Constructing Steel Modular Buildings with Varying Interconnections

By

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Abstract

Modular construction uses prefabricated building components called modules, which are fabricated in a factory, transported to a site, and then assembled together to create a building. A "module" is one of the building blocks used to construct a modular building and may be comprised of a load bearing structure, MEP components, interior finishes, and exterior cladding. This alternate way of building using prefabricated units leads to advantages such as: faster construction, cost savings, and sustainability benefits.

Historically, modular construction has predominantly been used in the development of low rise, temporary, or portable buildings. However, recently this prefabricated building technology has spread into multi-story applications and a wider range of building types such as hospitals, residential complexes, and schools. As more high-rise buildings are being built using modular construction, new structural challenges must be addressed. Lateral and gravity loads increase with height and the design of building connections and their lateral force resisting systems becomes ever more critical. Although several case studies describing modular buildings are publicly available, there is a lack of detailed scientific data explaining their structural performance. This thesis attempts to shorten the knowledge gap by investigating the effect interconnections have on the behavior of a modular building.

In this study modular interconnections are defined as the connections within modular buildings which link discrete modules together allowing them to act as a single structure. Modular interconnections are a keen area of interest as their design affects the global behavior of a modular building. To understand the effect different interconnections have on the stability of a modular building, a study is conducted where several building prototypes with various interconnections are modelled and analyzed.

Acknowledgements

I would like to thank Professor Gordana Herning for her support, wisdom, and patience this past year at MIT. I could not have asked for a better advisor during my time here. I would also like to thank Antonios Vytiniotis and John Kennedy for their advice and the memorable moments I experienced in their classes.

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Chapter 1. Introduction

Modular construction is a term used to describe the use of pre-fabricated building components that are assembled on site to create a complete building. Figure 1.1 shows the basic steps involved in erecting a modular building, including: manufacturing modules in a factory, delivering them to a site, and assembling them together. Because modular buildings are composed of discrete modules, they must be connected together, using interconnections, so that the entire building behaves as a single structure. The manner in which the modules are connected together will largely impact the global structural behavior of the building. The objective of this work is to investigate different types of interconnections and their impact on the structural behavior of modular steel buildings.

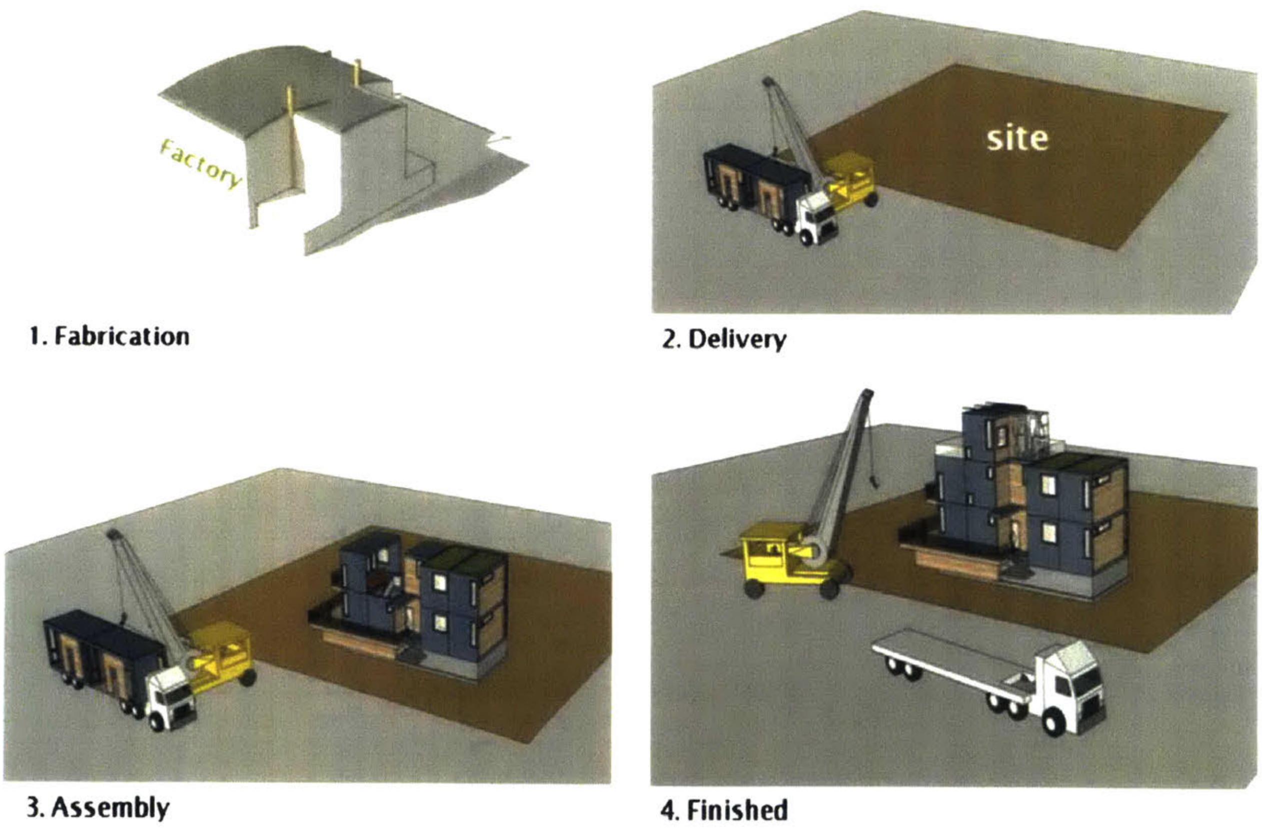


FIGURE 1.1 MODULAR CONSTRUCTION PROCESS (LEE ET AL. 2014)

1.1 Motivation

The global urban population is increasing by 200,000 people per day (World Economic Forum, 2016), all of whom need a place to live and work. It would seem that in the face of such an incredible need for new infrastructure, the construction industry would be constantly evolving to meet this demand. However, while labor productivity for industries which have existed since 1964 in the US has on average increased by 153% over the past half century, productivity in the construction industry has significantly fallen (Figure 1.2). It has been slow to adopt new innovations in technology, material use, and manufacturing processes which have greatly benefited other market sectors. The construction industry would potentially see a tremendous boost in productivity from prefabrication and standardization. Prefabrication reduces project delivery time and construction costs by enabling more productive sequencing in the construction process. Standardizing building components reduces risk and provides a greater opportunity for reuse. Modular construction incorporates both prefabrication and standardization to produce building components in a factory setting in a fraction of the time it would take to assemble them on site.

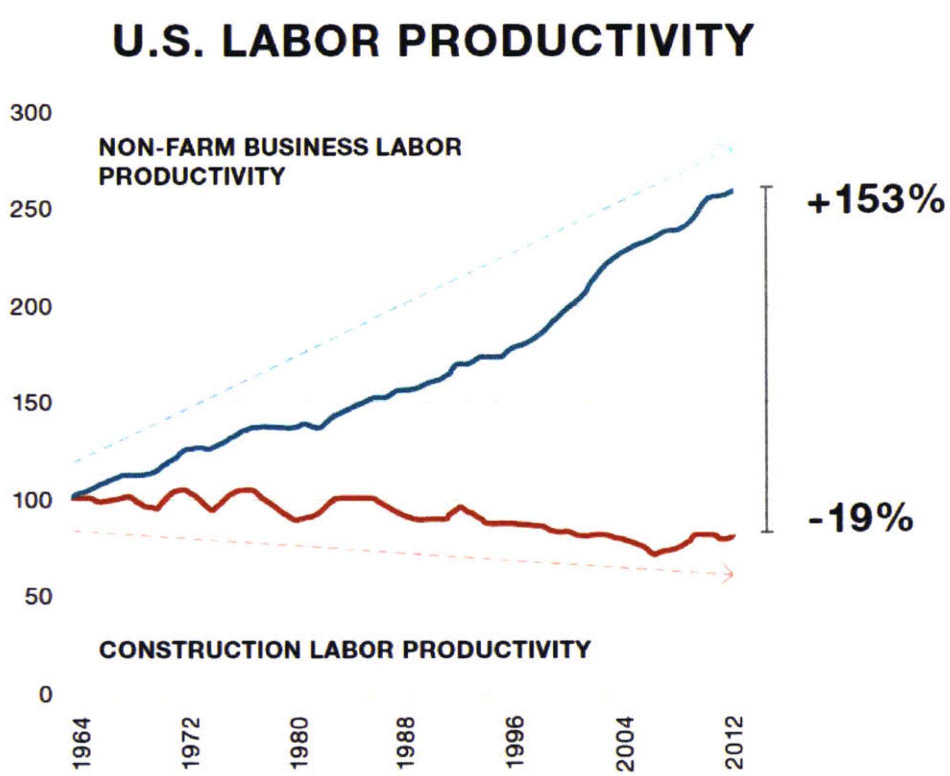


FIGURE 1.2 - US LABOR PRODUCTIVITY (WORLD ECONOMIC FORUM 2016)

Modular buildings provide economic and sustainability benefits over traditionally built buildings such as reduced construction costs and less material waste (described in detail in section 1.1.2). Off-site construction also creates the opportunity for buildings to be assembled faster and with less disturbance to the surrounding neighborhood. According to the Modular Building Institute (MBI), as of 2018 modular construction composed only 3.5% of the construction industry in the United States. However, this represents a 50% increase from 2014 where modular construction represented only 2.3% of the permanent construction market share. The Sage Policy group, (a consulting firm which works with the MBI), estimates that modular construction could account for 10% of the construction market share in the United Stated by as soon as 2040. This percent increase in market share represents a change of more than 10 Billion dollars per year of new construction where buildings will be designed and engineered to be constructed modularly instead of using traditional on-site construction practices. Currently, few engineers and architects in the US have designed a modular building and there is a lack of detailed case studies which investigate the structural performance of modular buildings. As the market share of modular buildings grows, it is anticipated there will be an increased need for engineers to understand the behavior of modular buildings so as to optimize their design and engineer more efficient structures.

1.1.1 Defining Modular Construction

In this thesis the term "modular" is used to refer to buildings assembled with three-dimensional or volumetric units (Figure 1.3). However, there are many types of prefabricated construction processes; Table 1.1, from (Gibb., A.G.F Off-site Fabrication – Pre-Assembly, Pre-Fabrication, and Modularization report 1999), illustrates various levels of off-site construction methodologies. Level 0 corresponds to a traditionally built buildings where only the material, such as the steel beams or concrete masonry units, are manufactured offsite. Level 1 represents buildings systems that incorporate components such as pre-cast concrete slabs or prefabricated trusses. Level 4 corresponds to buildings built completely from modular units where around 60-70% of the building manufacturing is completed off site (Gibb 1999). These levels of off-site construction practices are not fixed and there are many buildings which were constructed with a process that fits somewhere in between two levels; however, they show a clear distinction between modular buildings and those using other prefabricated methodologies. This study examines only fully prefabricated building systems (level 4), composed of three-dimensional modules, similar to the one depicted in Figure 1.3.

FIGURE 1.3 - INSTALLATION OF A MODULAR HOSPITAL (STEELCONSTRUCTION.INFO)

TABLE 1.1 LEVELS OF OFF-SITE CONSTRUCTION

Level	*Components*	*Description of technology*
0	Materials	Basic materials for site-intensive construction, e.g., concrete, brickwork
1	Components	Manufactured components that are used as part of site-intensive building processes
2	Elemental or planar systems	Linear or 2D components in the form of assemblies of structural frames and wall panels
3	Volumetric systems	3D components in the form of modules used to create major parts of buildings, which may be combined with elemental systems
4	Complete building systems	Complete building systems, which comprise modular components, and are essentially fully finished before delivery to the site

Source: Adapted from Gibb., A.G.F., *Off-site Fabrication—Pre-Assembly, Pre-Fabrication, and Modularisation*, Whittles Publishing Services, Dunbeath, Scotland, 1999.

1.1.2 Advantages & Disadvantages of Modular Construction

The key advantages a modularly built building has over a traditionally built one can be defined in terms of time, cost, and quality. These advantages cannot be summarized in one set of quantitative charts because of the vast variability between different construction sectors. Modular systems have the greatest competitive advantage over site-based construction in buildings where there is a high degree of repeatability. Hotels and student residences are therefore two sectors where modular systems are most widely used; the repetitiveness of rooms requires less unique modules to be designed which in term translates to increases in construction speed and cost savings. Modular systems are used much less in office and mixed-use buildings because of the open floor plan and wider column spacing that is desired in these sectors. Table 1.2, summarizes which sectors, modular and other off-site manufacturing systems are most widely used in.

TABLE 1.2 USE OF OFF-SITE MANUFACTURING IN VARIOUS CONSTRUCTION SECTORS (LAWSON ET AL. 2014)

	Levels of off-site manufacture (OSM)			
	2. Elemental or planar systems			
Sectors for which OSM is most relevant	*Structural frames*	*2D panels*	*3. Mixed-construction systems*	*4. Fully modular systems*
Housing		✓✓		✓
Apartments—multistorey	✓✓	✓✓	✓	✓✓
Student residences	✓	✓✓	✓	✓✓✓
Military accommodation				✓✓✓
Hotels	✓	✓	✓✓	✓✓✓
Office buildings	✓✓✓		✓	✓
Retail buildings	✓✓✓		✓	✓
Health sector buildings	✓✓✓	✓	✓	✓✓✓
Educational buildings	✓✓✓		✓	✓✓
Mixed use, e.g., retail/residential	✓✓	✓	✓✓✓	
Industrial, e.g., single storey	✓✓✓		✓	
Sports buildings	✓✓✓	✓	✓	✓
Prisons and security buildings	✓		✓	✓✓✓

Note: ✓✓✓, widely used; ✓✓, often used; ✓, sometimes used.

A major driver in many development projects is "time to operation." The faster a building is completed, the faster it can start creating revenue for the owner. The time to operation can be simplified into two parts: design and construction. The architectural, structural, and mechanical design of a building takes roughly the same amount of time whether on or off-site construction processes are used. However, Gibb (1999) found that the construction period of a modular building can be half of what it would be for a comparable traditionally built building. The relative speed advantage modular construction has over on-site construction comes from the ability to complete several aspects of the building at the same time. Modules may be manufactured while a building's foundation is still being excavated. Although traditionally constructed buildings may incorporate top down construction techniques to start building upper levels before finishing the below ground levels by leaving access holes in each basement slab to allow for excavation of lower levels, this process is only used for buildings with more than 2 sublevels. Similarly, a modular factory may manufacture several floors of a building at the same time whereas an on-site construction crew is limited to building one floor at a time.

To compare the differences in schedule and cost between traditionally built and modular buildings a case study method is often utilized. It consists of identifying projects which match target criteria and documenting quantitative and qualitative data through literature reviews, questionnaires, and interviews. However, because most development projects are unique it may be difficult to find a direct comparison between a building built using a modular process and one built using on-site construction techniques. Smith (2015) proposes an alternative method to compare the cost and construction speed between traditionally built and modular buildings. The method consists of finding an existing building which was built using on-site construction but its layout and application made it feasible to possibly be built modularly. The building's plans and specifications would then be put out to bid by modular building manufacturers and partnering general contractors centered in the same area where the existing building is located. The bid data may then be compared to the actual cost and schedule of the existing traditionally built building. Figures 1.4 and 1.5 depict a summary of the results found by Ryan Smith and the Modular Building Institute when comparing the cost and construction speed of similar modular and traditionally built buildings using the case study method.

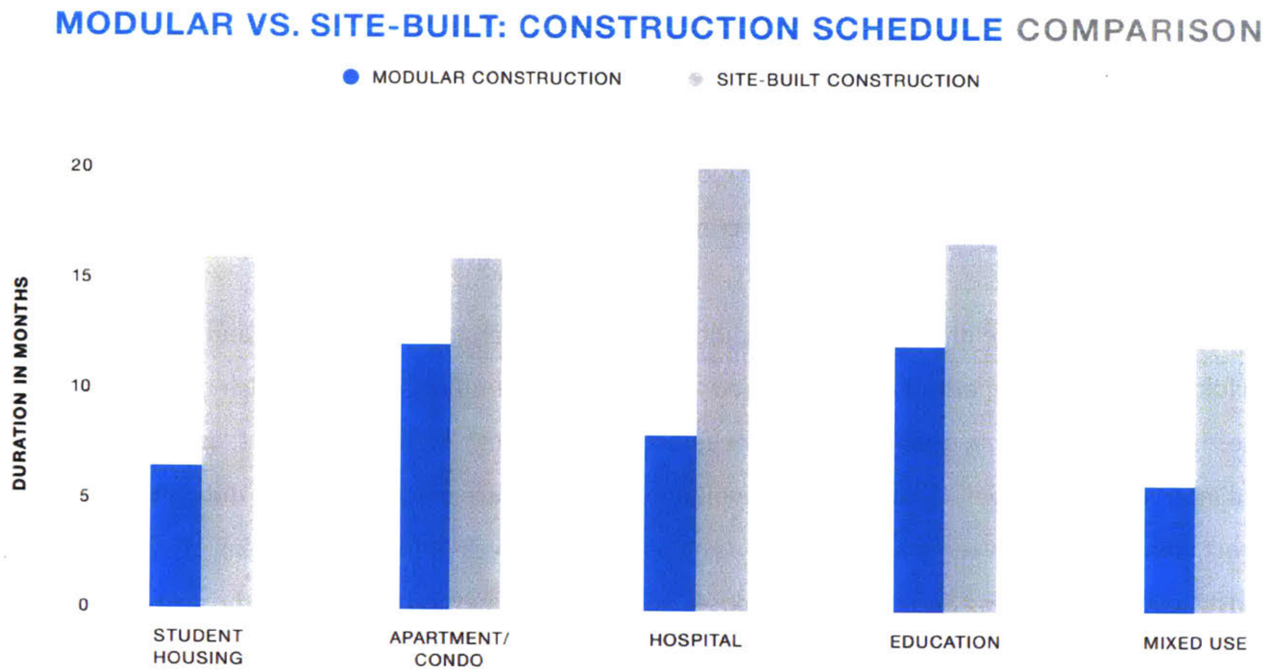


FIGURE 1.4 - MODULAR VS SITE BUILT: CONSTRUCTION SCHEDULE (RYAN SMITH & MBI: PERMANENT MODULAR CONSTUCTION, 2015)

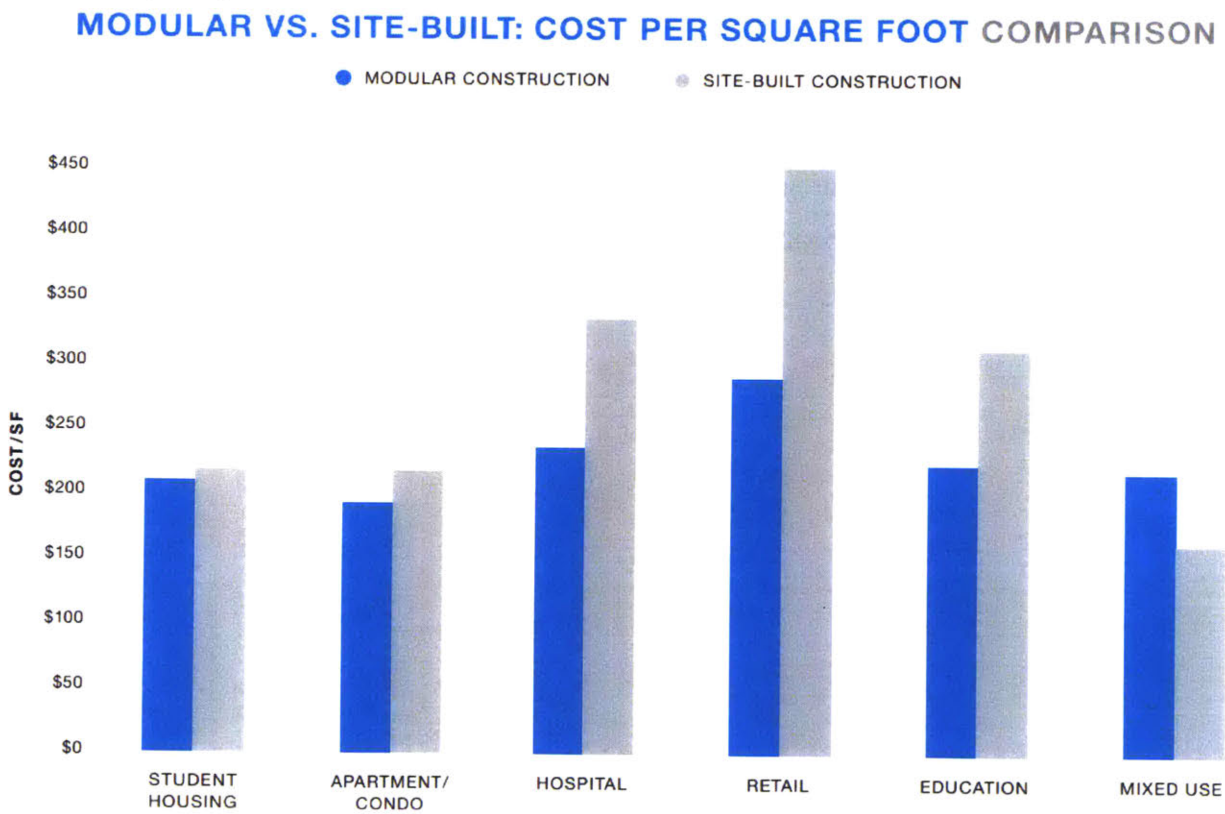


FIGURE 1.5 - MODULAR VS SITE BUILT: COST COMPARISON (RYAN SMITH & MBI: PERMANENT MODULAR CONSTRUCTION, 2015)

The controlled environment in which modules are produced allows modular buildings to be constructed more efficiently and with a higher level of quality. The advantages of modular construction over on-site construction processes may be summarized as follows:

- Less material waste – due to automation and repeatability
- Higher quality of interior finishes – due to controlled production environment of Level 4 pre-fabricated buildings
- Shorter build times – proven by Smith (2015)
- 75% fewer safety incidents – due to controlled factory production (Smith, 2015)
- Potential ability to dismantle and reuse the building – due to individual module structures
- Less disturbance to the surrounding neighborhood – due to factory production

A major disadvantage of modular construction is the limitation placed on the width of interior building spaces. The maximum size of a module is usually governed by transportation restrictions but larger modules may be used if permitting is acquired and an escort service is used when the modules are transported. Most modular manufacturers in the US, manufacture modules only up to 16 ft. in width (MBI). This limits the ability to have large open spaces which are constructed with modules. According to New Holland Pennsylvania based modular manufacturer, Niagara Relocatable Buildings (NRB), modules may get damaged while being transported which causes installation issues. If the frame of a module is bent out of place either during transportation or installation then it may not properly align with surrounding modules. Any adjustments which have to be made to modules once they arrive on site are often very costly (NRB). Although modular buildings provide many benefits over traditionally constructed buildings the infrastructure needed for factory production requires greater investment in fixed manufacturing facilities and construction repeatability is required to achieve an economy of scale in production.

