

STRUCTURAL BEHAVIOUR OF MODULAR BUILDING**Safeek.A**Dhanalakshmi Srinivasan College of Engineering, Coimbatore, Tamil Nadu, India
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Abstract

Prefabrication through off-web website online production leads to a decreased basic production schedule, progressed quality, and decreased resource wastage. Modular constructing is consequently an increasing number of famous and promoted. With the latest advertising a wide variety of applicable research were finished, however, a assessment of the layout, production, and overall performance of modular homes beneath specific loading situations is lacking. This paper provides a contemporary assessment of modular constructing systems. First ,structural bureaucracy and production substances are supplied as a quick creation to the modular systems. Modular constructing is proven to refer now no longer to a unmarried shape kind, however quite a few structural structures and substances. These modular systems may carry out in a different way to comparable conventional systems and the structural overall performance is highly dependent on inter- and intra -module connections . The structural reaction of modules to specific dangers is then taken into consideration, accompanied through the contemporary layout exercise and methodology. As a presently growing place there may be terrific cap potential for innovation in modular systems and numerous key studies regions are diagnosed for in addition paintings.

1. Introduction

Modular building is a construction technique whereby building modules are prefabricated off-site. It is a type of off-site fabrication referring specifically to volumetric units which may be a structural element of a building. Modular building refers to the application of a variety of structural systems and building materials, rather than a single type of structure. Prefabrication by off-site manufacturing leads to a reduced overall construction schedule, improved quality, and reduced resource wastage . The disadvantages include the lack of design guidance and relatively small structural spans due to modular transport limits. The advantages of modular building outweigh the disadvantages particularly for hotel and residential development applications. Modular building is therefore increasingly popular and promoted. With the recent promotion a number of relevant studies have been conducted. This paper presents a state-of-the-art review of modular building structures. First, recently developed structural forms and construction materials are presented as a brief introduction to the modular structures. The focus is on steel framed modules with concrete and timber frame modules excluded, not for lack of importance, but for lack of recent research into

the structures. Structural connections are the key to overall performance and so a detailed review of connection types is presented. Then, the structural response of modules to different hazards is considered, followed by the potential applications and future research work.

1. Module classification and developments in structural form

Modules are classified as steel, precast concrete and timber frame modules according to the primary construction material. Steel modules are further classified as Modular Steel Building (MSB) modules, light steel framed modules and container modules. Their applications, advantages and disadvantages are given in. Load bearing steel modules are also categorised as column supported or continuously supported. Column supported modules have edge beams which span between corner or intermediate columns. Continuously supported modules have load bearing walls which provide continuous support. Recent study of modular building was focussed on light steel framing applied to modular buildings followed by consideration of overall building design using modules and then high-rise building applications. A broad overview of modular construction using light steel framing was given with the application of relevant British and European standards. Several types of modular construction have been presented. Many modular buildings are not exclusively modular but are hybrid structures. Modular construction is combined with a primary steel or concrete structure, for example, using a podium or skeletal structure, or a concrete core around which modules are arranged. Column supported MSBs are well suited to medium and high-rise building applications and popularly used in current practice.

2. Component materials

Prefabricated components should be as light as possible as they are transported sometimes long distances. Traditional materials of steel, concrete and timber are commonly used. The potential applications of composite sandwich structures have not been well explored. Manalo et al. gives an overview of fibre reinforced Polymer (FRP) sandwich systems in the context of lightweight civil infrastructure. Many of the developing materials presented by Manalo et al. may find application in modular buildings. To date such composite systems have been developed for application as roof, wall or floor components. Some examples of composite Use of FRP composite materials for complete modular building façades is appealing although their performance, especially with respect to wind-borne debris impact and fire, is a

developing area. In addition, the design and manufacture of reliable jointing systems is noted as a challenge which has received growing research attention

