac{3}{8} m_c + m \right) g \sin(\theta) + \left(\frac{33}{140} m_c + m \right) u - \text{sign}(\theta) \left(\frac{3}{8} m_c + m \right) B \dot{\theta}^2 - \left(\frac{3}{8} m_c + m \right) \ddot{U}_g \cos(\theta) (\mathbf{B})
\end{aligned}$$

The post-impact velocity in the case of change of a pivot can be calculated using the equation ‘C’ below, considering the conservation of angular momentum theory. The relative horizontal velocity is assumed to be same after and before impact. Hence, $u_2 = u_1$.

$$\begin{aligned}
\dot{\theta}_2 = & \frac{(I_{bc} - c B^2 m_b) + m_c \left(-B^2 + \frac{H^2}{3} + \frac{33}{140} u_1^2 \right) + m_t (-B^2 + H^2 + u_1^2)}{I_{bc} + m_c \left(B^2 + \frac{H^2}{3} + \frac{33}{140} u_1^2 \pm \frac{3}{4} B u_1 \right) + m_t (B^2 + H^2 + u_1^2 \pm 2 B u_1)} \dot{\theta}_1 - - - - - \\
& - - v \mathbf{C})
\end{aligned}$$