1.2 Problem Statement

1.2.1 Design of Modular Buildings

Steel modular buildings present a unique set of construction and structural challenges. Most practicing architects and structural engineers in the US have little to no experience designing modular structures. However, as more development projects are incorporating modular construction, architects and engineers will need a better understanding of the design requirements and structural performance of modular buildings.

The schematic and early design development (SD and DD) phases of a project are critical to a project's success. Issues in the final design and construction of building often stem from mistakes made in these early design phases. It is therefore imperative that questions regarding the overall structural behavior of a modular building be addressed in these early design stages.

A major structural aspect which should be considered in in these early stages is the design of modular interconnections. The overall structural response of an assembly of modular units is influenced by the behavior of interconnections. The design of these interconnections will impact the selection of a building's gravity and lateral force-resisting systems as well as the overall architectural layout. The selection of modular interconnections will affect a buildings drift, acceleration, and member deflection; therefore, understanding the tradeoffs between different modular structural systems can lead to a more efficient design.

1.2.2 Thesis Goals

There is a need for engineers to understand the impact different interconnections have on the structural performance of modular building in the early design phases of a project so as to reduce the amount of design iterations and engineer an efficient structure. Therefore, the goals of this thesis are:

- Investigate the tradeoffs in terms of structural performance, constructability, and cost between steel modular buildings with different interconnections.
- Investigate feasibility of building high-rise modular buildings with rigid frames acting as the main lateral force resisting system

1.2.3 Approach

The approach used in this thesis is to model several modular building prototypes using SAP2000 v18, a structural analysis and design software developed by Computers and Structures INC (CSI), which have varying interconnections and building aspect ratios. The structural performance of the modular interconnections is then investigated using the notional element removal approach detailed in section 3.1. Less quantifiable attributes of the interconnections such as constructability and cost aspects are studied through a literature review and interviewing modular building manufacturers.

1.3 Outline of Chapters

In Chapter 2, modular construction, its various forms and applications are explored as it is still a relatively new construction. The literature review provides the context for this thesis and establishes the need for the studies conducted. The chapter also identifies the parameters and constraints which influence the design and behavior of modular buildings.

In Chapter 3 the methodology used to conduct the study and achieve the thesis goals is presented. Several published guidelines which outline approaches to study modular buildings are explained and compared.

In Chapter 4, four different interconnections are analyzed. Their use in the industry and influence on the structural behavior of modular buildings is examined. The results of the structural integrity study are presented and their implications are discussed. The different modular connections are compared and their use in modular buildings of various heights is critiqued.

In Chapter 5, the benefits and challenges of constructing high-rise modular building with various lateral force resisting systems and interconnections are explored.

Chapter 2. Modular Use and Design

2.1 Background to Modular Construction

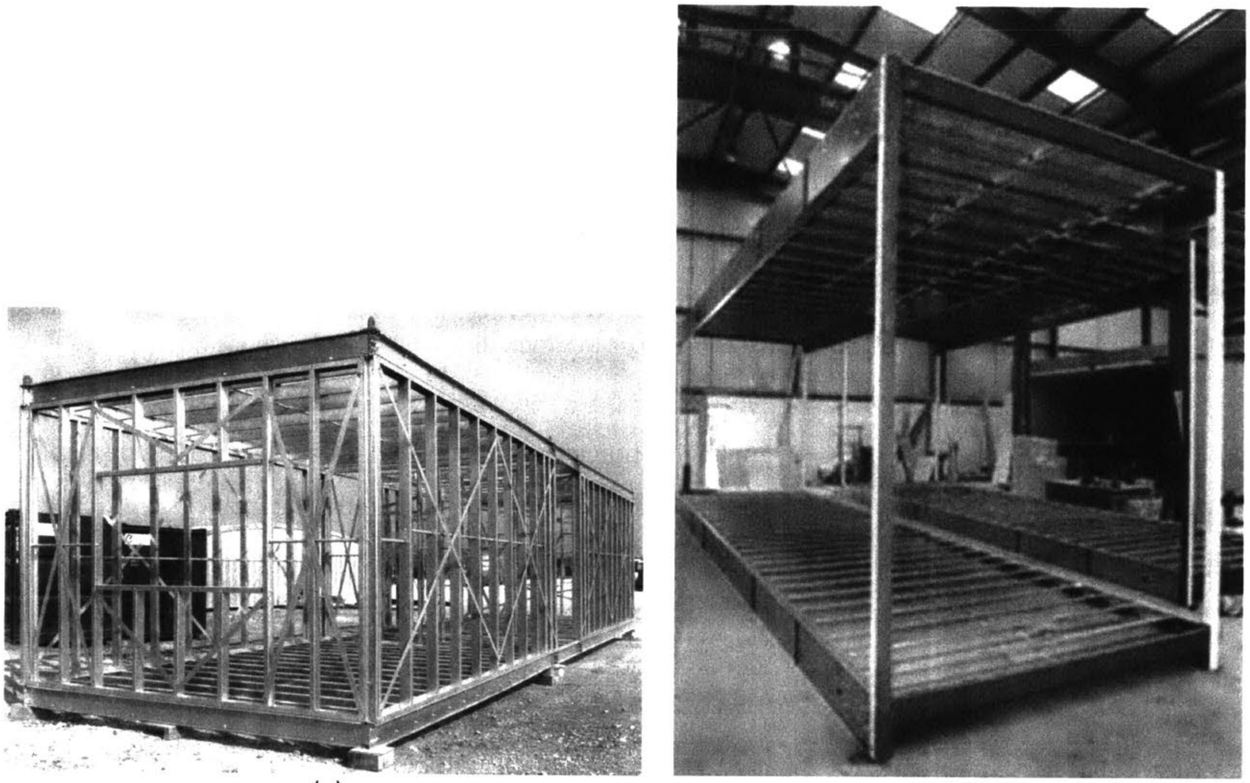
The use of modular construction is expanding to innovative applications. The first modular structures were small and simple buildings; but today modular construction is being used in buildings that are 30+ stories tall with complex geometries and non-uniform layouts. To better understand the possible applications of modular construction, various types of prefabricated modules and their uses are outlined below.

2.2.1 Module Categorization

Modules may be manufactured out of various types of materials including: steel, concrete, and timber. This thesis focuses specifically on modular buildings made of steel, but some of the concepts and ideas may be applied to concrete and wood modular buildings as well.

Steel modules are often classified based on their structural systems as follows:

1. Four-sided modules (Figure 2.1 a)
   - Four-sided modules are continuously supported on all their longitudinal sides which bear on the walls of the modules below.
   - Walls, floors and ceilings, are usually composed of cold formed C section studs placed singly or in pairs
   - Lateral force resistance is provided by X or K-bracing in the walls of the modules to create diaphragm action of wall and floor panels.
2. Corner Supported Modules (Figure 2.1 b)
   - Corner supported modules have open sides with columns only incorporated at their corners and sometimes at intermediate points
   - Columns are usually composed of hot rolled square HSS or angles
   - Lateral force resistance is provided by rigid connections, added bracing, or external systems.



(a) (b)

FIGURE 2.1 - MODULAR CATEGORIZATION. (A) FOUR-SIDED MODULE (B) CORNER SUPPORTED MODULE (LAWSON ET AL. 2014)

Four-sided modules are used primarily in buildings no taller than three stories because of their cold formed design (Lawson 2011). Corner-supported modules offer greater flexibility when designing an interior layout because of their open sides. Although Figure 2.1b depicts a module with a standard rectangular geometry, corner supported modules made be manufactured in L-shapes, have balconies cantilevering from them, or be outfitted with staircases.

2.1.2 Lateral Load Resisting systems in Modular Buildings

In traditionally built buildings, many types of lateral force resisting systems have been incorporated including those illustrated in Figure 2.2. Lateral force resisting systems are selected based on factors such as the building' geometry, geographic location, seismic region and serviceability criteria. Wind loads increase with height and therefore lateral force resisting systems are critical to the structural integrity of taller buildings.

While most of the structural systems illustrated in Figure 2.2 could be theoretically applied to a modular building, so far, the vast majority of modular buildings have incorporated braced frames or concrete cores to resist lateral loads (Lawson et al. 2014). The tallest modular building constructed so far is the Atlantic Yards Tower located in New York City, (described in detail in section 2.3). The modular tower is 32 stories tall and therefore at this height such building systems as belt trusses or bundled tubes would be excessive and most likely not cost effective. X or K-bracing may be incorporated inside modular walls as illustrated in Figure 2.1a. The braces in modular buildings are often made of flat rectangular bars or angles and can be designed to resist tension and compression or tension only. The majority of modular buildings which rely on in-wall bracing as the main lateral force resisting system are no taller than 6-8 stories (Lawson et al. 2014). In conventionally constructed buildings, braced frames have been used in buildings of upwards of 40 stories (Ayman 2010); however, in modular buildings the braced frame must fit within the walls of a module and therefore the bracing is smaller than what it could potentially be in a conventionally built building.

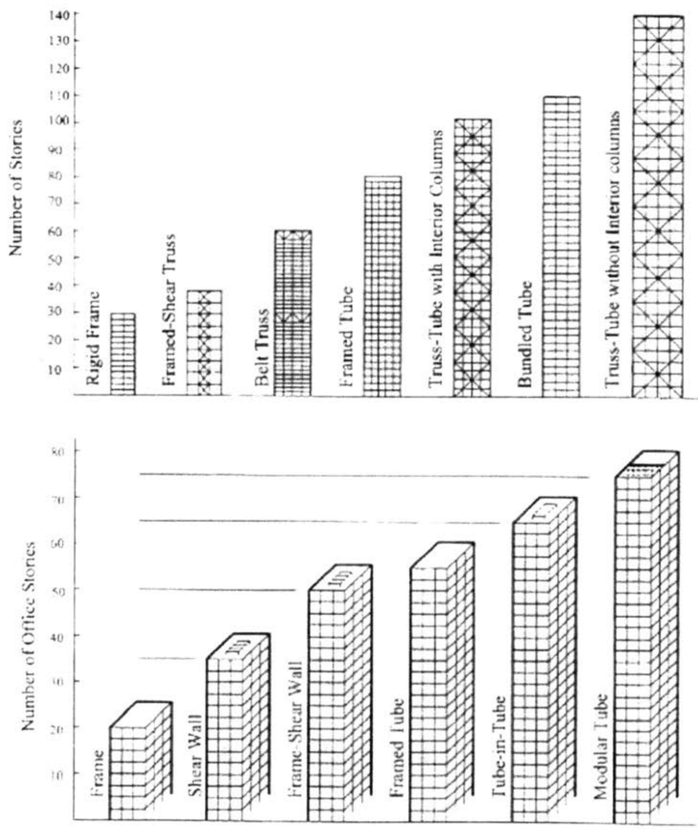


FIGURE 2.2 - BUILDING SYSTEMS (AYMAN 2010)

For modular building taller than 7 stories, concrete cores have been the predominantly used lateral force resisting systems (Lawson et al. 2014). The core is often constructed using traditional cast in place construction techniques. (Figure 2.3 a) depicts the Paragon, an 11-story residential complex located in Brentford London, UK. A self-climbing concrete formwork system was used to construct the concrete core of the Paragon which acted as the main lateral force resisting system and also housed the elevators and stair cases. A pre-cast concrete module such as the one depicted in (Figure 2.3b) may also be used to construct a buildings core. Whereas it typically takes a day to erect one stories of cast in place concrete core, more than 5 stories of a precast core can be erected in a single day. However, a precast core has to be heavily reinforced with steel at its lifting points so that it can resist the tension loads when it is being assembled into place. The precast concrete relies on frictional bearing but connection bolts are used to ensure positional accuracy between the modules. A third option for the central core is utilizing a twin wall. Twin wall construction is a combination of in situ and precast concrete construction. The twin wall, consists of two precast reinforced concrete skins which are connected and spaced with a steel lattice. The concrete skins act as a permanent form work and act compositely with the in-situ concrete which is cast between them. The twin wall system takes longer to construct than a fully precast core but the in-situ placed concrete creates a stronger connection between the core units. "Corefast" is another type of prefabricated core, (Figure 2.3c). It is a steel composite core, similar to the twin wall except steel panels are used instead of the precast concrete panels. The steel panels confine the concrete which increases its load carrying capacity, however the exposed steel must still be fire-proofed. Steel brackets which are welded to the Corefast walls are used to connect the modules to the wall.

Concrete cores are commonly used in both concrete and steel conventionally built buildings as lateral force resisting systems because of their structural and constructability advantages. Concrete cores are often placed near the center of buildings and may be designed to take large portions of the gravity loads which in turn reduces the number of interior columns required and frees up floor space. Services system such as elevators and stairs require fire proofing which often comes in the form of concrete walls, thus the concrete core is able to serve a dual purpose. The hollow tube shape of the concrete core enables it to resist multi direction wind and seismic forces as well as any torsional loads. A concrete core is also easy to construct as it can be erected before the rest of the structural framing.

(a) (b) (c)

FIGURE 2.3 - MODULAR CORE SYSTEMS. (A) PARAGON UK, CASE IN PLACE (B) PRE-CAST (C) COREFAST SYSTEM (CALEDONIAN MODULAR, MAY 2018)

Although the vast majority of 7+ story modular buildings have been constructed with a concrete core, there are several disadvantages of having to rely on a core as the only means to provide lateral stability in a high-rise modular building. In order for a concrete core to provide lateral stability to the rest of a building, there must exist a continuous load path where seismic and wind forces can be transferred to the core. In conventional buildings, the roof and floors are integrated with the structural framing system through a concrete slab or composite deck so that they effectively act as a single diaphragm which provides a continuous load path. However, in modular buildings this lateral congruity is not inherent because of the discontinuous nature of modules. The majority of modules used in pre-fabricated buildings are connected only at their corners which complicates the transfer of lateral loads to the core (Sharafi et al. 2017). Often extra reinforcing in the form of in-plane trusses must be incorporated in the corridors to assist in transferring these loads (Lawson et al. 2014). In some instances, an in-situ concrete floor slab is poured after the modules on a particular floor have been assembled to create a continuous diaphragm which connects to the concrete core. However, this practice reduces benefits such as construction speed and reduction of construction waste that a fully modular building provides to the end user (Gunawardena 2016). An in-situ poured concrete slab also inhibits the ability to reuse or relocate the modules.

In many conventional buildings with concrete cores, the concrete core is designed to resist both lateral and significant gravity loads. As depicted in the floor framing plan shown in Figure 2.4, beams often span directly between perimeter columns to the concrete core forcing the core to support a large tributary area. In traditionally constructed buildings this beam layout reduces the number of interior columns and creates a more open interior space. However, as depicted in Figure 2.5 modular buildings are designed so that gravity loads are transferred directly from module to module and therefore do not rely on the concrete core to carry much of the gravity loads. Modules must be supported at all four corners in order to be lifted by a crane and installed without deforming, therefore, creating an open floor plan with much of the gravity loads resisted by the concrete core as shown in Figure 2.4 would not be feasible.

In both conventional and modular buildings, concrete cores are often placed in the center of the building. As shown by Figure 2.6 the torsional displacements a building experiences are minimized when the eccentricity between the center of mass of a building and the center of rigidity is reduced. However, because service areas are often located inside the central concrete shear core this creates a restriction on where stairs and elevators may be placed. This restriction limits the freedom architects have in designing a buildings floor plan.

Because of the disadvantages present in using a concrete shear core as the main lateral force resisting system of a modular high-rise building it is worth exploring other potential structural systems.