3. Connection systems

Interconnection of frame members and modules is critical to the capability of modular buildings to withstand applied loads [45]. Despite the need for a thorough understanding, studies on the connections are limited [45]. Connections are grouped into three types: inter-module, intra-module, and module to foundation as shown in Fig. 4. Table 3 provides a summary of connections for steel

modules with further details given in the following Sections. Inter-module connection Inter-module connection includes horizontal connections (HC) between adjacent modules in two plan directions, and vertical connection (VC) between stacked modules. It is reported that bolted connection is preferred over site welding. A gap is usually provided between the floor and ceiling beams, as shown in Fig. 1(L), allowing for external access to inter-module connections and for services to pass between the beams. This suits connection between the columns, rather than between the beams. Bolted connections can be complex to accommodate connection of modules stacked in three directions while ensuring access to fasteners is provided during the install sequence. Use of long slotted holes may introduce the potential for tolerance accumulation over multiple levels, and vulnerability to slip failure in the event of large horizontal force. The potential for connection slip may be controlled with the use of friction-grip or pre-tensioned bolts. VCs may incorporate a shear key or spigot [46], which assists in positioning modules and may provide structural connection where physical access is not possible or practical. In some cases, concrete or grout is used to lock the joint in place, creating a composite concrete-steel connection. Table 4 presents a summary of inter-module connections for steel modules from the literature, and identifies the numerical and experimental investigation completed. In the latest studies, force-displacement (F-d) and moment-rotation (M- θ) behaviours of the connection are established by detailed numerical analysis of the connection.

3.1. Intra-module connection

Intra-module connections, i.e. connections within a module, are generally representative of traditional connection details. For MSBs both welded and bolted connections are used. Considering column to beam connections, the bolted connection types include single web (or fin) plates double angle cleats [4] and bolted end plates. It is suggested that a moment resisting connection consisting of an end plate or a deep fin plate may provide lateral stiffness for low rise buildings. This is unusual in that fin plate connections are often classified as simple shear connections. Fin plate connections have relatively low moment capacity, ductility and rotation capacity, hence their use is suggested only for low rise (3-storey or less) buildings. However, the use of such connections may make open modules susceptible to progressive collapse. In this case the fin plate connection may have inadequate moment capacity, and so require strengthening. In contrast, Annan et al. investigated steel floor framing with secondary beams welded directly to the main beams. This is compared with conventional steel construction which may use clip angles permitting greater rotation. The welded connections do not necessarily permit rotation such that steel members should be designed for hogging moments and axial forces which may be developed as a result. Linear elastic analysis is demonstrated to be adequate for this issue and Annan et al. present a process which may be adopted in design.

3.2. Module to foundation connection

Foundations may consist of in situ or precast concrete footings, bored concrete piles, augered steel piles, or some combinations. Low rise modular buildings located in areas with high lateral loading may be vulnerable to overturning and sliding failures if not adequately restrained by connection to an appropriate foundation. Building modules are commonly connected by chains, cables, keeper

plates or welding to concrete or steel piles, or large mass concrete footings. Each connection type has associated disadvantages including tensioning requirements for chain and cable. In medium and high-rise construction foundations are more substantial. Base plates may be incorporated in modules and fixed to cast-in anchors, or welded on site to accessible cast-in plates. Park et al. [45] developed an embedded column connection (Fig. 5), as an alternative to the traditional cast-in or post-fixed steel bearing plate. This connection was developed to ensure best use of the full column strength and provide good ductility. The disadvantages include the requirement for site welding between MSB columns and the end plate.

4. Case study

To define the range of existing modular buildings, a list of multi-storey modular building projects has been compiled based on a review of the literature. Table 5 shows a selected sample. The tallest identified prefabricated building is J57 Mini Sky City located in Changsha, China, being 57 storeys or 207.8m tall. In Australia, the tallest prefabricated building identified is La Trobe Tower, Melbourne, being 44 storeys or 133m tall. It is in an area with relatively low wind and earthquake loading. In areas with higher lateral load requirements the maximum building height is less. For example, the tallest building within Australia's severe cyclone region is Concorde South, being 6 storeys.

5. Hazards and structural responses

The hazards for modular buildings include transport activity, cyclone (hurricane, tornado), earthquake, explosion, progressive collapse, and fire. Hazards can be classified as natural, anthropogenic, or technological and the interaction can be concurrent, cascading or independent [60]. Each hazard has associated actions, or loads, for which the building structure must be designed. Different loads have different characteristics, design criteria and mitigation strategies (Table 6). No studies have been identified to address the method for selection of an optimal design to suit the competing requirements of different loads for modular building.