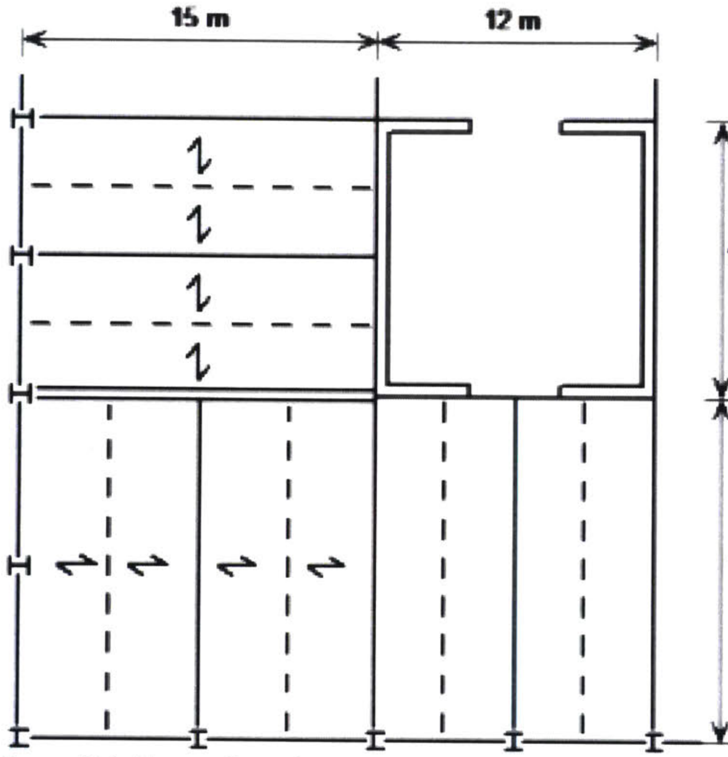


FIGURE 2.4 - TYPICAL BEAM LAYOUT AROUND CONCRETE CORE (STEELCONSTRUCTION.INFO)

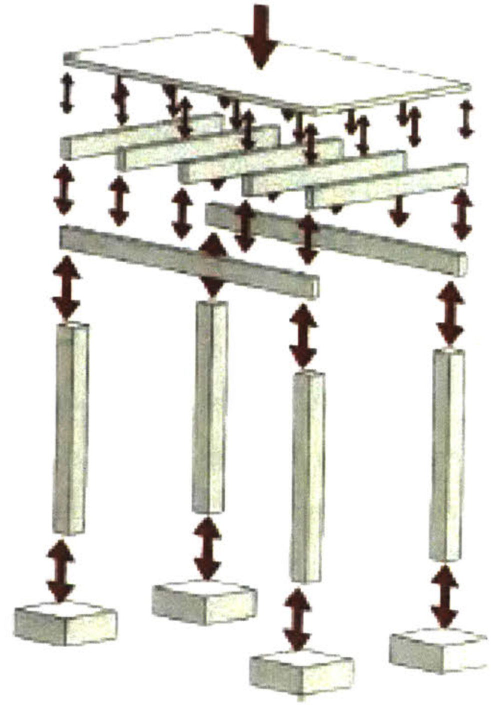


FIGURE 2.5- GRAVITY LOAD TRANSFER IN CORNER SUPPORTED MODULAR BUILDING (SHARAFI ET AL., 2017)

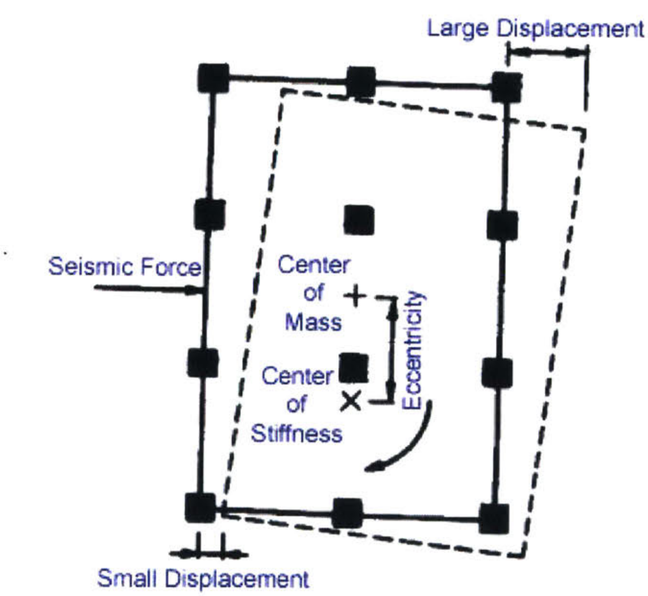


FIGURE 2.6 - STORY ECCENTRICITY (MIDASUSER, MAY 2018)

2.2 Modular Interconnections

Connections between modules have a significant influence on the structural stability of a modular building. Modular interconnections are made at the top and bottom of modules so that adjacent modules are connected at their corners. Steel module floor systems are often comprised of a metal deck which is supported on purlins made of hot rolled rectangular HSS sections or cold formed channels. The subfloor is then constructed using cementitious particle boards followed by acoustic padded and floor finishes. In this floor system no concrete is used making the modules relatively light. On average steel modular systems without concrete slabs weight approximately 65% of a conventional RC flat-slab building (Farnsworth, 2014). However, without a cast in place concrete slab the modules are structurally unattached to each other. Interconnections are required to provide a continuous load path. Lateral loads are transferred between the individual modular diaphragms through the interconnections, until they reach the lateral force resisting system. Interconnections need to be designed to transfer horizontal forces due to wind, earthquakes, and extreme or accidental loads. Interconnections are also designed to provide alternative load paths in the event of accidental damage to structural members. Sections 2.2.1 and 3.1 describe in more detail how modular interconnections are critical in providing structural integrity in modular buildings and preventing progressive collapse.

Mechanical fasteners, such as those shown in Figure 2.7, are the most common type of modular interconnection. They are usually comprised of an arrangement of horizontal and vertical plates that are bolted or welded to the external face of modules. To realize the advantage of using pre-fabricated modules with finished interiors, all structural connections should be made external to a module. The modular interconnections are usually installed using external scaffolding and other mobile access platforms (Lawson et al. 2014). Although alternate proposals of attaching modules such as through interlocking or post tensioning are being explored, mechanical fasteners located on module corners are the most popular connecting systems due to their simplicity, they do not require a highly skilled workforce and ease or service. As illustrated in Figure 2.7 many of these mechanical fasteners can be constructed with only a single steel plate and two bolts. However, the design of these simple connections makes them uncapable of transferring significant moments between modules. They are often modelled as simple pin connections that transfer only shear and axial forces.

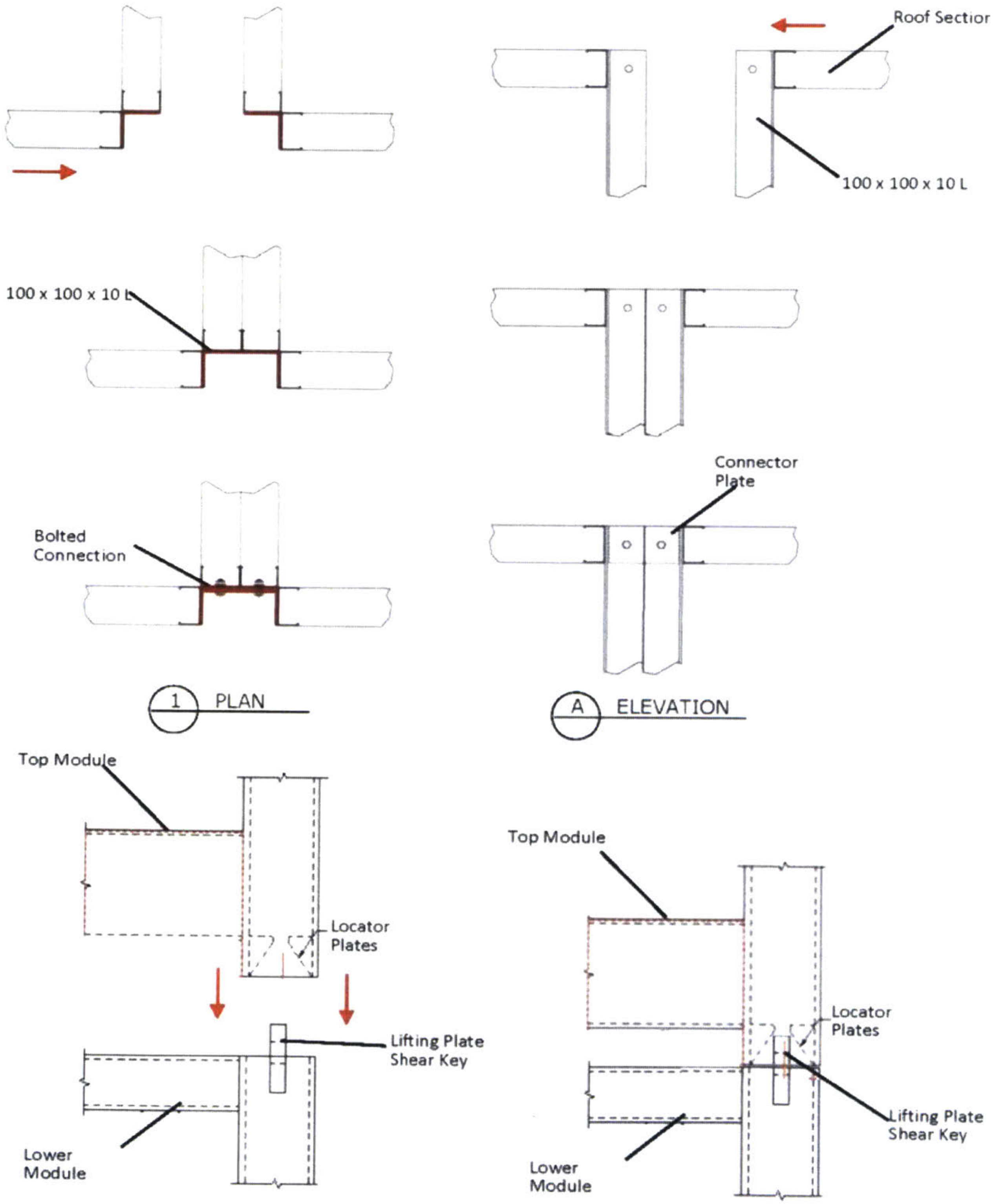


FIGURE 2.7 - TYPES OF MODULAR CONNECTIONS (MILLS ET AL. 2015)

2.2.1 Interlocking Modular Systems

In the majority of modular buildings, modules are designed to connect to each at their corners through mechanical fasteners to ensure vertical stability of the structure. However, several researchers have proposed novel ways to interlock modules without the use of interconnections; they are worth exploring because of the disadvantages inherent in mechanical fasteners and the potential constructability and structural benefits that these innovative interconnections may bring.

In his paper "Interlocking system for enhancing the integrity of multi-story modular buildings", Sharafi et al. (2017) introduce an innovative way to interconnect modules called the "modular integrating system" or (MIS). The system is comprised of a pair of interlocking joints (Figure 2.8) which fit on each of the six edge beams of every module. The topological interlocking system consists of rigid integral mechanical attachments defined as "connection type A" and "connection type B" in Figure 2.8. Both connection types are composed of tongues and grooves that when attached will interlock two or more modules and prevent relative motion in major directions of translation and rotation and prevent unwanted separation, (Sharafi et al., 2017). The only difference between connection type A and B is that the interlocking strips have opposite tongue and groove patterns. Incorporating only two different types of connections and significantly simply the design and construction of a building. A traditionally built steel building will often have several different connections due to range of different beam and column sizes. Similarly, modular buildings which are connected with mechanical fasteners may require different interconnections for joints which connect different groups of modules.

In the modular integrating system, two adjacent modules are connected by simply using a crane to push one of them in the direction perpendicular to the length of the tongues and grooves. In order for the modules to become properly interlocked a specific assembly method must be followed. The corner module on the base level is placed first, then adjacent modules in the first row and column of the module layout plan are assembled using a simple push slide motion. This is repeated until the last module on the opposite corner is placed and then the same order is followed on each upper level. This assembly method will cause all adjacent modules to be integrated and for the whole system to be monocoque. By interlocking the modules together, the assembly of modules will behave as a single structure which will improve the structural integrity of the building. In many traditionally built concrete and steel buildings the floors and roofs behave as a single diaphragm because of the concrete or composite slab which is cast between the framing members. Lateral forces are transferred continuously through the diaphragm and their collector elements until they reach the lateral force resisting system of the building. Modular buildings with mechanical fasteners at the corners of the modules rely on the fasteners to transfer loads between the diaphragms of each module. The failure of a single fasteners will cause a discontinuity in the load path and may cause stress concentrations in the other fasteners (Lawson et al. 2014). However, the modular integrating system allows for adjacent modules to be connected at several locations so that the failure of any individual tongue to grove connection has a minimal impact on the structural integrity of the building.

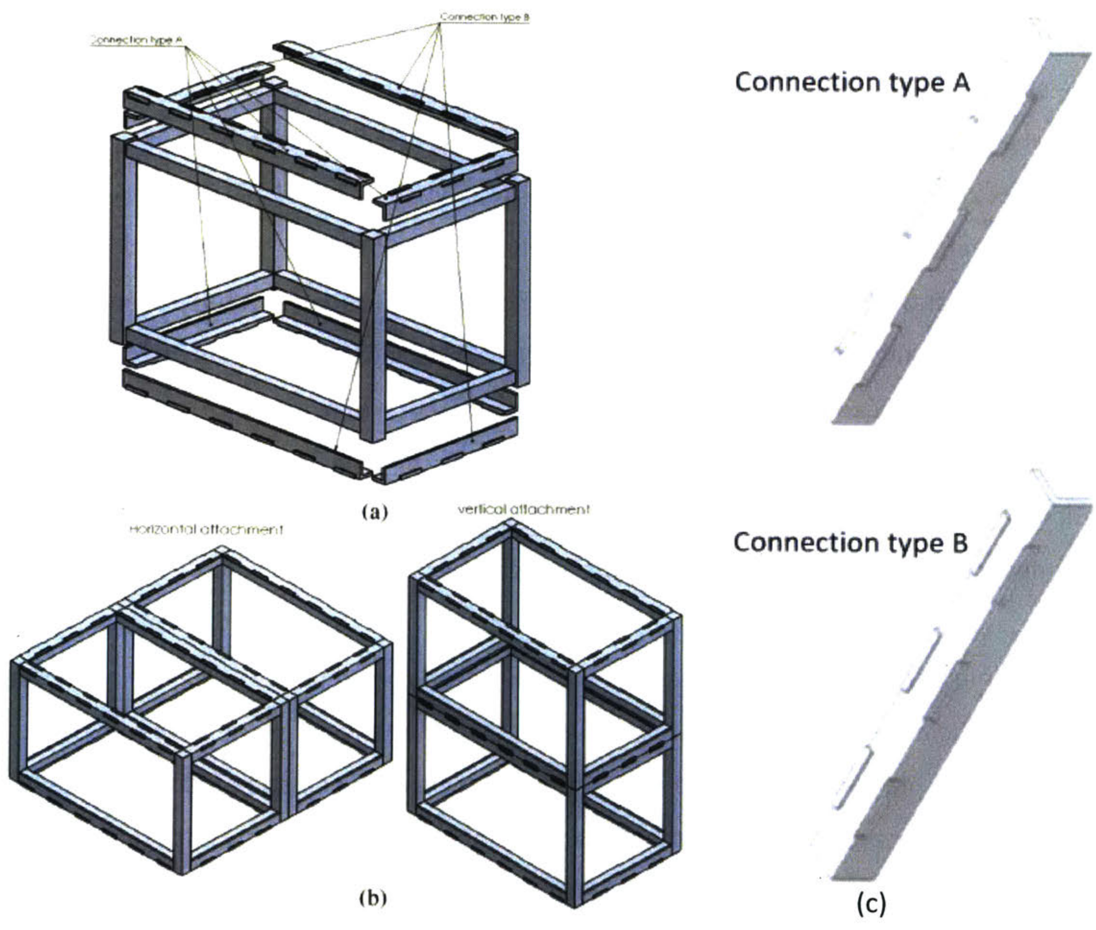


FIGURE 2.8 - (A) ATTACHMENT OF INTEGRATING CONNECTION TYPES. (B) ATTACHING TWO ADJACENT MODULES. (C) INTERLOCKING STRIPS (SHARAFI ET AL. 2017)

The integrating connection transfers loads between modules by having the tongues and groves bear on each other. The mechanical interface resists shear and compressive stresses that are imposed from external loads which try to displace the modules Figure 2.9. Moments can be resisted by the modules because the interlocking tongues and grooves resist overturning. Moment connections in traditionally built steel buildings are often difficult and costly to construct because of the large amount of on-site labor required to make a rigid connection between two steel frame members. The integrating design allows for modules to act as rigid frames and be assembled in a manner where it easy to visually see if connections were properly constructed. However, a disadvantage of the interlocking connection is that it does not resist tension in the direction opposite of the applied load. The modular integrating system is also limited to modular buildings which consist only of rectangular modules which are the same size. It would be difficult to integrate the system into a building with more complex geometries.

One of the main advantages of the modular integrating system is that it provides a more redundant interconnecting system than linking modules only at their corners using mechanical fasteners. Adjacent modules may also be connected by incorporating intermediate mechanical connections such as those shown in Figure 2.10. The intermediate mechanical fastener consists of angles which are welded along the outside perimeter of the modular ceiling edge beams. When the modules are assembled on site the angles of adjacent modules are then bolted together as depicted in Figure 2.10b; several of these connections may be incorporated along the edge beams of each module. These intermediate fasteners would add a level of redundancy to a modular building so that if a corner connection between two adjacent modules failed, the intermediate fastener would still apply a tying force between the two modules. Adjacent modules tend to slip with respect to each other when subject to lateral loads because of the discontinuity between modules (Lawson, 2005). These intermediate fasteners would help to resist the horizontal shear forces caused by lateral loads and limit non-uniform displacements between modules.

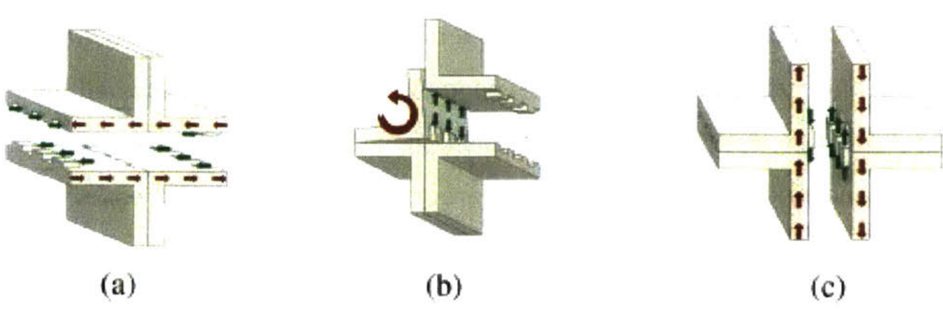


FIGURE 2.9- LOAD RESISTING MECHANISM. (A) HORIZONTAL MOVEMENT, (B) OVERTURNING (C) VERTICAL MOVEMENT (SHARAFI ET AL. 2017)

(A) (B)

FIGURE 2.10- INTERMEDIATE MODULAR FASTENER. (A) CONNECTION LOCATION (B) INTERMEDIATE BOLTED CONNECTION (IMAGES TAKEN BY THE AUTHOR)

2.2 Applications of Prefabricated Modular Structures

Prefabricated construction has been used for several centuries in Europe and Japan. One of the earliest accounts is that of John Rollo, a Scottish military surgeon, who described the use of portable hospital buildings in the West Indies in the 18th century. From then on, modular construction has been used in new and innovative applications.

2.2.1 The London Crystal Palace

One of the earliest and most iconic examples of prefabricated construction is the London Crystal Palace seen in Figure 2.11. In Europe it opened the way to the Modern Movement and influenced the use of new construction materials and methods in the US. The London Crystal Palace was designed by Sir Joseph Paxton for the Great Exhibition of 1851. The "Great Exhibition of the Works of Industry of all Nations" showcased the latest technologies and innovations from around the world and so it was imperative the building itself was an engineering marvel.

Paxton, who before designing the London Palace, was a famous gardener, was influenced by his passion for biomimicry (the design of structures and systems that are modeled on biological processes). His previous work included designing the public gardens at Bikenhead Park which directly influenced the design of Central Park in New York City. He was particularly inspired by the giant leaves of the Victoria Amazonica waterlily (Figure 2.11b) and attempted to mimic its compact design (Merin 2013).

The design of the London Crystal Palace consisted of large open spaces supported by cast iron columns, trusses and x-braces which supported a completely glass façade (Figure 2.11c). However, rather than design the Palace to be constructed beam by beam, Paxton implemented a modular approach by creating a system of right angled triangles which held 10in x 49in glass panels and were supported by a grid or mirrored cast iron beams and pillars. The modular glass components were prefabricated and installed at a rate of 18,000 panes a week allowing the project to be constructed in 5 months.