5.1. Transport and handling

Modules are designed to be lifted, with designated lift points provided on the module. They are usually lifted by a crane, although forklifts may be used in the manufacturer's yard. The number and positioning of lift points is often determined by deflection criteria chosen to protect fragile components. Lift points are typically positioned in from each end, reducing member design actions and deformation of the chassis. General guidance on the design of lifters is available in existing documents, for example, American Standard [65-67] and DNV Offshore Standard [68]. The stiffness of a whole module is likely to be influenced by internal and external

linings and manufacturers may test complete modules to establish composite stiffness. For transportation, including road, rail and sea, the loading is defined by acceleration coefficients, for example 0.8g forward, 0.5g rearward, and 0.5g transverse during road transportation. General

guidance is available in the existing documents, for example, the CTU Code [69] for cargo transport units (CTU). Transit bracing is often provided, particularly in open modules, to strengthen the structural frame and increase stiffness to help control damage to fragile components. Few studies on the response of modular buildings to transport and handling have been reported. Smith et al. [70] investigated transport and handling for single storey timber framed modules. Field testing and numerical modelling using SAP2000 were conducted and the relevant data is provided in the referenced report. It was reported that the main form of damage was cracking of internal plasterboard linings. This was caused by the lifting practices with road transport propagating cracks. It was recommended that areas for further work included the development of laboratory techniques to simulate transport under controlled conditions and the development of dampers to reduce dynamic forces.

5.2. Wind

Natural hazards such as cyclones and tornadoes encompass multiple actions including both primary wind loading, and secondary debris impact and water ingress. Wind loads are characterised by low frequencies, approximately 0.01 to 2.5 Hz, and mean and fluctuating components [71-73]. Design criteria are typically based on stability, strength and serviceability by considering inter-storey deflection, overall deflection, and vibration. For example, the limits of $H/600$ for total building drift and $h/500$ for inter-storey drift [61] are specified, where H is the total building height and h is the storey height. Generally, structures are regarded as wind sensitive if the fundamental frequency is less than 1 Hz, and slenderness ratio is greater than five. Static analysis is therefore appropriate for buildings with height less than 50 m [74], based on an empirical formula for the fundamental frequency [76, 77]. However, results presented in the literature for modular buildings suggest a value of 30 m could be more appropriate, with a frequency of approximately 1 Hz obtained numerically for a 10-storey modular building [17] (refer Section 6.3.3). Therefore, dynamic analysis is required for modular buildings over 30 m high. No studies are identified to address cyclonic wind loads for modular buildings. Few studies on the response of modular structures to wind loading have been conducted. Gunawardena et al. presented base shear and storey drift results for static analysis of a 10-storey modular building with wind loading applied following AS 1170.2:2011 [62] for Region A, Terrain Category 4. Three different cases of inter-connection were considered for this building – rigid connection by a rigid floor diaphragm, flexible connection by a semi-rigid diaphragm, and no connection. The results indicated the actual structural behaviour fell between that for the semi-rigid diaphragm and no diaphragm. Styles et al. [47] investigated the effect of joint rotational stiffness on the response to wind loading for an 11-storey modular building. It was reported that increasing horizontal inter-module and intra-module connection stiffness effectively reduced inter-storey drift due to wind load. Intra-module connection stiffness was shown to have a greater effect than horizontal inter-module connection stiffness. It is indicated that further studies are needed to address the factors including vertical inter-module connection stiffness and building height.

5.2.1. Wind-borne debris

Wind-borne debris impact has been observed in many strong wind events and is well documented in the existing literature [79-81]. Debris impact causes localised damage and if the envelope is penetrated may cause additional damage due to water ingress and internal pressurisation. Debris impact loading is typically specified

for standard debris items, with velocity related to regional wind speed (VR). For example, AS/NZS1170.2 [62] specifies a 4kg timber with 100x50mm cross-section impacting at 0.4VR and 0.1VR, and an 8mm (2g) steel ball impacting at 0.4VR and 0.3VR for the vertical and horizontal trajectory component respectively [62]. Modular buildings are generally lightweight to facilitate transport and so may incorporate lightweight cladding materials which are vulnerable to debris impact. Recently, a systematic study of the performances of structural panels against wind-borne debris impact has been conducted experimentally by using a pneumatic cannon and numerically by using LS-DYNA [82-86] and some cost-effective engineering adaptation methods have been developed. The development of lightweight and resilient structural panel as building envelope can be incorporated into the design of modular buildings.