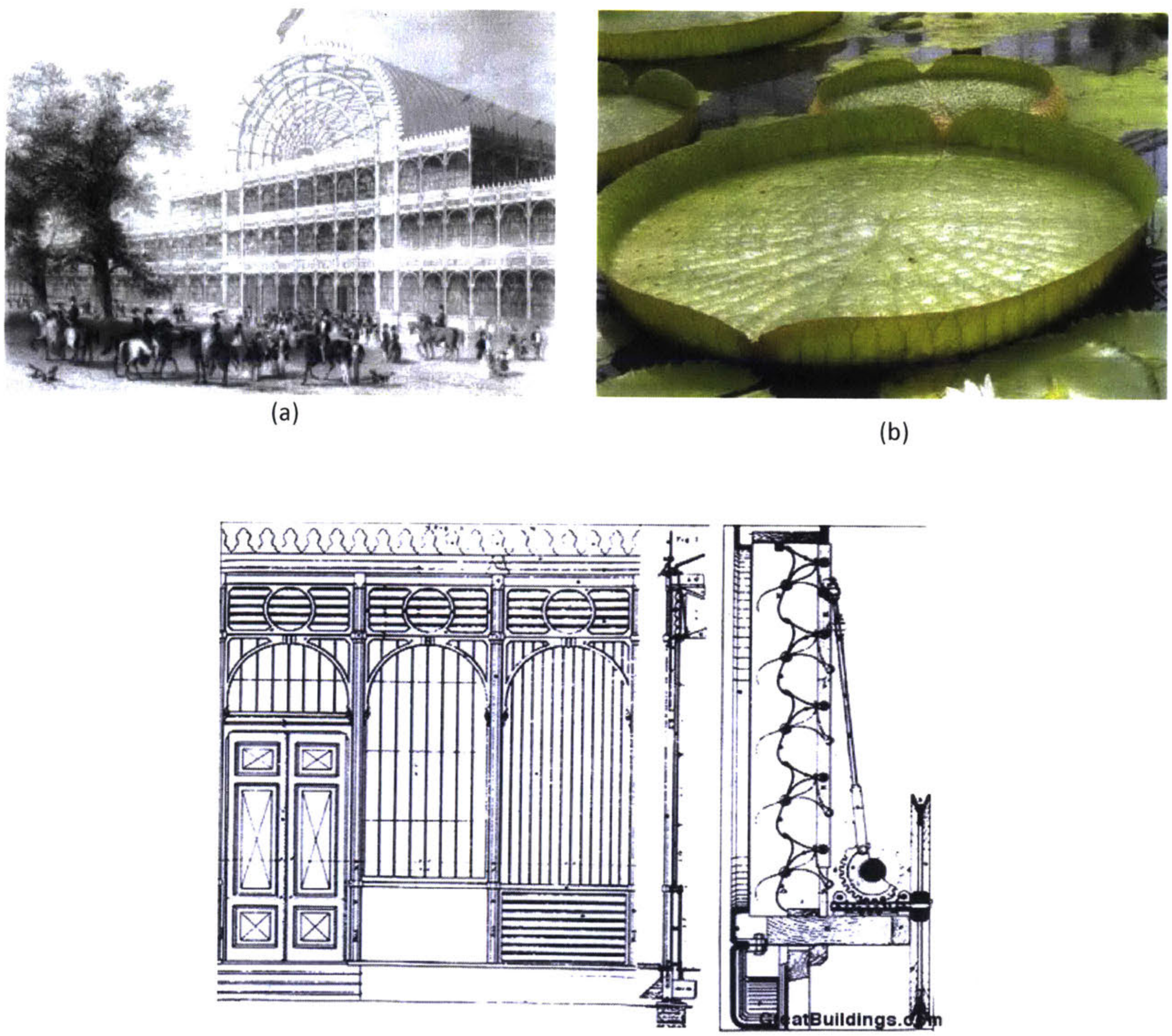


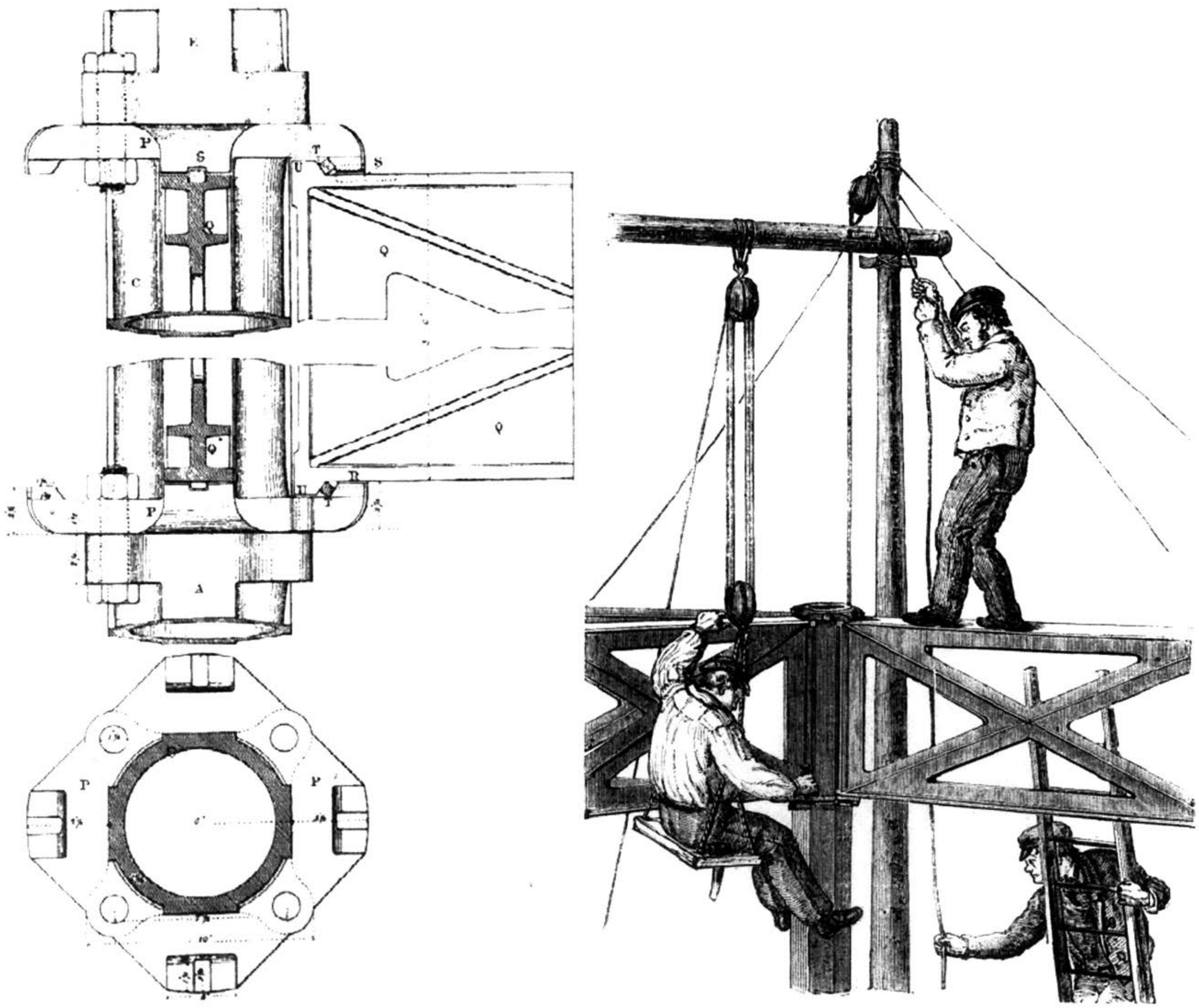
FIGURE 2.11 - LONDON CRYSTAL PALACE. (A) FRONT VIEW (B) VICTORIA AMAZONICA WATERLILY (C) MODULAR GLASS PANELS (MERIN, 2013)

Paxton combined together tested building designs, materials, and construction methods from various manufacturing, engineering, and construction fields in order to construct the building in record time. Paxton standardized several elements of the Crystal Palace's structural system such as the cast iron columns. All ground floor columns were 22 ft. long and those above were 20 ft. long. In order reduce the number of different connections, the external dimensions were kept constant for the majority of columns and girders (Addis 2006). The wall thicknesses of the columns and girders varied based on the applied loads but their cross-section widths and depths remained constant. A fixed connection was created between the cast iron columns and girders using a series of wedges (Figure 2.12) Both cast iron and oak connections were used. The oak allowed for greater longitudinal movement as the Crystal Palace expanded and contracted with changes in temperature.

The roof of the Crystal Palace was supported on the "Paxton Gutter." During its construction, the Paxton Gutter was a novel system which carried both rainwater and condensation from the interior surface of the glass along separate channels into the top of the hollow columns (Addis 2006). Paxton limited the deflection of the gutters and reduced ponding by pre-cambering the gutters with adjustable trussing rods. The hollow columns which doubled as drain pipes were incorporated in sections where the roof of the Crystal Palace was flat and water could not runoff naturally. The columns were connected at their bases to horizontal pipes using molten led, which then led to the main sewer system at the south of the building.

The lateral force resisting system of the Crystal Palace was comprised of both Vierendeel trusses and diagonal braces. The Vierendeel action was provided by the rigidity of the column to girder connections created by hammering in cast iron wedges. The Vierendeel system was rare at the time and Paxton explained how it worked by comparing it to a rigidly connected wooden table (Addis 2006). Diagonal braces were fitted in 220 vertical bays to keep the cast iron beams and columns from carrying excessive bending stresses. The timber floors of the Crystal Palace acted as part of the diaphragm carrying wind loads from the glass facades, to the diagonal braces and fixed frames, and down to the concrete foundation.

Structural pre-assembly was a major reason the Crystal Palace was erected in just 27 weeks. Rather than receive prefabricated components from a modular factory, as is done today, mini manufacturing workshops were set up around the palace. Six horse-power steam engines were used in the workshops which provided power to drive the machinery for routing, shaping, sawing, and drilling the structural components. The rigid connections allowed for the frames to be assembled with minimum temporary supports. The timber ribs which made up the arches of the transept vault were too slender to lift into position individually without deforming. Several timber ribs, purlins, and wrought iron diagonal bracing were preassembled on the ground floor and lifted as a single unit which prevented any of individual pieces from breaking and decreased the construction time.



(A) (B)

FIGURE 2.12 - CRYSTAL PALACE COLUMN TO GIRDER CONNECTION. (A) CONNECTION SECTION VIEW (B) CONNECTION INSTALLATION (ADDIS, 2006)

2.1.2 Victoria Hall, London UK

Victoria Hall is a modular built student dormitory located in Northern London. The 19 Story building, shown in Figure 2.3, is comprised of 435 student rooms spread among 3 wings which surround a central spiral shaped tower. The student residence was designed by O'Connel East Architects for the developer Clovis Propco. Futureform Buildings Systems manufactured the modules in their facility in

West Sussex and then transported them 90 miles to Wembley where they were assembled by the general contractor Mace.

The wings of Victoria Hall are composed of modules which are 52' long x up to 12.5' wide. This module size makes it possible to incorporate a twin corridor and two rooms, reducing the amount of on -site work required. The corridors are finished after modules are installed and provide access for services to be connected. Each wing of Victoria Hall consists of 10 modules per floor and 3-4 floors were installed per week. It is estimated that the student dormitory was completed 6 months faster than it would take it build it using only on-site construction (MBI). The façade of the buildings consists of light weight rain screen cladding supported on horizontal rails attached to the modules. The modules are weather tight, and fully insulated. The use of modular construction allowed Victoria Hall to be constructed much more sustainably than a traditionally built building.

FIGURE 2.13 - VICTORIA HALL, UK (FUTURE FORM 2011)

The main lateral force resisting system implemented in Victoria Hall is a concrete core. While the wings are made of steel frame modules, the central tower was built on site using cast in place concrete. Figures 2.4 and 2.5 shows the core being cast in place using self-climbing concrete formwork system while the at the same time modules are being installed in the three wings. The walls and floors of the modules are constructed by welding together cold formed steel C-sections. Although the modules themselves are composed of rigid connections, because the building height is over 200 feet, the overall structure would not be able to resist the high wind loads without the support of the core. The majority of modular buildings over 6 stories rely on a core for lateral stability (Lawson et al. 2014). The modules themselves make up the individual rooms and carry the majority of the gravity loads. The central core houses elevators, stairs and service risers, it increases the rigidity of the building and limits drifts and displacements due to wind. In order for the modules to properly engage the core, additional bracing had to be incorporated within the module floors and ceilings. Lateral forces were transferred between modules through the modular interconnections. Ensuring the modules are properly braced to the core creates structural and construction challenges. The concrete core is subject to creep and shrinkage, while the steel modules experience little long-term deformation. A maximum manufacturing deviation of 0.2 inches was achieved between adjacent modules. Construction tolerances for the concrete core were much greater which could cause out of verticality in the modules. The interior spaces within the core are built on site which reduces the benefits in sustainability and quality control that a fully modular building would experience.

FIGURE 2.14 - VICTORIA HALL MODULE INSTALLATION (FUTUREFORM 2011)

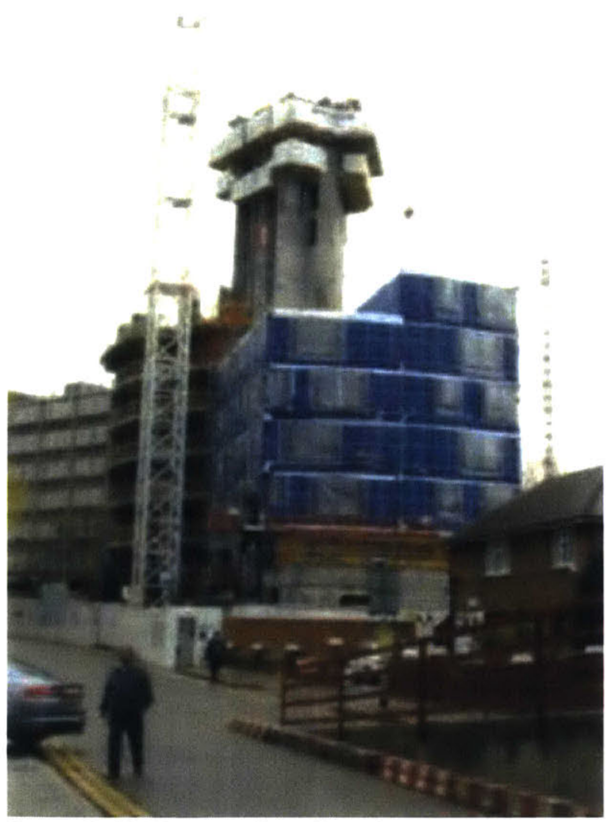


FIGURE 2.15- VICTORIA HALL CORE CONSTRUCTION (FUTUREFORM 2011)

2.1.3 Atlantic Yards, New York

Atlantic Yards B2 Modular Residential Tower, (referred to from now on as Atlantic Yards) is the tallest modular building in the world. At 32 stories, (Figure 2.61), the tower, which was finished in 2016, is by far the most ambitious modular building constructed in the United States. Compared with countries like the United Kingdom, Japan and South Korea where modular construction has been used much more widely, the US has lagged behind in adopting modular construction, with only 3.5% of new development projects being built with modular construction (MBI, 2018).

Located and manufactured in Brooklyn, New York, the tower was designed by SHoP and Arup for the developer Forest City Ratner. The tallest modular building before Atlantic Yards was a 24-story apartment tower in Wolverhampton, UK; the design team for Atlantic Yards was tasked was developing a modular system which was optimized for construction market conditions in NYC and could be delivered at a price competitive with conventional flat-slab construction. Figure 2.17 shows the modular floor plan which consists of 36 modules per floor; a 90 ft. transfer girder, (Figure 2.19), is used over the entrance of the Barclays Center to support 19 stories of modules above. The substructure was built conventionally with steel floor framing and reinforced concrete perimeter walls.

FIGURE 2.16 - ATLANTIC YARDS (SHOP ARCHITECTS)

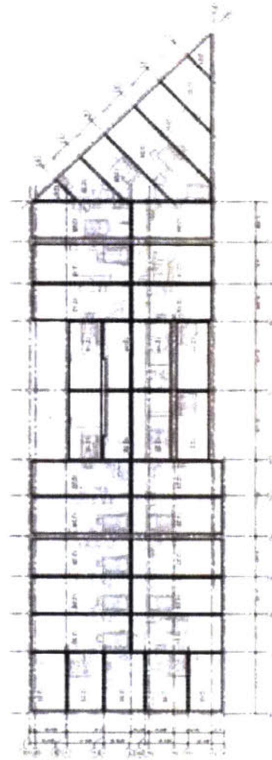


FIGURE 2.17 – MODULE LAYOUT (FARNSWORTH 2014)

The module sizes were governed by transportation restrictions, with the largest modules being 50 ft long, 15 ft wide and 10.5 ft tall. The module sizes were also influenced by crane requirements; the heaviest modules were designed to be close to the 26-ton tower crane to fall within the lift limits at different crane radii. Atlantic Yards had 225 unique module structure types. Incorporating such a large variety of modules is uncommon because modular construction is most cost efficient when standardization in manufacturing is achieved. However, Forest City Ratner desired to have a wider range of apartment types and chose mass customization over mass production (Farnsworth 2014).

The modules are stacked on a conventional steel-framed plinth level above the ground floor which makes it possible to have open areas, unencumbered by module walls in the lobby and provides a level platform to stack the modules on. Figure 2.8 depicts the structural components of a single module. The fully welded steel-framed chassis is comprised of 6" square HSS corner columns, 8" by 4" rectangular HSS bottom chords and 2" by 3" intermediate posts. The module floor system consists of a 2" metal deck running in the long direction of the module, supported by 6" by 3" rectangular HSS purlins. Where hallways or door did not need to pass through the module walls thin steel bars were included as diagonal bracing to minimize module weight and deflections. The sides of modules without steel bracing act as welded Vierendeel trusses spanning between corner columns.

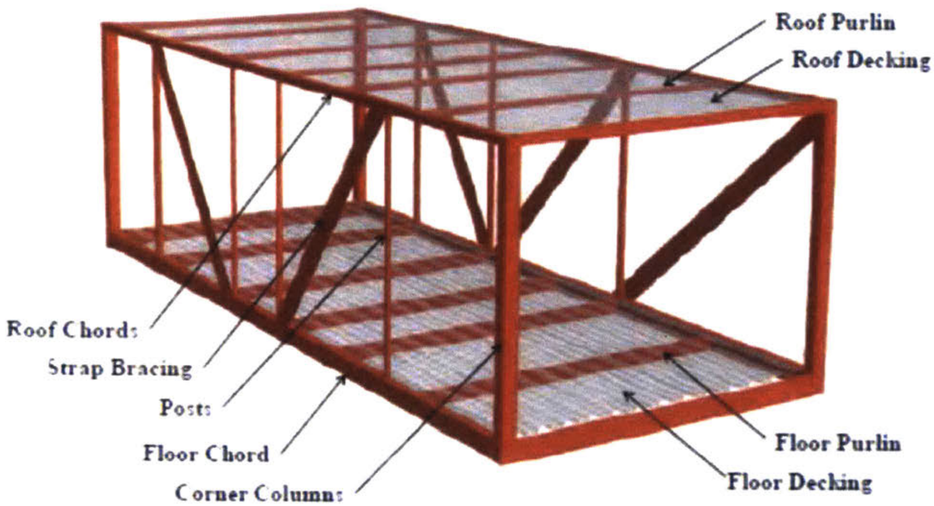


FIGURE 2.18- MODULE STRUCTURE (FARNSWORTH 2014)

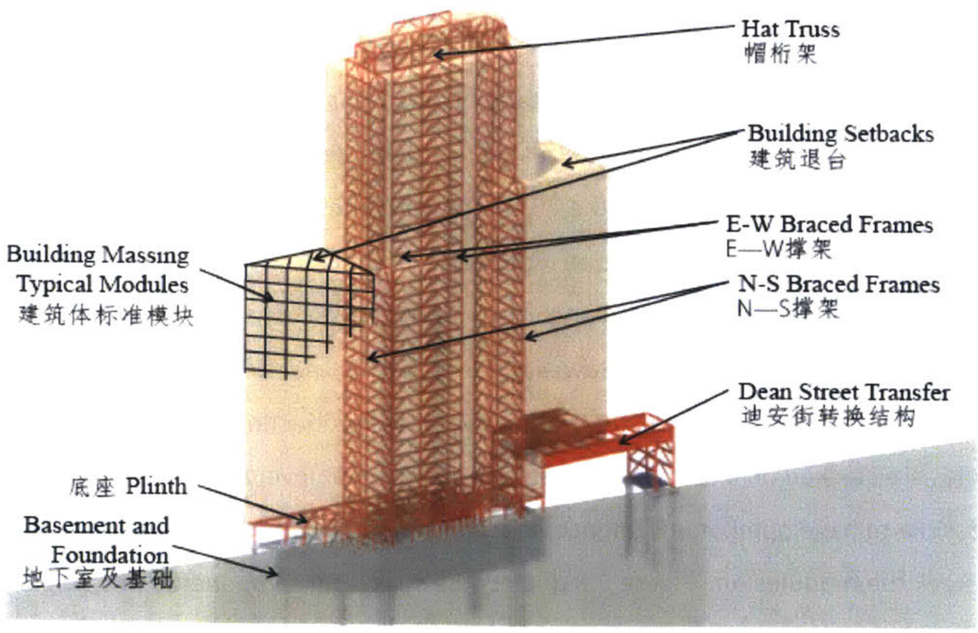


FIGURE 2.19 - ATLANTIC YARDS STRUCTURAL SYSTEM (FARNSWORTH 2014)

The lateral force resisting system consisted of conventionally built steel braced frames. Two inverted v-braced frames were constructed in each primary direction and tied together at the roof level with a hat truss (Figure 2.9). Lateral loads were transferred to the braced frames from the roofs of the modules which were composed of 1" metal decks and supported by 3" rectangular HSS purlins and acted as the lateral diaphragm. The module roofs were designed to act as the lateral diaphragms instead of the module floors to minimize the potential for on-site activities to damage the finished apartments (Farnsworth 2014). Although modular interconnections were incorporated between the module roofs, they were designed to transfer the full lateral loads to the braced frames (Figure 2.10). The modules were designed to carry only gravity loads. The interconnections were designed as an assembly of thin steel tension plates to resist against progressive collapse.