5.3. Earthquake

Earthquakes are characterised by ground motions with predominant frequencies in the range of 0.5 to 25Hz, which normally excite the fundamental and low vibration modes of engineering structures and result in global structural responses [71, 72]. Extensive research works have been carried out to investigate the seismic behaviours of traditional building structures, and displacement-based damage criteria such as the ductility ratio and inter-storey drift ratio are widely used by the design guides to measure their seismic performances [72]. For example, Australian Seismic Design Code AS 1170.4 [64] specifies an inter-storey drift limit of 1.5% for

the ultimate limit state. Modular buildings, due to their obvious advantages, are increasingly used in areas with high seismic hazard. Their seismic performances are, however, not adequately understood because they are a relatively new structural form. The rocking and sliding response of modules after connection damage could be the governing response modes. In such cases the conventional displacement-based criteria such as drift ratio may no longer be applicable to quantify modular building damage. Therefore, it is essential to investigate the performance of modular buildings under earthquake loadings and further propose the corresponding criteria to evaluate their seismic performances.

5.3.1. Seismic responses

Very limited studies on the seismic responses of modular structures have been conducted and these studies mainly focused on the MSB-braced frames. Annan et al. [20] conducted experimental studies on the hysteretic behaviours of an MSB-braced frame and a regular concentrically-braced frame with similar physical characteristics. It is reported that both specimens showed stable and ductile behaviour up to very high drift ratio (3.5%). The MSB frame was more vulnerable to column bending deformation, whereas the traditional frame was vulnerable to the out of plane buckling of

bracing. Annan et al. [21] further conducted incremental dynamic analysis (IDA) for 2-, 4- and 6-storey two-dimensional (2D) MSB-braced frames. It is reported that the selected MSB-braced frames exhibit a predominantly first-mode response (e.g. the mass participation factors for the 2-storey frame are 94% for the 1st mode and 5% for 2nd mode, similarly, for the 4-storey frame, the percentages are 81% and 15% respectively, and for the 6-storey frame, the values are 77% and 17%) and limited redistribution of internal forces can result in concentration of inelasticity at the first level. Fathieh and Mercan [22] conducted non-linear static pushover and IDA analyses for 2D and 3D MSB-braced 4-storey frames. It verified the concentration of inelasticity at the first level due to limited internal force redistribution and brace inelasticity, and found the 2D model overestimated the structural capacity against incipient collapse because torsional response was not accounted for in the 2D model. Gunawardena et al. [24] conducted non-linear time history analysis for a freestanding 10-storey modular building subject to six selected ground motions. Column hinge formation was found unavoidable in severe ground motions and column ductility was important to redistribute post yield loads. It was indicated that further studies are needed to investigate dynamic behaviours for mid- to high-rise modular buildings.

5.3.2. Over-strength and ductility

In the seismic analyses of traditional engineering structures, forces derived from an elastic response spectrum analysis are adjusted to allow for the ductility and other reserved strength. AS 1170.4 adopts a structural ductility factor (μ) and structural performance factor (S_p) as a measure of the ability to withstand inelastic displacement and other reserved strength respectively. For the modular structures, Annan et al. [18] investigated the inelasticity of modular buildings by conducting non-linear static pushover analyses for 2-, 4- and 6-storey 2D MSB-braced frames. The over-strength factor ($R_0=1/S_p$) and structural ductility (μ) are reported as per the National Building Code of Canada (NBCC) [87, 88]. Two methods were used to calculate the column actions due to bracing – the SRSS (Square Root of the Sum of the Squares) method [89, 90], and the DS (Direct Summation) method. The SRSS method was found to be unconservative and so the DS approach was recommended. The ductility, which is calculated as the ratio of the ultimate drift (Δu) to the yield displacement (Δy) i.e. $\Delta u/\Delta y$, is shown to reduce with increasing MSB frame height. The values range from 4.6 to 1.8 for 2- to 6-storey MSB-braced frames. The variation with height is reported to occur for two reasons. First, the yield displacement of braces increases with increasing frame height due to a decrease in brace slenderness (i.e. the brace section size is not constant) [18]. Second, the ultimate drift is determined by the redistribution of load after the compression brace yields [18]. The vertical inter-module connections affect this ability to redistribute load, and the affect increases with increasing frame height [18]. Similarly, the over-strength factor, which is calculated as the ratio of the ultimate load to the design load, is shown to reduce with increasing height [18]. The values range from 2.5 to 1.9 for 2- to 6-storey MSB-braced frames. The variation with height is again related to the ability to redistribute internal force [18]. Similar results are reported for steel moment resisting frame (SMRF) modules. Choi et al. [49] conducted non-linear static pushover analyses for 3- and 5-storey SMRF modules with varying inter-module connection type and stiffness, and reported over-strength factors between 2.15 and 3.76 for the 3-storey frames, and between 1.19 and 1.94 for the 5-storey frames. For each frame height, the SMRF frames with greater

inter-connection stiffness had greater over- strength factor.