Atlantic Yards deviates from the majority of high-rise modular buildings in that it does not use a reinforced concrete core as the main lateral force resisting system. A central core was considered but was ultimately abandoned to minimize the number of union trades on site. A steel only solution reduces construction tolerances between systems and makes it simpler to transfer lateral loads from the module diaphragms to the braced frame. Connection issues between the modules and lateral force resisting system are reduced in the steel-only option because the creep and shrinkage of a reinforced concrete core do not have to be considered. The stairwells and elevator core were constructed modularly and did not have to be confined to being placed in a central concrete core.

Although Atlantic Yards stands as the tallest modular building in the world, while it was being constructed it was faced a myriad of issues that caused Forest City to part ways with its modular division. The tower ended up taking twice as long as scheduled to construct at a cost far more than projected. Half of the first 39 units suffered water damage and the interior finishes had to be replaced. Several steel chassis were damaged during transportation which caused the modules to be misaligned when installed. Forest City partnered with Skanska in manufacturing the modules at their plant in Brooklyn's Navy Yard. However, disputes over the scope of work broke out between the two firms causing Skanska to temporarily close down the factory and eventually withdraw from the partnership. Skanska CEO, Richard Kennedy, claimed that misalignment issues were created because of the limited adjustability of the conventionally built steel frames. The plethora of issues associated with the construction and design of the Atlantic Yards tower prove that although modular construction has the possibility of revolutionizing the construction industry, more research and scientific studies are required to successfully implement modular construction practices in high-rise buildings.

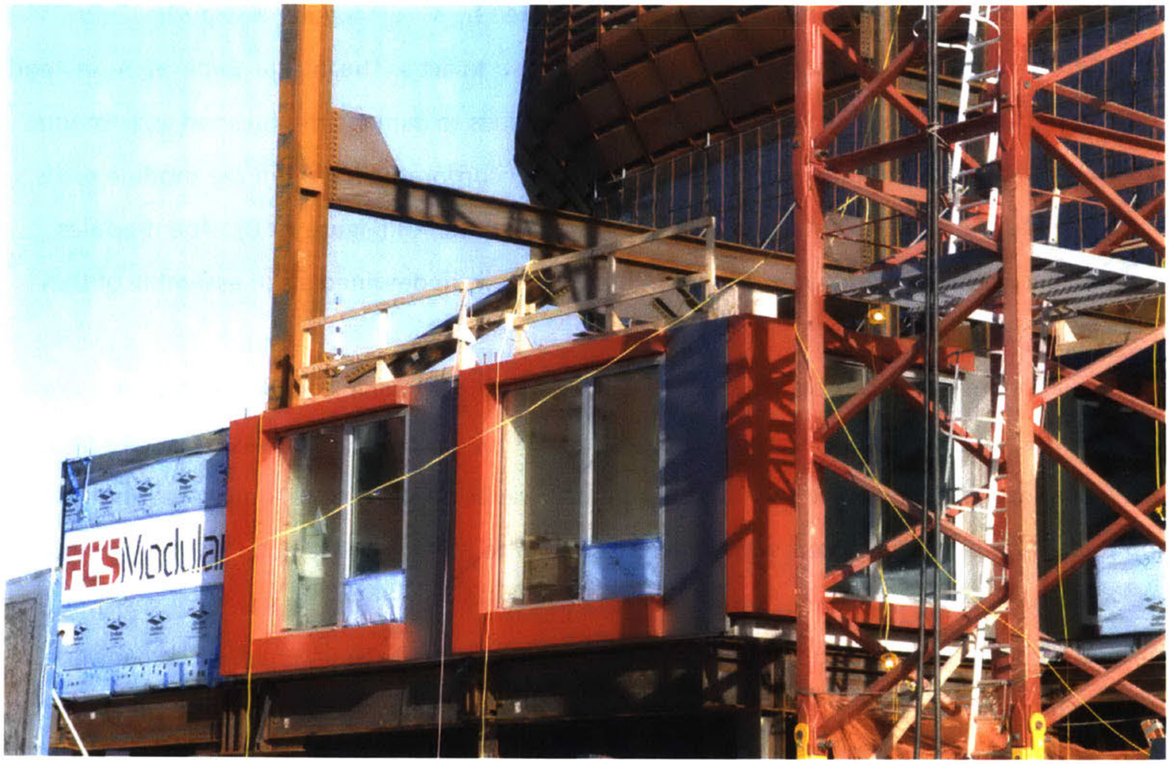


FIGURE 2.20 - ATLANTIC YARDS CONVENTIONALLY BUILT BRACED FRAMES (FIELD CONDITION 2014)

Chapter 3. Methodology

3.1 Problem Definition

Structural integrity or "robustness" is the ability of a structure consisting of separate components to remain stable and safe under extreme loading events (Zurick 2010). Structural integrity concerns the overall behavior of a building rather than the performance of individual elements and ensures that all structural components act as a single unit. It prevents global collapse under accidental loading or local failure.

Buildings built with conventional construction practices include floor and roof assemblies that behave as continuous diaphragms. The diaphragms which are integrated with the structural framing provide a continuous load path and ensure structural integrity is maintained under accidental loading. The discrete nature of modular construction naturally creates discontinuities which complicates the ability of a modular building to remain structurally integrated. To achieve structural integrity in a modular building, interconnections must support adequate tying action between adjacent modules to provide for alternative load paths. If no alternative load path is established than failure of a single interconnection may lead to collapse of the entire building (Sharafi et al. 2017). Including redundancy in the design can help ensure structural continuity is maintained in the event of extreme loads.

3.1.2 Testing Structural Integrity

To ensure the structural integrity of modular buildings and ensure modular structures are capable of localizing the effects of accidental damage the following guidelines have been established by Gorgolewski et al. (2001). For interconnections in modular floors and roofs the resultant tying forces should be arranged in continuous lines and in two directions which are approximately orthogonal wherever possible. Steel members of a modular building which act as interconnections should be capable of resisting the factored tensile loads to ensure structural integrity. For pre-fabricated building composed of four-sided modules Gorgolewski et al. (2001) recommend that the following tensile loads be applied as a separate load case to the interconnections of a modular building.

Table 3.1 Tensile loads resisted in modular buildings to ensure Structural Integrity (Gorgolewski et al. 2001)

For Floor Ties:	$0.5(1.4g + 1.6q) * L$
For Internal Ties:	$0.5(1.4g + 1.6q) * s * L$
For Edge Ties:	$0.25(1.4g + 1.6q) * s * L$

Where:

g is the specified dead load per unit area of the floor or roof

L is the average of any two adjacent spans between vertical supports

q is the specified distributed imposed floor or roof load

s is the mean transverse spacing of ties

In four-sided modules, interconnections should be distributed to ensure the entire module is effectively tied. The tying forces in the module walls should be equal to at least 1% of the factored vertical loads. If intermediate columns are part of the main structural system, the interconnections which tied the columns to their nearest edge beams should be capable of resisting a factored tensile load equal to or greater than that for an internal tie in Table 3.1. Splices in columns should be capable of resisting tensile forces that are at least two thirds of the factored design dead load that the column must support.

In addition to applying the load cases listed in table 3.1, there are several approaches used to check the structural integrity of modular buildings which are composed of four-sided modules. The first approach involves applying horizontal loads to a modular building and checking for compatible displacements of the modules. This approach is performed by applying notional static or dynamic loads to verify if the modular interconnections are able to prevent excessive eccentricities from forming between the modules. Notional loads are non-existent horizontal loads applied to a structure to account for eccentricities between modules. For modular construction Lawson et al. (2014) recommends that the notional horizontal force equal to 1% of the factored vertical load acting on each module is applied at each of the intermodular connections and the force may be shared between the structural elements of the adjacent modules. The notional load approach investigates whether a modular interconnection is able to resist the forces created by eccentricities between modules as depicted in Figure 3.1. Eccentricities occur due to inaccuracies in manufacturing and installing modules as well deformations which may occur during the transportation and lifting of modules. Although, as described in section 2.1, modules are designed to transfer gravity loads directly between module columns and walls, interconnections must be able to resist loads created by eccentricities between modules and provide lateral restraint. The interconnection design will affect the load transfer between modules and therefore impact the design of the module corner posts. For high-rise modular buildings, the notional load method may not be adequate to ensure structural integrity. Taller structures are subjected to increased horizontal forces and therefore the notional loads, which are applied only in cases where no wind or seismic forces applied, are of less importance.

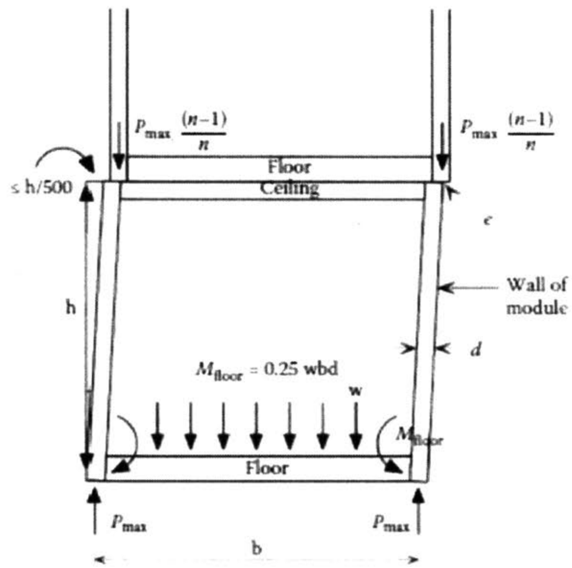


FIGURE 3.1- ILLUSTRATION OF ECCENTRICITY OF FORCES APPLIED TO THE COLUMNS OF A MODULE (LAWSON ET AL. 2014)

The second approach to checking the robustness of a modular building is investigating the behavior of the building in the condition of a notional module removal (Gorgolewski et al., 2001). This method involves omitting a structural module in the analysis of a modular building as shown in Figure 3.2. The approach is meant to simulate the complete failure or removal of a module so that the rest of the structure cannot depend on it to support any loads. After the notional module removal, the building as a whole must remain stable and any effects to the rest of the structure must be localized. The forces due to loss of the support are resisted by tying forces between the modules, as illustrated in Figure 3.3. The change of the load path requires that the modules be tied together with horizontal interconnections or span over the damaged area as part of a rigid frame. Several key performance criteria that are examined are the displacement and rotation of modules affected by the notional element removal. The affected structural elements must resist the redistributed loads without becoming overstressed. The additional stresses the members will have to resist depend on the loads carried by the affected modules and the design of the interconnections.

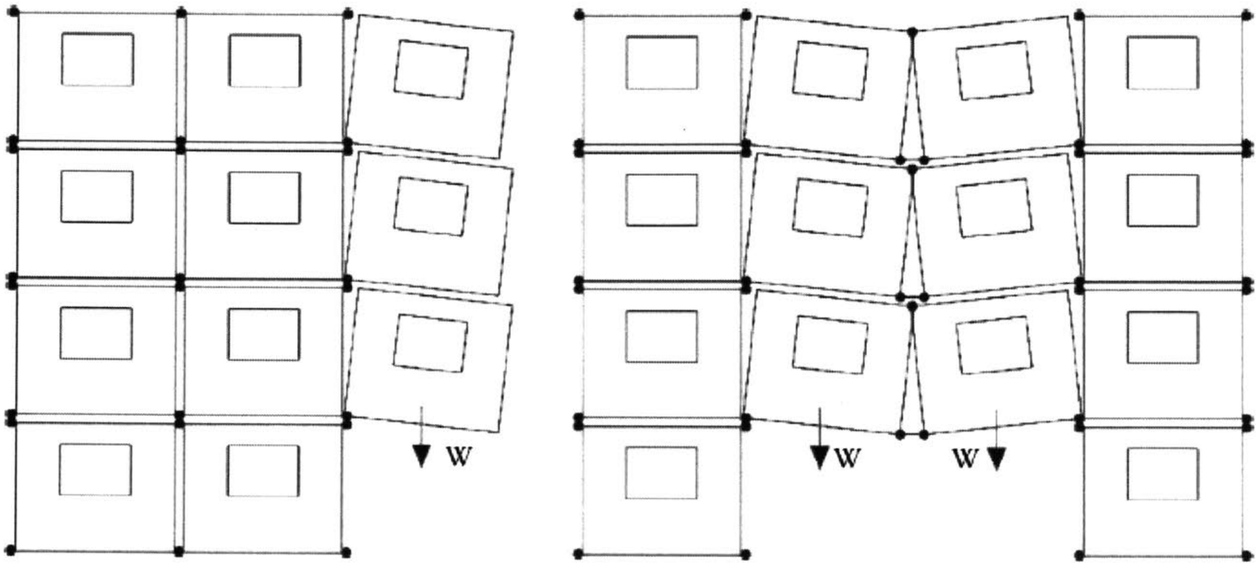


FIGURE 3.2 - STRUCTURAL INTEGRITY SCENARIOS IN MODULAR BUILDINGS (LAWSON ET AL. 2014)

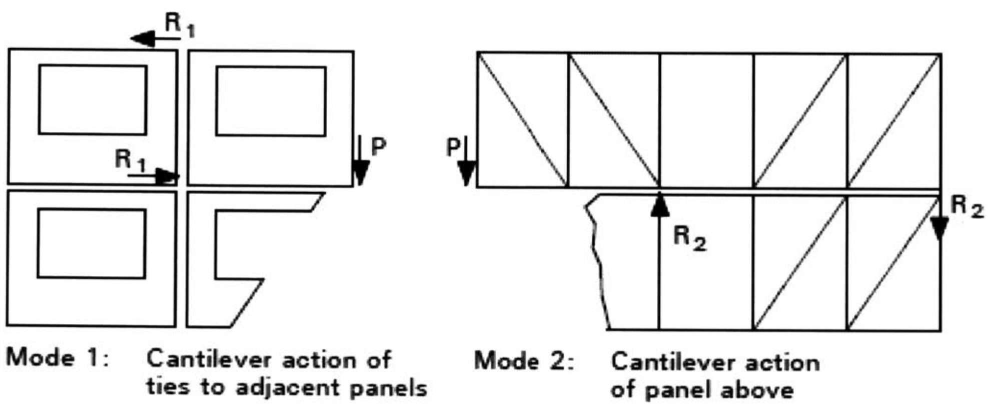


FIGURE 3.3 - CANTILEVER ACTION OF MODULES (GORGOLEWSKI ET AL. 2001)

3.2 Procedure for Structural Integrity Study

The thesis goal is to conduct a study which investigates how pinned, rigid, and post tensioned interconnections affect the structural integrity of a modular building. Several modular buildings of different story heights, ranging from 7 to 20 stories, are modelled using the computer program SAP2000 v18. SAP2000 is a structural analysis and design program developed by Computers & Structures Inc. (2016) The structural integrity of the modular buildings will be examined using the notional element removal described in Section 3.12. A more detailed description of the modelled building and the interconnections examined is presented in Section 3.3 and Chapter 4.

The aim of the study is to answer:

1. How do pinned, fixed, and post tensioned modular interconnections affect the structural integrity of a modular building?
2. How do modular interconnections affect load transfer under notional module removal?
3. Are post tensioned modules a viable lateral force resisting system?

In order to achieve the thesis goals, the following steps are completed:

1. Define the geometry of the modular building to be investigated.
2. Identify how the selected modular interconnections will be modelled
3. Build the analytical model for each prototype of the modular building
4. Use SAP2000 to model notional removal of structural elements and verify the integrity + behavior of the system

3.2.1 Model Definition

The first step is to define the modular building geometry which the structural integrity study will be based on. As described in section 1.1.1 modular construction is most often applied to buildings with standardized rooms. Economy of scale is achieved in manufacturing when modules of the same size are used. Therefore, in this study, in the modelled modular building all modules in will have the same dimensions.

Figure 3.4 depicts how typical modules are modelled in the building prototypes. Each module is 30 ft long, 10 ft wide, and 10 ft tall. The pre-fabricated units are all corner supported modules, with posts only at the corners and edge beams spanning between the posts. The module in Figure 3.4 represents a simplified but valid representation of a typical corner supported module such as the one shown in Figure 2.1b. The module beams are designed as HSS sections of the size 6 in x 3 in x ½ in. The columns are all designed as Square HSS sections with ½ inch thick walls. The column cross section dimensions vary from 5 to 8 inches depending on the modular assembly prototype and which story the module is located. The selection of the module columns is automated with SAP2000 based on the loads defined in section 3.2.2. Individual floor purlins are not included in the building prototypes. Steel modules are typically manufactured to be rigid so they do not deform when transported or lifted by cranes. The floor and ceiling assemblies are modelled as rigid diaphragms and the corner connections between the module columns and beams are modelled as fully fixed.

A plan view of the typical floor layout used in each module prototype is shown in Figure 3.5d. Each floor is composed of 8 modules arranged in a 2x4 layout so that the total building width is 41' 6" and the total building length is 60' 6". The prototypes that are studied include 7, 10, 15, and 20 story modular buildings (Figure 3.6). Each prototype has the same orientation of modules in plan view (2x4) as that shown in Figure 3.5d.

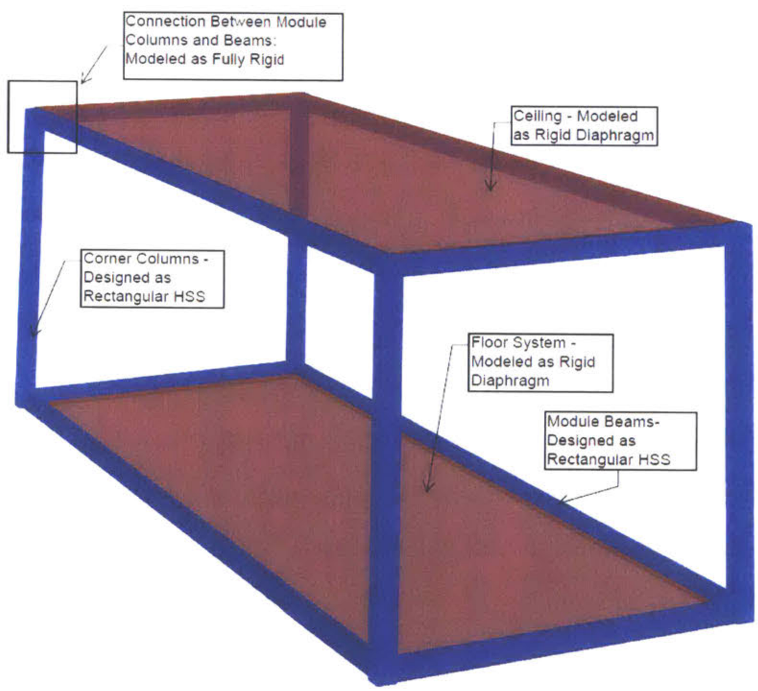


FIGURE 3.4 - TYPICAL MODULE USED IN BUILDING PROTOTYPE

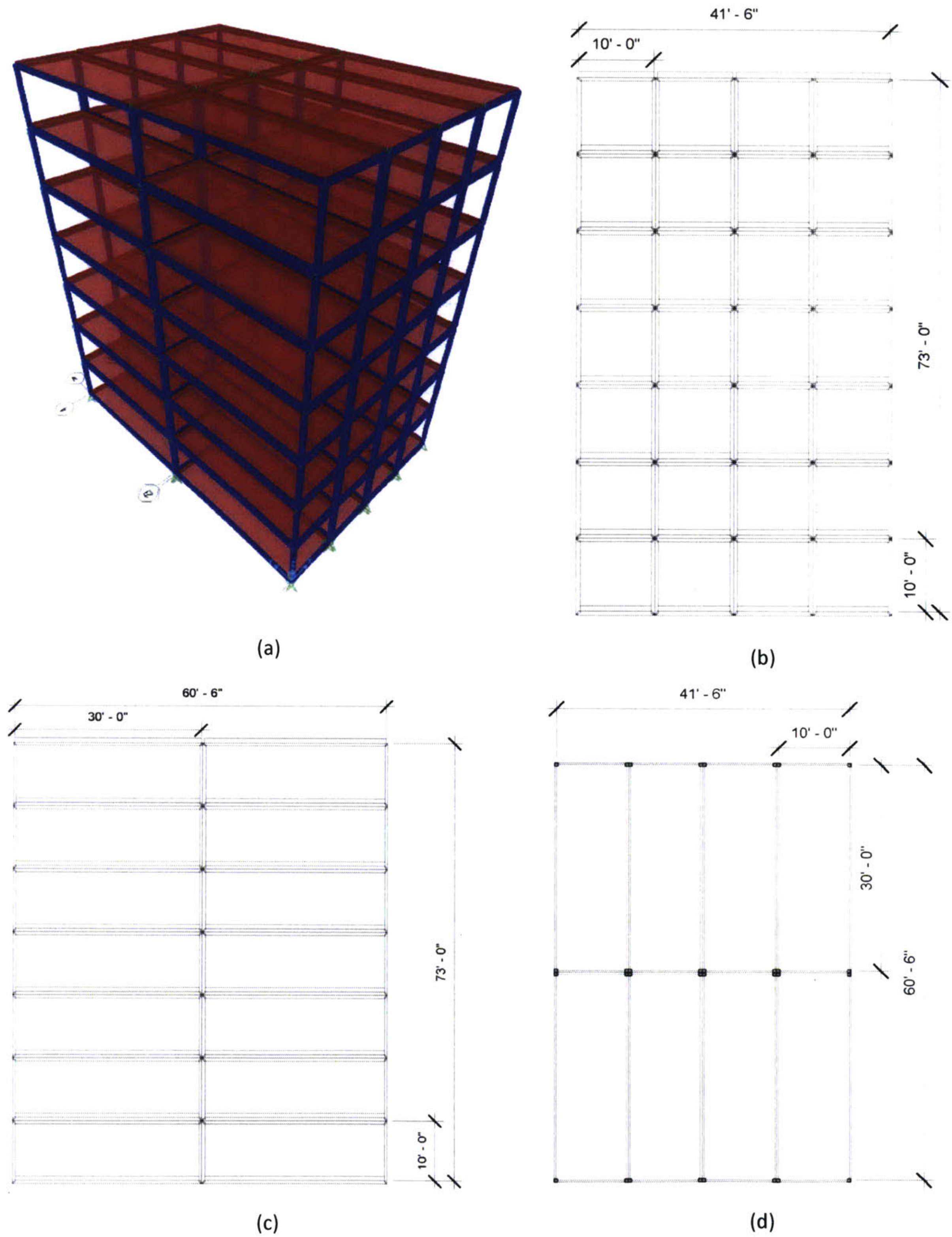
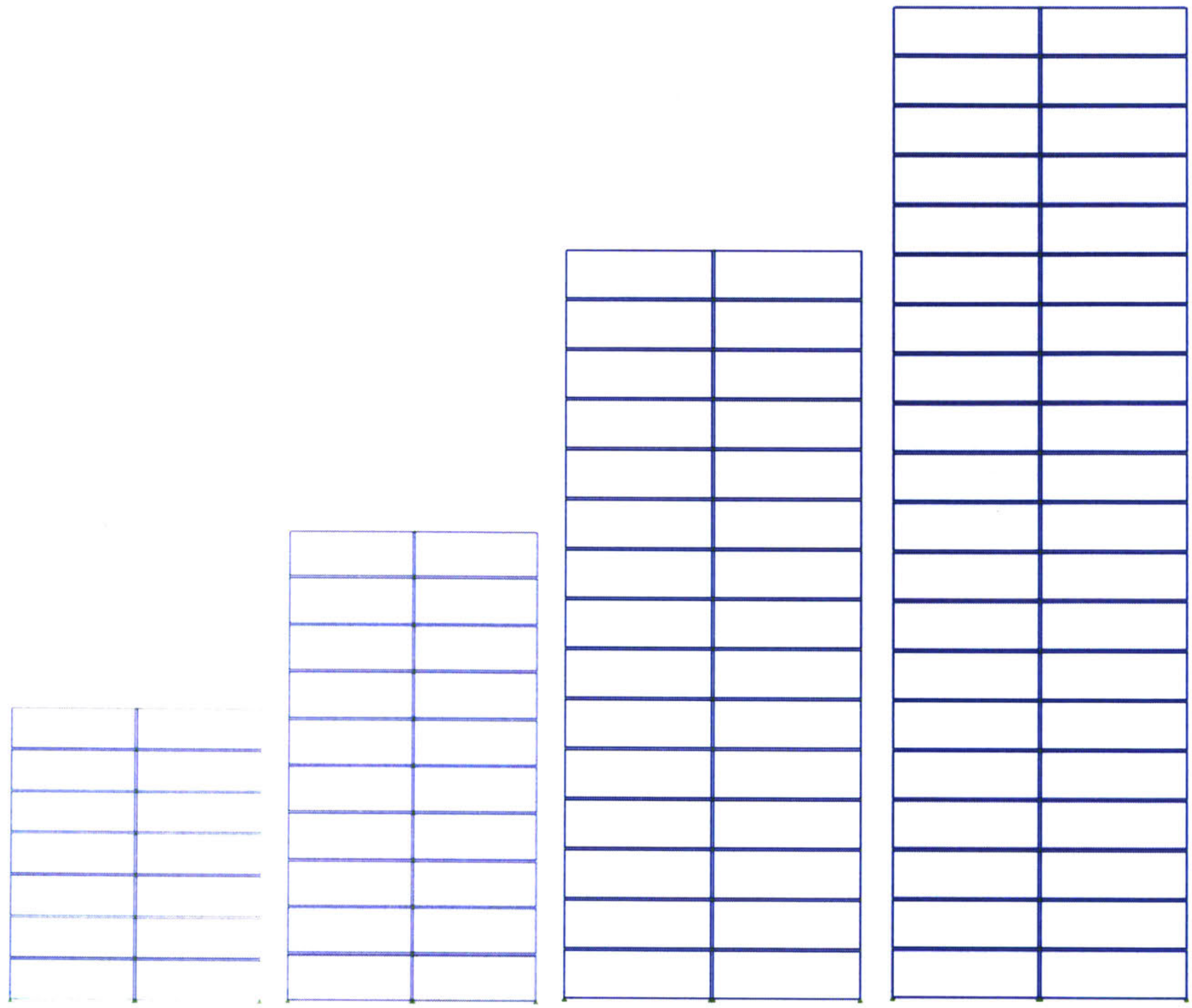


FIGURE 3.5- 3D FINITE ELEMENT MODEL FOR 7 STORY STRUCTURE. (A) ISOMETRIC VIEW (B) FRONT ELEVATION (C) SIDE ELEVATION (D) LONGITUDINAL SECTION VIEW



(a) (b) (c) (d)

FIGURE 3.6 - MODULAR PROTYPE SIZES (A) 7 STORY (B) 10 STORY (C) 15 STORY (D) 20 STORY

The building prototypes consist of corner supported modules which are only interconnected at their corners. Figure 3.7 shows a zoomed in view of the building prototype which depicts how the modules are linked to each other with interconnections which are modelled in SAP2000 as link elements. The link elements may transfer shear, moments and axial forces between adjacent modules. The forces each link transfers are based on the applied loads and configurations of the modular assemblies.

Separate building prototypes are modeled which include horizontal and vertical pre-stressed components. Figure 3.8a depicts a zoomed in view of the horizontally post tensioned modular building system. Post tensioned cables, modelled as tendon elements, run through the center of the modular HSS beams. A single tendon is placed in each edge beam that runs parallel to the building plane shown in Figure 3.5b and is anchored at both exterior columns. The vertical post tensioning system is shown in Figure 3.8b. It includes a single tendon which runs through each column and is anchored at the top and bottom of the building. All tendons incorporated in the building prototypes are modeled after Williams Engineering post-tensioned systems; each tendon is 1-1/4" in diameter and is prestressed depending on the modular prototype they are used in. The tendon elements are prestressed 35 kips in the 7 story prototypes, 50 kips in the 10 story prototypes, 75 kips in the 15 story prototypes, and 100 kips in the 20 story prototypes. The post tensioned modular systems are described in more detail in section 4.4.

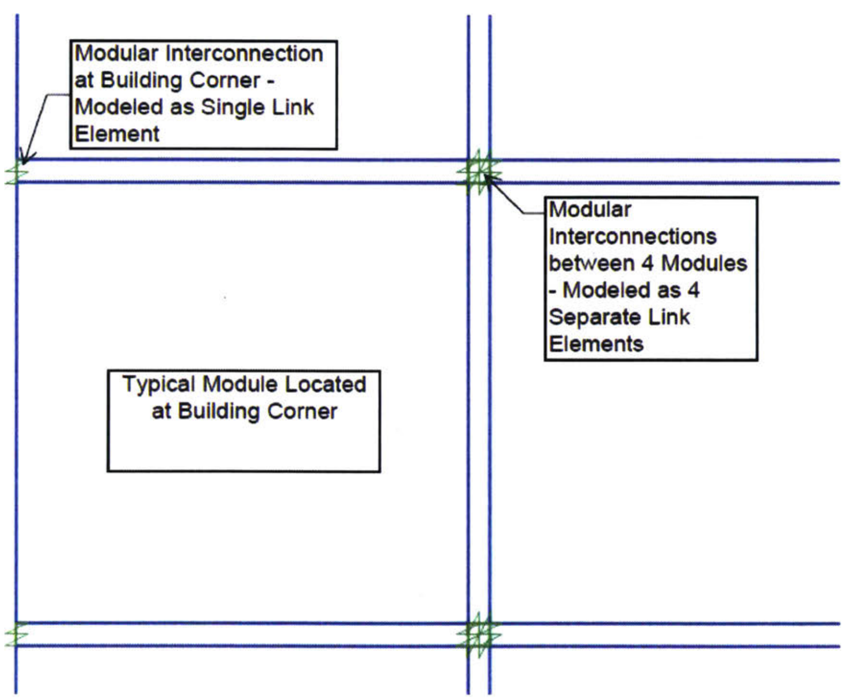
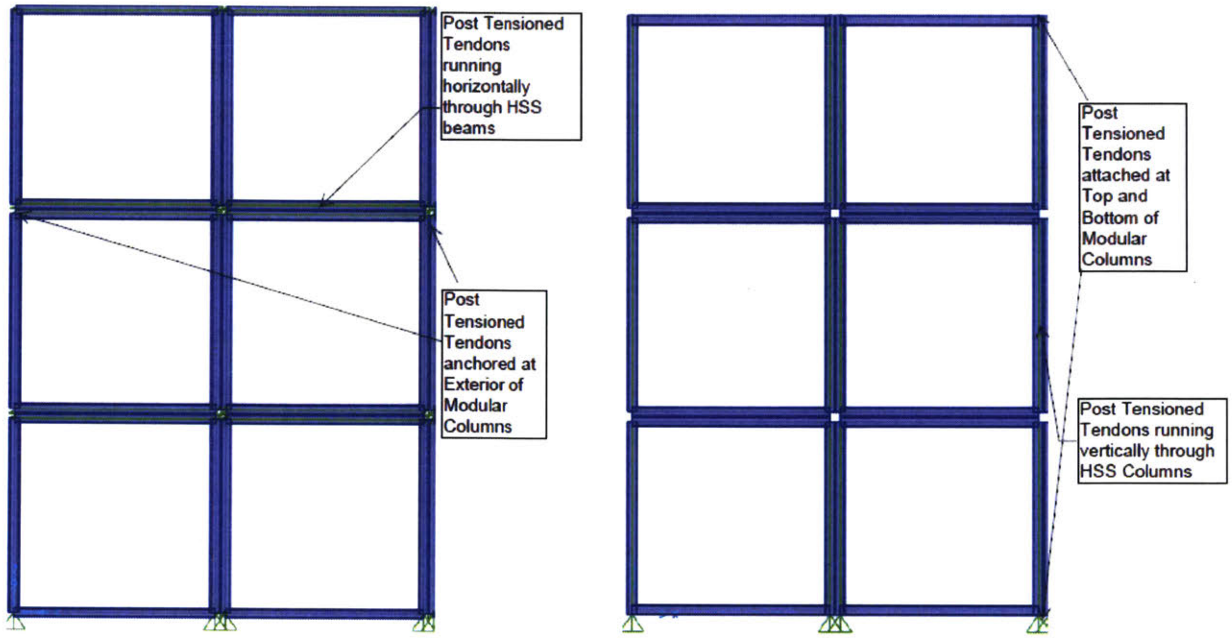


FIGURE 3.7 - ZOOMED IN VIEW OF INTERCONNECTIONS



(a) (b)

FIGURE 3.8- POST TENSIONED MODULE PROTOTYPES (A) HORIZONTAL POST TENSIONING (B) VERTICAL POST TENSIONING

The building is designed to be located in Manhattan, NY. The applied loads are based on ASCE 7-10, and the building is designed to be compliant with ASIC (2016) guidelines.

The loads applied to the model are as follows:

Dead Load

- 60 psf, applied uniformly across the floor of each module

Live Load

- 50 psf, applied uniformly across the floor of each module

Wind Load

- 98 mph basic wind speed, Directionality Factor =0.85, Exposure Category B, Gust Factor= 0.85, Enclosed Building

Seismic Load

- Seismic Importance Factor = 1.25, Structural Response Coefficient = 3.00, Site Class B, Seismic Coefficients for Zip Code 10034, (Upper West Side, Manhattan NY) given by the United Stated Geological Survey (USGS)

3.2.2 Varying Parameters and Test Scenarios

In the study, the notional element removal approach is used to study the structural integrity of several modular building prototypes. In the prototypes the following parameters are varied to investigate their effects of the global stability and design requirements of the modeled modular steel building: interconnection fixity, module post tensioning, and building aspect ratio.

The cases which are tested are as follows:

- Interconnection fixity is varied between conditions equivalent to those in fully pinned and fully rigid structural steel connections
- Post tensioning is used horizontally and vertically, as well as not used at all.
- The corner ground floor module, and middle ground floor module are notionally removed.

The control state is defined as the case where the modular building consists of intermodular connections which are modeled as pinned, and where no post tensioning is introduced.

From each testing case the following values are obtained:

i. Maximum horizontal diaphragm displacement under controlling lateral force
ii. Maximum modular vertical displacement due to notional module removal Δ_1
iii. Module rotation due to notional element removal
iv. Percent change of each parameter with respect to those obtained for the control case

Chapter 4. Modular Connection Study

4.1 Examination of Various Modular Interconnections

Modular construction is increasingly being used more often in high-rise buildings like Atlantic Yards and Victoria Hall. The design of interconnections, which transfer horizontal forces within modular buildings and impact their structural stability, become even more critical for taller modular buildings which experience greater lateral loads and accumulated eccentricities. Modular interconnections may consist of bolted members, welded members or interlocking systems. The focus of Chapter 4 is an assessment of the influence of simple pinned, rigid, and post tensioned modular interconnections on the structural behavior modular buildings

4.2. Simple Pinned Intermodular Connection

The most common method of connecting modules is attaching them at their corners with mechanical fasteners. These modular interconnections usually consist of an arrangement of horizontal and vertical plates that are bolted to the external faces of modules. Figure 4.1 depicts a standard variation of one these connections.

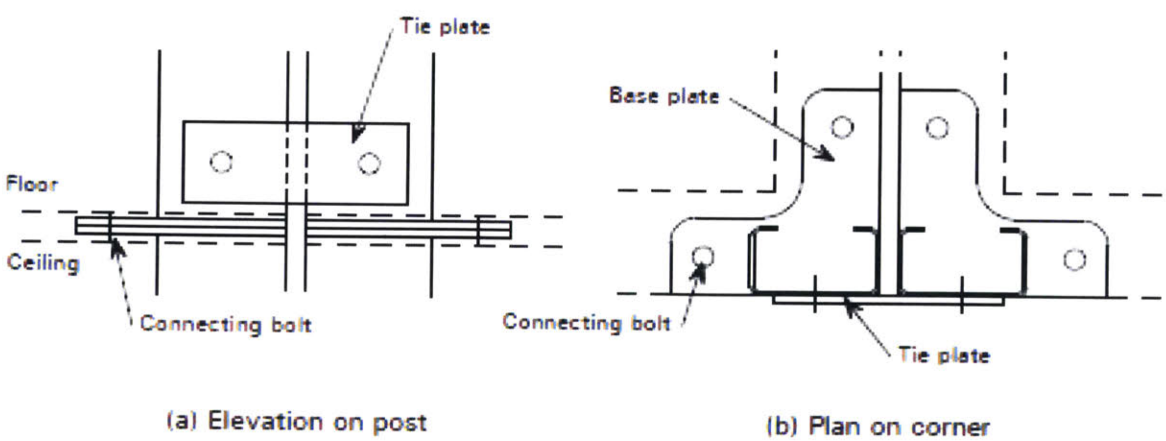


FIGURE 4.1 - SIMPLE PIN CONNECTION BETWEEN MODULAR UNITS (GORGOLEWSKI ET AL. 2001)

4.2.1 Problem Definition

A simply pinned modular connection is the control case for this study because of its common use in the modular construction industry. The rigid modular interconnection and the post tensioned interconnection will both be compared to the pinned connection to see how they impact the behavior of modular building. The simple pin connection is modeled in the building prototype by using link elements at the corners of each module as shown in Figure 3.7. Although the connection shown in picture 4.1 is not a true pin and will resist some rotation, based on the results from previous studies, the link elements in Figure 3.7 are modeled to transfer shear and axial forces and no moment.

The research completed on modular steel buildings by Annan et al. (2009) provides insight on how to model a simple steel connection like the one depicted in Figure 4.1. The bolted plate effectively acts as a simple pin connection that joins the adjacent modules (Figure 4.2). Other researchers have modeled this connection in a similar manner. Sharafi et al. (2017) used a separate node-to-node linear spring to represent each corner fastener; up to 12 linear springs are used at the intersection of 8 interior module corners. The arrangement of modules and linear springs he used to represent a modular building is shown in Figure 4.3. Sharafi et al. (2017) showed that modelling modular buildings in such a manner proved to be a valid representation of their actual behavior. Sharafi et al. (2017) conducted an experimental study using a shake table and high-speed camera to measure the accelerations and displacements of a modular assembly consisting of scaled modules with connections similar to those shown in Figure 4.1. Sharafi et al. (2017) then modeled the modular assembly in a finite element analysis program and found the modular building behaved in a similar manner. Following this work, the present study uses linear springs with zero rotational stiffness to model the behavior of pinned interconnections in the structural integrity study. To assess the impact the modular interconnections, have on the structural integrity of a modular building, SAP2000 is used to model notional removal of modules from the building prototypes show in Figures 3.5 and 3.6. Figure 4.4a depicts a close-up view of a building prototype with a ground floor corner module notionally removed.

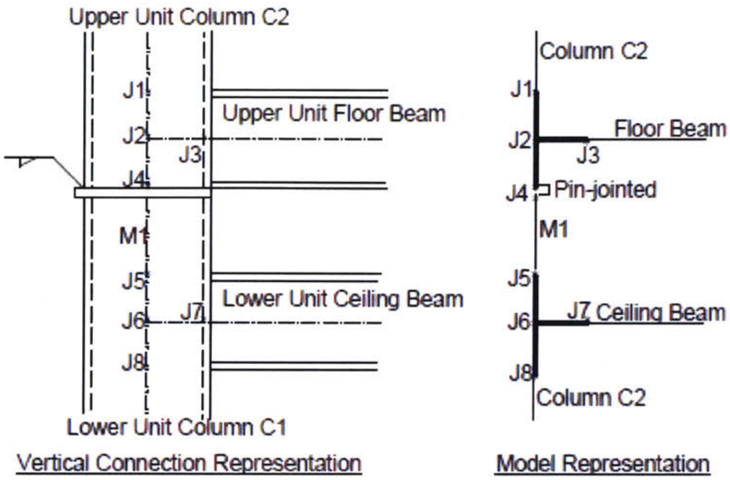


FIGURE 4.2 - MODEL OF VERTICAL CONNECTION OF MODULAR UNITS (ANNAN ET AL. 2009)

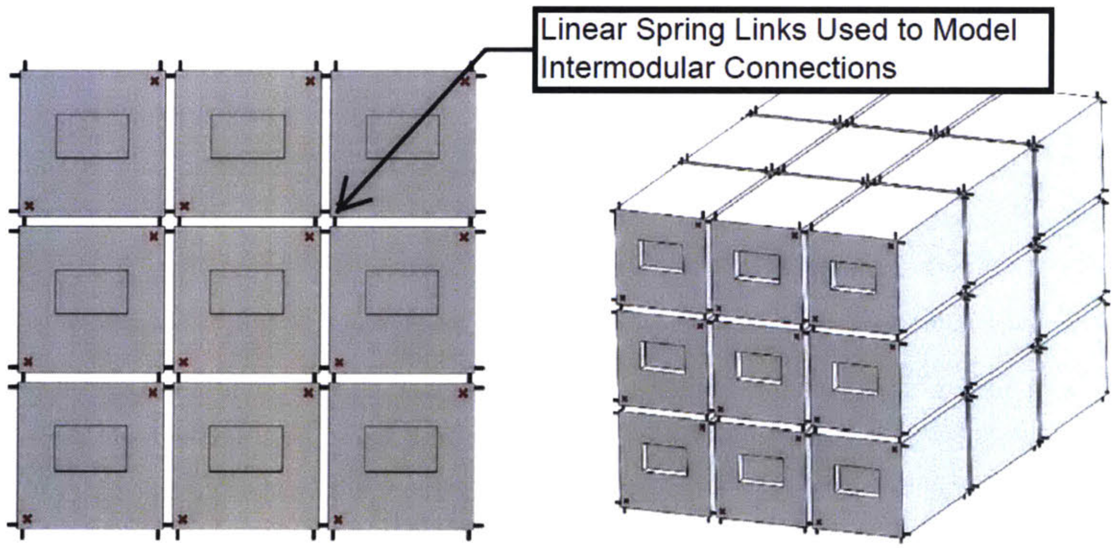


Figure 4.3 - Modelling of modules and Interconnections. (Sharafi et al. 2017)

Modules are discrete structural units which have to be tied together to act as a unified structure. The notional module removal shown in Figure 4.4a depicts the structural collapse which would occur if loads were not redistributed through the modular interconnections. In Figure 4.4a, Δ_1 is the maximum vertical displacement of the module directly above the notionally removed corner module. Figure 4.4b depicts a close-up view of a building prototype where the two intermediate ground floor modules are notionally removed simultaneously. Structural collapse is shown to occur by the intermediate modules rotating inwards because of the lack of an alternate load path. In Figure 4.4b, Δ_2 is the maximum vertical displacement of the module directly above the notionally removed intermediate module.