5.3.3. Fundamental period of modular building

The fundamental period of a modular building is an important parameter for the prediction of its seismic behaviour [91]. Many standards provide empirical formula for the fundamental period (T_1 [s]) of traditional building structures. For example, AS 1170.4 [64] gives $T_1 = 1.25h^{0.75}/k_t$, where h is the height from the base to the uppermost seismic mass in metres, and k_t is a constant depending on the structure type with a value ranging between 0.05 (for all other structures) and 0.11 (for moment-resisting steel frames). Recently, some studies have been carried out on modular buildings. Fig. 6 shows the fundamental periods for rectangular modular buildings together with the upper and lower bounds defined in AS 1170.4. These fundamental periods are the result of numerical analysis of 2D MSB-braced frames [21], 3D MSB-braced frames [17, 22] and steel moment resisting frame (SMRF) modules [49, 92], and experimental analysis of stacked timber frame modules [93]. It can be seen that the period values follow the trend of increasing with building height. From the limited data presented, $k_t=0.05$ gives a reasonable estimate for both the 2D MSB-braced frames and the timber frame modules. $k_t=0.075$ appears more appropriate for the 3D MSB-braced frames. For the SMRF modules greater variation in the fundamental period is shown. This is the result of varying inter-module connection type and stiffness [49], and varying imposed gravity load [92]. For these SMRF modules use of $k_t=0.11$ to estimate the fundamental period is most accurate for the case of rigid inter-module connection. It should be mentioned that the data presented for the MSBs considers only the main structural elements and excludes the cladding elements. Cladding incorporated into modules may have a significant effect on the fundamental period due to the additional mass.

5.4. Blast

Blast loads can induce local and global response. Local response, typically the result of close-in explosion, causes localised failure which might lead to progressive collapse [94]. Global response occurs for transverse load with longer exposure time [95], such as gas explosion and far field explosion. Design is typically based on descriptive building damage level (i.e. low, medium, high) and component response. Modular buildings may be subjected to blast load from several sources including chemical / industrial / terrorist explosion. They are therefore used in industry to protect personnel [96-98] where they are referred to as blast resistant modules (BRM) or blast resistant portable buildings (BRPB) (Fig. 7). Steel modules have a primary steel frame with walls formed by infill between roof and floor. For example, walls may consist of steel plate, cold-formed steel stud frame or sandwich panels. Typically, BRMs use 3.2 to 7.9mm thick crimped steel plate walls fully welded to the steel frame [96]. For this type of module construction, finite element analysis is recommended to capture failure modes, although a single degree of freedom (SDOF) approach may be adopted [96]. For general design, reference is often made to the standards including UFC 3-340-02 [99] and ASCE 41088 [94]. Hao et al. [100] provide a summary of the current practice. For steel components, response limits are given based on ductility ($\mu=1$ to 20), support rotation ($\theta=1$ to 12°)

and side sway (height/50 to $h/25$) for moment-resisting frames. Overall, published qualitative studies for blast loading are limited and further studies are needed to investigate the behaviour of mid- to high-rise modular buildings.

5.5. Progressive collapse

Progressive collapse occurs when local damage from an extreme event results in overload and then failure of successive alternate load paths [101, 102]. It is a relatively rare catastrophic failure which may occur due to lack of continuity, ductility and/or redundancy [101]. The methods of design are typically categorised as indirect or direct approaches. The tie force method is an indirect approach wherein robustness is enhanced by provision of specified minimum ductility, continuity and strength [101, 102]. Direct design approaches include the alternative path method and key element design (or enhanced local resistance) [101, 102]. For modular steel buildings the tie force method may not be appropriate. This method does not consider beam rotations due to catenary action and failure may occur in low ductility connections such as standard fin plates [103].