The vertical displacements due to the notional removal of ground floor modules are found under service dead + live loads. The behavior of the pre-fabricated units caused by notional removal of ground floor modules is described in more detail in section 4.3 and 4.4. The total building drift of each building prototype due to service wind loads is found as shown in Figure 4.5a. Wind load are calculated using an automatic lateral load pattern defined in SAP2000 v18. The resultant wind forces are calculated as per the Directional Procedure outlined in ASCE (2010). Figure 4.5b shows the 4 load cases applied to the model. The resultant wind force is at the center of each diaphragm.

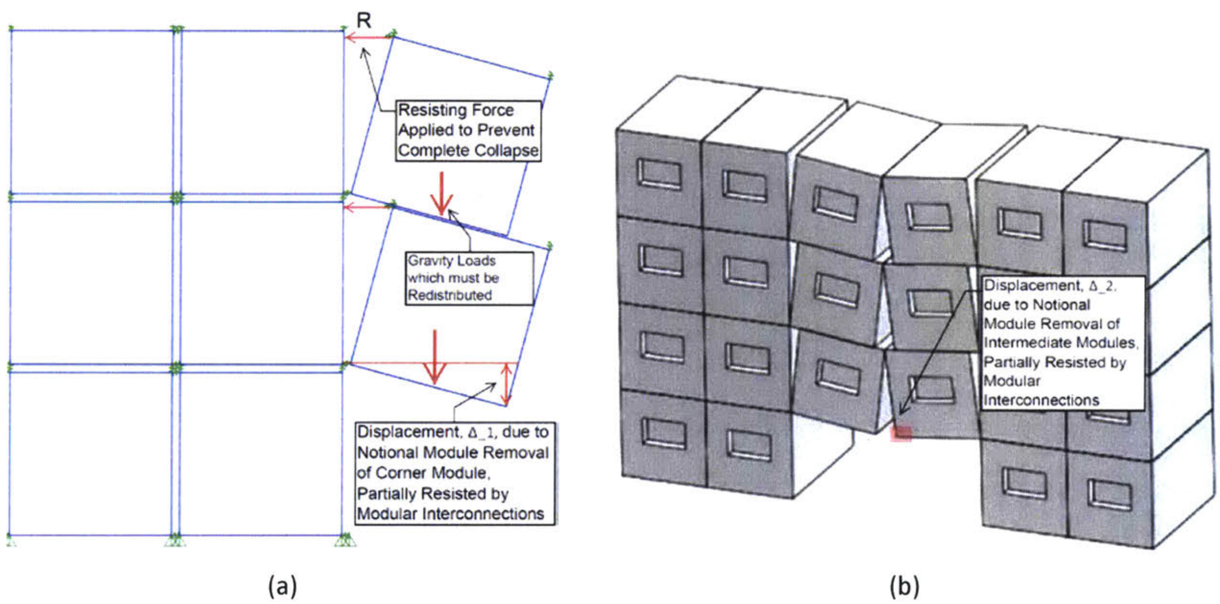


FIGURE 4.4- VERTICAL DISPLACEMENT DUE TO NOTIONAL MODULE REMOVAL (A) CORNER MODULE (B) MIDDLE MODULES (SHARAFI 2017)

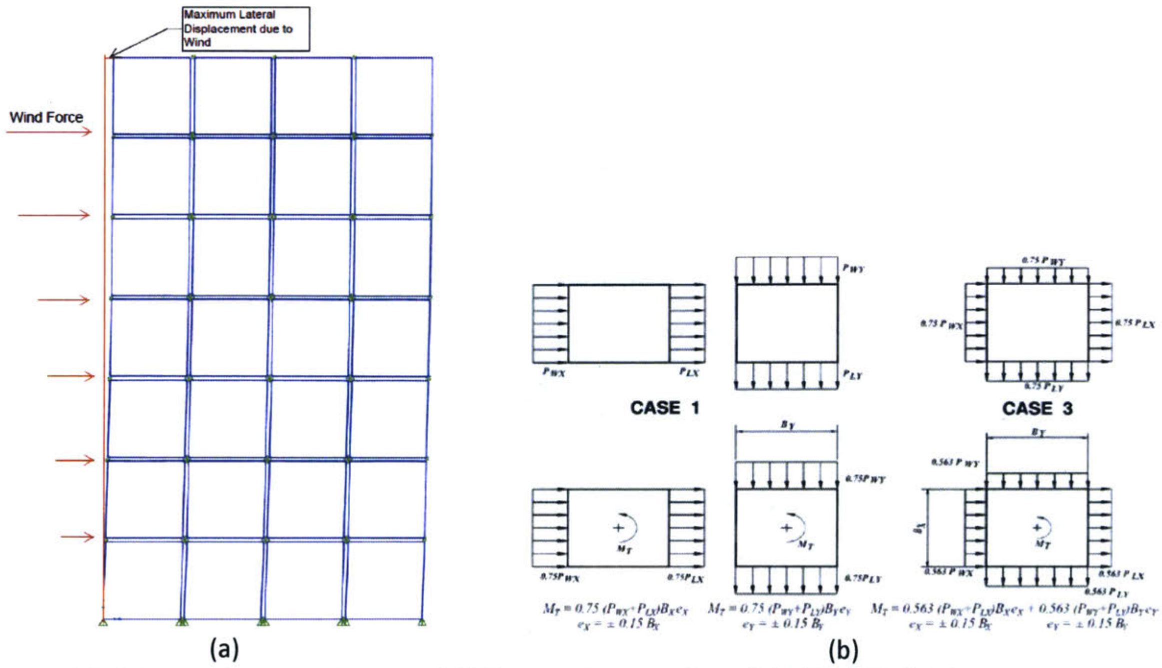


FIGURE 4.5- WIND ON MODULAR BUILDING, (A) MODULAR BUILDING DRIFT (B) ASCE (2010) WIND LOAD CASES

4.2.2 Results

As discussed in section 2.2, diagonal bracing and reinforced concrete cores are the most common types of lateral force resisting systems used in steel modular buildings. The building prototypes, shown in Figures 3.5 and 3.6, when designed with simple pin interconnections are not meant to represent a valid modular assembly on its own without including additional bracing systems. The main reason for modelling a modular building in this manner was to determine the amount of additional bracing needed to support the pin connected modular assembly and compare it to the rigidly connected and post tensioned protypes. The values for the horizontal displacement due to wind and vertical displacements due to the notional removal of modules for pin connected building prototypes at 7, 10, 15 and 20 stories are shown in Table 4.1.

A pin connected modular assembly is not meant to represent a realistic modular building on its own. However, the vertical module displacement due to the notional removal of modules is found for pin connected modular buildings beyond 7 stories assuming a concrete core is used in the building. The inclusion of a central concrete core may affect the values obtained for Δ_1 or Δ_2, depending on the connections used between the modules and the core. The guidelines in the commentary for Chapter L of AISC (2016) state that deflections which exceed the 1/150 of the length of cantilevers are visible and may lead to general architectural damage or cladding leakage. The beams spanning across the notionally removed module in Figure 4.4a effectively become cantilevers after the module is removed. 1/150 of the 10-foot module beam length is only 0.8 inches which is significantly smaller than the Δ_1 and Δ_2 values listed in Table 4.1. As described by Lawson et al. (2014) and Sharafi et al. (2017) notionally removing modules and determining the effect the removal has on the rest on the structure is a valid approach to testing the structural integrity of the building. However, this approach was not feasible for the 15 and 20 story prototypes that were tested with pin connections because of the excessive loads that were experienced by the modules.

TABLE 4.1 – BUILDING DRIFT AND MAXIMUM MODULE DISPLACEMENT FOR BUILDING PROTOTYPES WITH PINNED MODULAR INTERCONNECTIONS

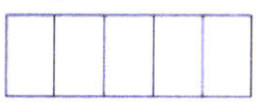
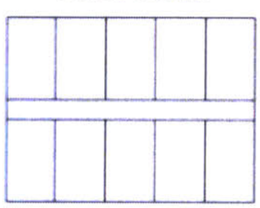
Number of Stories	Total Building Drift (in)	Maximum Module Displacement due to Notional Module Removal of Corner Module, Δ_1 (in)	Maximum Module Displacement due to Notional Module Removal of Intermediate Module, Δ_2 (in)
7	5.52	3.8	3.5
10	-	8.55	7.88

Because of its cost efficiency and ease of use the simple modular pin connection may still be the preferred interconnection for certain module assemblies such as low rise and portable structures. The maximum height of a group of modules depends on the stability provided under wind or seismic forces. The Steel Construction Institute (SCI, 2018) provides the guidelines shown in Table 4.2 on the use of four-sided modular assemblies connected with simple mechanical fasteners. The maximum story heights listed in Table 4.2 are governed by stability under wind action based on wind loading in the Midlands of England. The "double line" modular construction category in Table 4.1 closely matches the geometry of the modular building modelled in this study. The Steel Construction Institute recommends that modular building composed of four-sided modules, such as the one depicted in Figure 2.1a, may be built up to 6 stories tall. Table 4.1 details the results obtained for a 7 story modular buildings composed solely of corner supported modules without any lateral force resisting systems besides the modular interconnections. The maximum horizontal diaphragm displacement for the 7-storey, 73 ft tall prototype is found to be 5.52 inches under wind loads when a pinned interconnection is used. The total building drift coefficient is calculated as:

$$\frac{Total\ Building\ Height}{Maximum\ Horiztonal\ Diaphram\ Displacment} = \frac{73\ ft * 12\frac{in}{ft}}{5.52\ in} = 158.7$$

Serviceability guidelines recommend that the total building drift due to wind should range between H/350 and H/500 for steel structures (AISC 2016, Chapter L). Lateral deflections greater than the guidelines may cause building residents to feel the building oscillate under wind. As expected a 7-storey modular building with pinned interconnections and no other stabilizing system was unable to provide sufficient lateral stability under wind action for New York wind loads. The ratio of the building height to its horizontal displacement was H/158 which is twice as much as AISC (2016) recommends. However, according to the SCI (2018), a similar modular building would be able to be sufficiently laterally braced if four-sided modules were used instead of corner supported modules because of the diaphragm action created by the cold formed steel studs and sheathing boards in the module walls.

TABLE 4.2 – TYPICAL MODULAR BUILDING HEIGHT DEPENDING ON STABILIZING SYSTEM USING FOUR-SIDED MODULES (SCI, 2018)

Form of modular construction	Bracing requirements	Limit on size in concept design	
		Typical max. number of storeys	Min. number of modules in a group
Single line of modules 	No additional bracing	3	5
	With additional bracing in gables	5	8
	With additional stabilising core	7	No limit
Double line of modules with central corridor 	No additional bracing	6	2 x 8
	With additional bracing in gables	8	2 x 10
	With additional stabilising core	10 - 12	No limit

4.2.3 Discussion

As discussed in section 2.3, the most commonly used lateral force resisting system for modular buildings over 7 stories tall is a reinforced concrete core. Steel modules transfer lateral loads through their individual diaphragms and modular interconnections to the core which is often located in the center of building. Taller buildings are often governed by serviceability criteria instead of strength limits. In an area with relatively high wind loads and lower seismic loads, the size of a buildings lateral force resisting system is often governed by the maximum wind drifts a building is allowed to experience. Therefore, the preliminary design of a lateral force resisting system often starts by sizing it so that its maximum drift falls within the serviceability criteria. If a concrete core is chosen as the main lateral force resisting system, it may be modelled as a cantilevered beam so that its deflection due to wind can be found.

The concrete core is placed in the center of the modular building as shown in Figure 4.6. It is sized as 17' x 7' which would be large enough to hold a typical staircase and a single elevator. The x direction is defined as the direction parallel Its thickness is then calculated by finding the moment of inertia which limits deflection to L/400, which is the desired maximum total building drift due to wind. The concrete core is assumed to be prismatic and the compressive strength of the concrete is assumed to be 5,000 psi.

The pressure caused by wind can be estimated to vary linearly with height for buildings studied here. Therefore, for a prismatic building, the resultant wind force, which is equal to the base shear, acts at approximately 2/3rd of the building height. Because the modules are connected with simple pin interconnections, the concrete core is assumed to resist the full wind load. Figure 4.7 depicts the simplified model of the concrete core. The maximum deflection of a cantilevered beam loaded by a concentrated force is calculated as:

$$\Delta_{max} = \frac{VB^2}{6EI_{core}}(3H - B)$$

Where:

V is the concentrated force

B is the distance from the fixed end to V

E is the modulus of elasticity

H is the core hiehgt

I_{core} is the moment of inertia of the concrete core

The American Concrete Institute (ACI 2014) allows the modulus of elasticity of concrete to be calculated as follows:

$$E_c = 33w_c^{1.5}\sqrt{f_c'}$$

Where:

W_c is the weight of concrete in pounds per cubic foot

f'_c is the compressive strength of concrete at 28 days (psi)

For 5 ksi concrete which weighs 150 pounds per cubic foot the modulus of elasticity is found to be 4,286 ksi. The required moment of inertia of the concrete core needed to limit the total building drift can then be calculated as:

$$I_{core} = \frac{V * \left(\frac{2H}{3}\right)^2}{6E\Delta_{max}} \left(\frac{7}{3}H\right)$$

A concrete core which houses a stair case or an elevator will have to have openings in it which are supported by link beams but can be assumed to be prismatic when calculating the preliminary thickness.

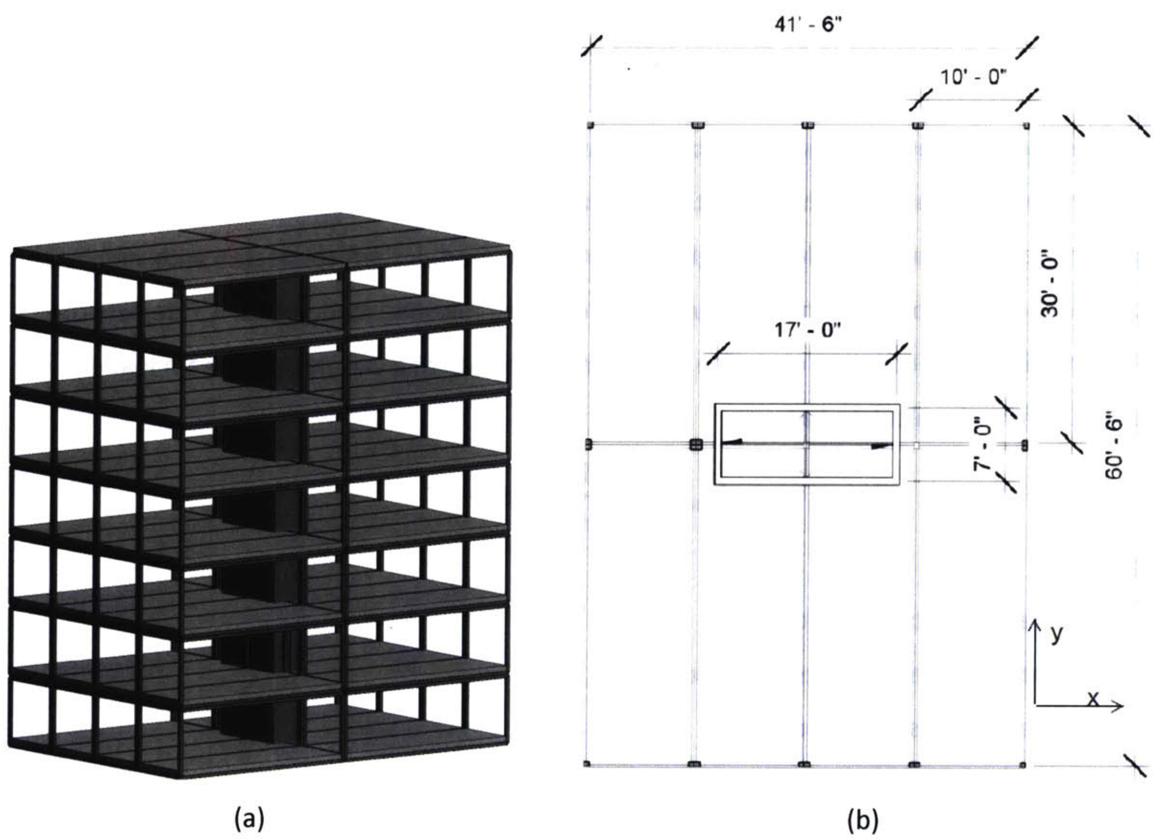


FIGURE 4.6 – MODULAR BUILDING WITH CONCRETE CORE (A) ISOMETRIC VIEW (B) CORE DIMENSIONS

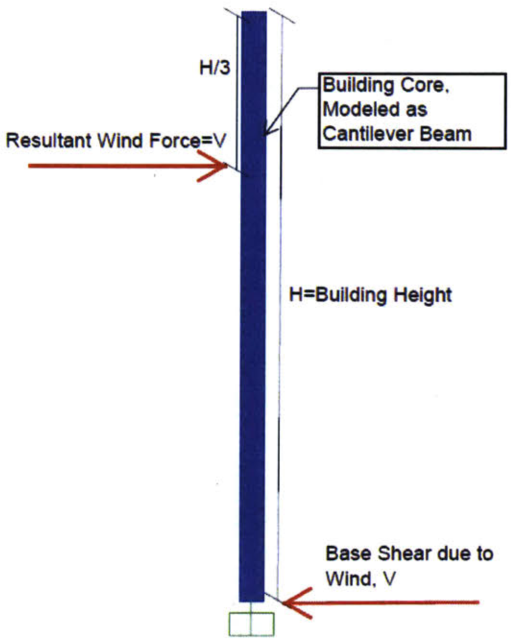


FIGURE 4.7 – CONCRETE CORE MODELED AS CANTILEVER BEAM

Table 4.3 shows the necessary concrete core thicknesses required to limit the total building wind drift to H/400 for each building prototype. For fireproofing and constructability purposes, cast in place concrete shear walls are often designed to be at least 8-12 inches thick. Therefore, the concrete cores for the 7, 10 and 15 story building prototypes would be overdesigned unless the core width or length was reduced. However, because a concrete core usually envelops a staircase or elevator shaft its dimensions cannot be reduced. Table 4.3 proves that cast in place concrete cores are not always the most efficient lateral force resisting systems for mid-rise modular buildings. Concrete cores, such as those depicted in Figure 2.3, made using a core-fast system or pre-cast modules allow for thinner shear wall thicknesses to be used because of the manner in which they are constructed.