Similarly, the key element method may not be appropriate for light steel framed modules [13]. This is because the lightweight structure may be unable to resist the relatively large design actions resulting from extreme events such as gas explosion [13]. Instead, the alternative load path approach is adopted [10, 13]. Lawson et al. [13] completed a static analysis for the loss of one corner or internal support to a light steel module, with load being transferred by shear walls back to a tying action in inter-module connections. To demonstrate the capability of the modules, tests were completed to establish the shear wall racking capacity and deflection of the module with removed supports. The required inter-module tie forces were found to be relatively low [13]. In another study, container express (CONEX) modules used for barracks were tested in Florida [104]. The modules were stacked three levels high and connected using twist locks. The work completed included simulating column removal using SAP2000, experimental column removal tests using hydraulic jacks, and finally detonation of an artillery shell at a critical location (Fig. 8). The results confirmed progressive collapse was not an issue as per the Unified Facilities Criteria [101]. Progressive collapse studies have therefore considered only light steel framed modules and container buildings stacked three storeys high. Modular steel buildings may be vulnerable due to the use of connections with limited ductility, which is worth studying.

6.6. Fire resistance

Fire resistance is a major challenge for modular buildings. Concerns have been reported considering the use of flammable materials and the presence of a void between modules which may permit a fire to spread [105, 106]. These concerns are similar to those raised in the case of the fire at Grenfell Tower, London, UK [107]. However, studies into the fire performance of modular structures are limited. One study considered fire simulation for a container building [27] and several studies investigated the performance of developing composite materials such as fibre reinforced polymer (FRP). Composite materials have advantages which make them well suited to application in modular building. They are lightweight with high strength and stiffness and offer flexibility in shape [108-111]. However, they generally have low fire resistance and their performance as a building envelope is not well known [108, 109]. Ngo et al. [108] investigated the performance of an office building constructed from modules with glass FRP composite components forming the façade. This study provides

insight into the fire performance based on numeric analysis. Nguyen et al. [109] investigated the addition of fire retardant to improve fire performance. Models were developed to predict “fire growth index and total heat release” for “organoclay/glass fibre reinforced polymer (GFRP) laminates” [109]. The performance was then considered for an office building with façade consisting of GFRP laminate on foam core. The addition of organoclay is found to prevent flash-over and horizontal flame spread. Fire performance studies are therefore mostly related to the development of FRP materials with no studies identified for modular steel buildings. In current practice, fire resistance time requirements are satisfied by providing layers of protective cladding and fire stops to control fire spread [4].

6. Design guidelines and theoretical analysis

General design guidance is available for light steel framed modules in the existing literature (Table 7). In current practice, guidance is drawn from the traditional literature. A limit state approach is adopted for design criteria considering stability, strength, and serviceability. Building serviceability is satisfied by adherence to overall drift limits and acceleration limits for human comfort. For example, for wind loading, Mendis et al. [74] and Griffis [61] report on acceleration and drift limits. Modules should also satisfy serviceability criteria during transport and lifting. Limited guidance is available and current practice is based largely on practical experience. Serviceability during transport and lifting is mainly concerned with the potential for damage to finishes and equipment. Therefore, guidance can be found in existing literature such as Griffis.

7. Conclusions

Modular building refers to the application of a variety of structural systems and building materials. Modular buildings perform differently to similar traditional structures owing to the requirements of site interconnection for modular building. Analysis, design and construction technologies of modular building are currently under

further development and several key areas are identified for further study.

1. Overall module size is limited by transport capabilities and internal layouts are limited by the requirement for structural elements. Greater flexibility in internal layout could be provided by a reduction in structural member sizes, and reduction in the number of braces, shear walls and columns.
2. There remains a need for efficient lightweight systems which also address issues of robustness. At the same time, there is also a need to develop new lightweight materials for specific application in prefabrication and modular building. The performance with respect to wind-borne debris impact and fire are developing areas.
3. Accumulated damage must be considered for portable buildings subject to repeated and fatigue loading. In other cases where a module might be transported only once or twice in its life, damage during transport and in situ degradation are worth studying. This affects the performance for all hazard types, yet published quantitative studies are limited.

4. Connections are key to the performance of modular buildings. The design and fabrication of reliable connection systems is a major challenge. Further research is required to better define the performance of existing connection systems and limit the potential for overdesign due to limited understanding.

5. The current studies on modular building under earthquake are limited to 10 storeys. Further studies are needed to investigate the dynamic behaviour due to earthquake ground motion for mid- to high- rise modular buildings.

6. No studies have been identified to address dynamic wind loading, or cyclonic wind loading for modular buildings. Further studies are needed to investigate the dynamic response of modular buildings under wind loading.

7. Further studies are needed to investigate the behaviour of mid- to high-rise modular buildings subject to blast loading. In addition, progressive collapse studies considered only light steel framed modules and container buildings. Modular steel buildings may be vulnerable due to the use of connections with limited ductility, which is worth studying.

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