TABLE 4.3 – CORE THICKNESS REQUIRED TO LIMIT TOTAL BUILDING DRIFT TO H/400 FOR PIN CONNECTED BUILDING PROTOTYPES

Number of Stories	H/400 (in)	Base Shear_x (k)	Base Shear_y (k)	Required I_x of Core (in^4)	Required I_y of Core (in^4)	Minimum Required Core Thickness (in)
7	2.2	71	48	166,974	530,077	1
10	3.1	115	78	1,080,358	2,180,321	2
15	4.7	203	139	5,523,301	9,662,274	4
20	6.3	298	204	17,040,591	27,338,805	10

4.3 Moment Resisting Intermodular Connection

Individual steel modules are most often constructed with rigid connections so that they do not deform before being assembled on site. Steel modules face potential accidental loads when they are transported and lifted with a crane. Deformed modules may not align properly with the rest of a modular assembly and may cause vertical or horizontal out of alignment. As detailed in section 2.2, the large module eccentricities compromise the structural integrity of a modular building. Figures 4.8 and 4.9 depict examples of welded and bolted modular connections which may are designed to transfer moments between the modular posts and beams.

The cast steel corner piece shown in Figure 4.8a is manufactured by Zekelman Industries and is called the VectorBloc. It is designed to form a rigid connection in corner supported modules composed of rectangular HSS beams and columns. The red lines represent a weld which is made between the VectorBloc, column and two beams. Welding moment resting connections in traditionally built steel buildings is expensive and time consuming. Because modules are assembled in a controlled factory setting, welded connections are much easier to assemble. Robotic welding arms may also be used to create a more consistent weld. Figure 4.9 shows a moment resisting connection made using a deep steel plate. The plate is welded to the modular column and bolted to the edge beam. In Figure 4.9 the steel module is constructed of hot rolled steel HSS columns and cold formed steel channel edge beams, however hot rolled steel channels or angles maybe used for the edge beams as well.

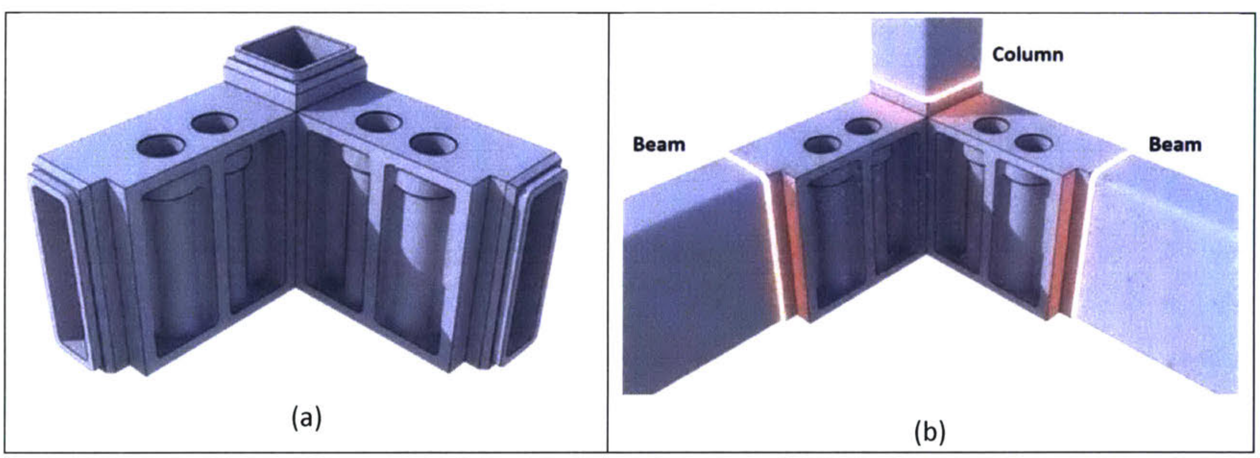


FIGURE 4.8 – RIGID MODULAR CONNECTION. (A) CAST IN PLACE STEEL CORNER (B) WELDING OF CORNER PIECE TO MODULE (ZEKELMAN INDUSTRIES, 2016)

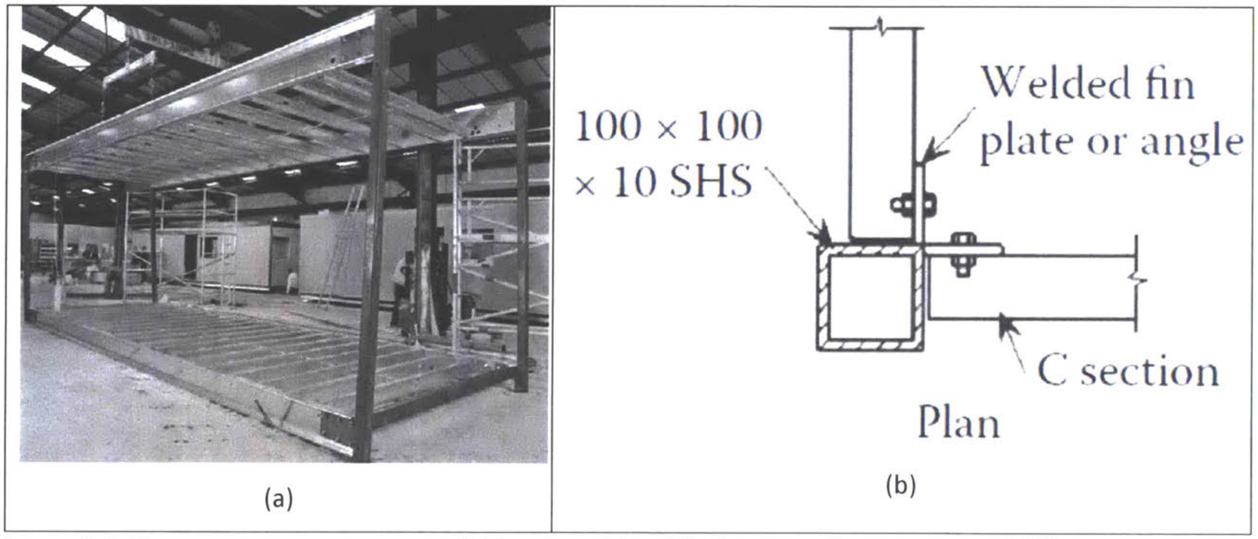


FIGURE 4.9- BOLTED MODULAR CONNECTION. (A) ISOMETRIC VIEW (B), PLAN VIEW (LAWSON ET AL. 2014)

The sides of the module depicted in Figure 4.9a act as a Vierendeel truss spanning between the corner columns. However, the ability to transfer moments between modules depends on the rigidity of the modular interconnections. As detailed in section 4.1.1, standard mechanical fasteners allow independent rotation of modules and therefore can not transfer moments between modules. From a review of case studies and interviews with modular industry experts it is believed that a moment frame system has never been used as the main lateral force resisting system in steel modular buildings taller than 3 stories. As detailed in section 2.2, the vast majority of modular buildings are laterally braced by either welding diagonal bracing inside module walls or connecting the modules to a reinforced concrete core. Although none have been implemented in actual buildings, several rigid modular interconnections have been proposed.

The modular interconnection depicted in Figure 4.10 is designed to transfer moments between corner supported modules that are equipped with VectorBlocs. Modules are vertically tied by deep steel bolts that screw into the bottom vector block. They are connected vertically with a gusset plate that is positioned in between all four modules and held in place by deep steel bolt which are placed through the top vector block and gusset plate and then screwed into the bottom VectorBloc. The resulting 12-way connection is able to transfer shear forces and bending moments between all four modules and create an effective moment frame system to resist lateral loads. The VectorBloc is also able to develop tensile tying forces between modules in the event of the loss of a support. Although the VectorBloc creates a moment resisting connection through the moment arm created by the deep bolts, it has so far only been implemented into modular buildings which also incorporate in wall bracing. The VectorBloc has been tested experimentally by Dr. Skreekanta Das and Dr. Oya Mercan from the Universities of Windsor and Toronto, and proven to be capable of being implemented into modular buildings taller than 80 stories (Zekelman 2016). The VectorBloc is capable of being used in high-rise building because of its small manufacturing tolerances. As described in section 2.3, the tallest modular building in the world, The Atlantic Yards Tower, faced numerous out of alignment issues because of excessive construction tolerances. The average manufacturing tolerance of steel modules is approximately ¼" (Lawson et al. 2014). VectorBlocs are based on plus zero, minus 1/16" tolerances controlled by precision fixtures and laser alignment systems.

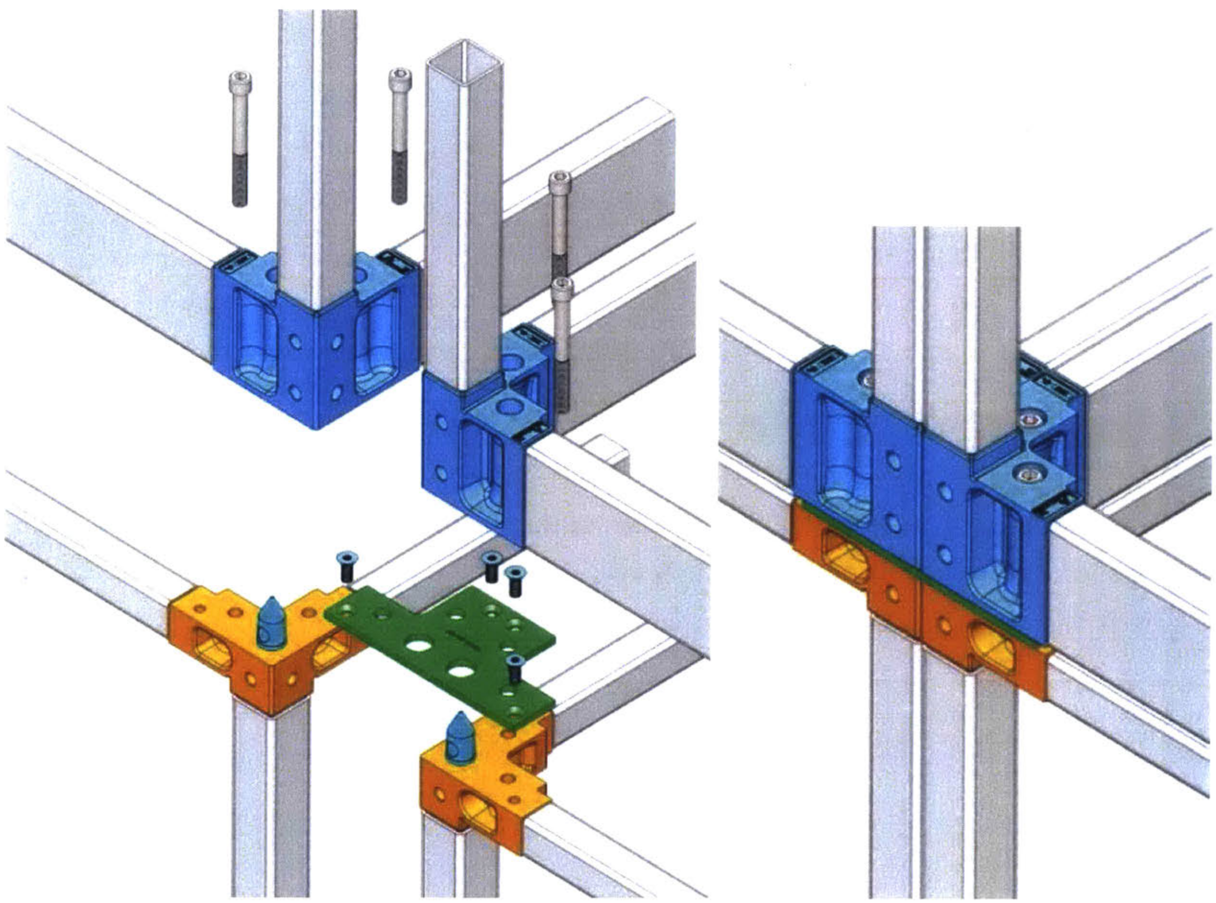


FIGURE 4.10 - MOMENT RESISTING INTERMODULAR CONNECTION (ZEKELMAN INDUSTRIES, 2016)

4.3.2 Results

The total module displacements due to the notional removal of ground floor corner and intermediate for the building prototypes with rigid connections are listed in Table 4.4. Figure 4.11 shows the forces developed by the modular interconnections when a corner ground floor module is notionally removed. Tying action is developed by tension in the modular interconnections to resist the module rotation depicted in Figure 4.4. For the seven story prototypes and others, this tying action created by tension forces in the modular interconnections is why the modular assemblies with rigid interconnections are able to resist most of the downward displacement caused by dead and live loads.

However, for taller building prototypes and larger loads the tying force in the rigid connections is not enough to prevent a more significant Δ_1 and Δ_2. For example, the 15 story building prototype has a 51.3% larger vertical displacement of the modules when it is modeled with pinned connections compared to rigid connections because the rigid connection creates a resistance to rotation along with a tension tying force. The interconnection can be modeled as a link or a spring element. In SAP2000 v18 a link element is used to connect two joints together. Each link element is assumed to be composed of six internal "hinges" or "springs", one for each of the six deformational degrees-of freedom. Each spring may then by composed of several components including springs and dashpots in series and in parallel. Figure 4.11b shows a link element with springs in three of the six deformation degrees of freedom: pure bending in the 1-2 plane, axial, and shear in the 1-2 plane. Shear and bending and torsion in the 1-3 plane are not shown but are the same as for 1-2 plane resistance as the link element shown in figure 4.11b. The simple pin connection modelled in section 4.1.1 was incapable of preventing interconnected modules from rotating independently and therefore it was assumed to have no bending stiffness. However, rigid modular connections resist both rotation and translation and therefore proper stiffness values must be assigned in each of the six deformational degrees of freedom.

TABLE 4.4 – STRUCTURAL BEHAVIOR OF BUILDING PROTOTYPES WITH RIGID CONNECTIONS

Number of Stories	Module Displacement due to Notional Module Removal of Corner Module, Δ_1 (in)	Module Displacement due to Notional Module Removal of Corner Module, Δ_1 (% Change from Pin Condition)	Maximum Module Displacement due to Notional Module Removal of Intermediate Module, Δ_2 (in)	Maximum Module Displacement due to Notional Module Removal of Intermediate Module, Δ_2 (% Change from Pin Condition)
7	3.6	-5.26%	3.3	-5.71%
10	5.81	-32.09%	5.32	-32.41%
15	9.37	-	8.58	-
20	15.11	-	13.85	-

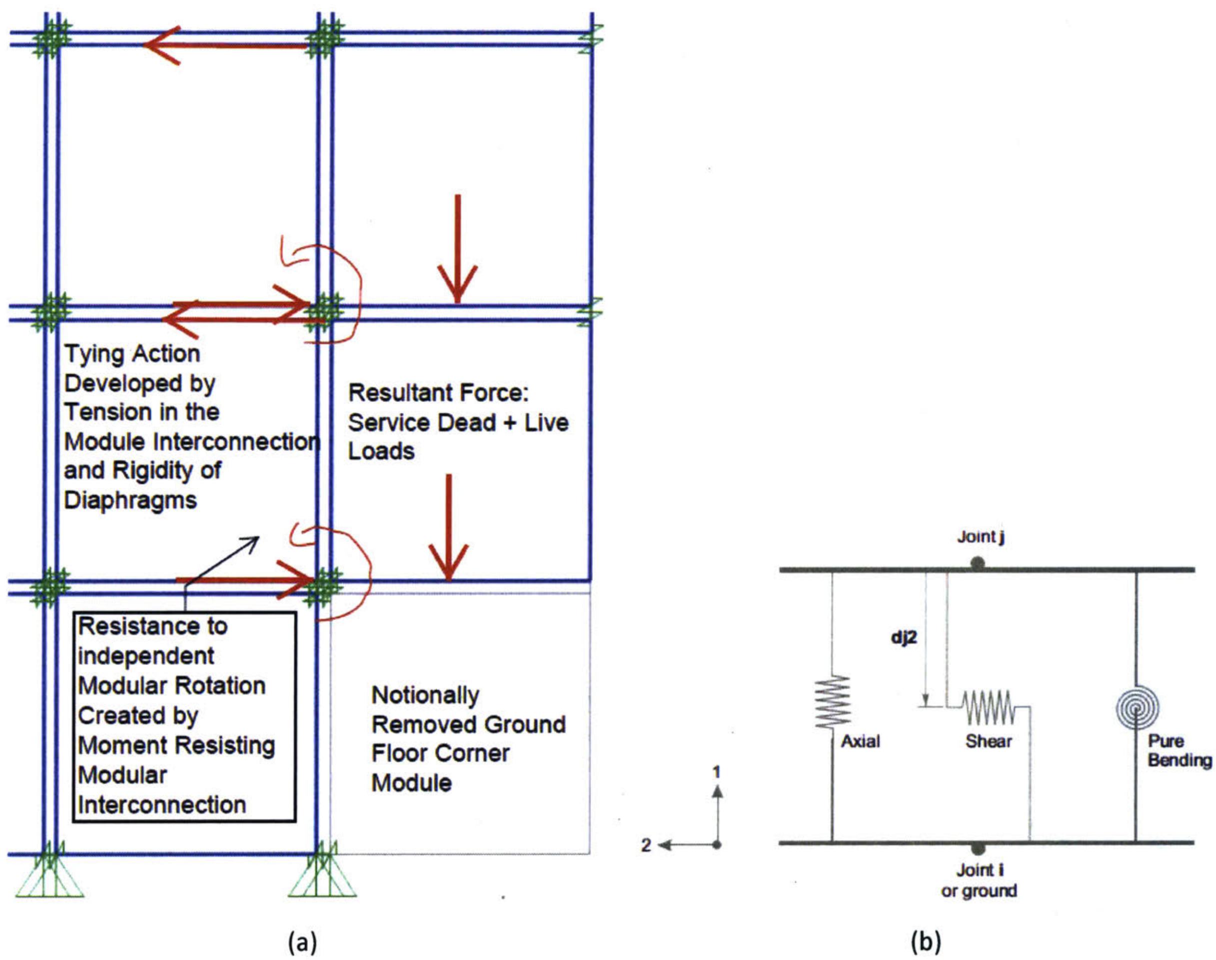


FIGURE 4.11 – (A) FORCES DEVELOPED BY NOTIONAL MODULE REMOVAL (B) THREE INDEPENDENT SPRING HINGES IN A LINK/SUPPORT ELEMENT (CSI REFERENCE MANUAL)

Figure 4.12 compares the drift of the prototype buildings modeled with rigid connections against the desired maximum total building wind drift, H/400. One of the goals of this thesis was to determine if a high-rise modular building could be laterally stabilized using only rigid modular interconnections. However, as shown by Figure 4.12, the prototypes with modular rigid interconnections did not meet the building serviceability criteria. As shown by figure 3.5a, the building prototypes were built without modelling any walls, elevators shafts or staircases which may have contributed significantly to the stiffness of the building if included in the model.

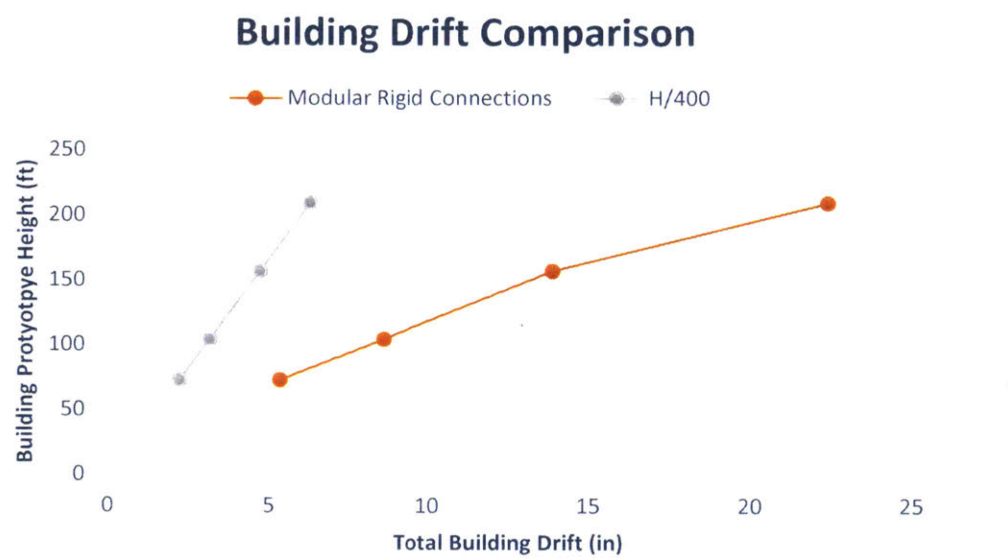


FIGURE 4.12 – COMPARISON OF TOTAL BUILDING DRIFT

4.3.3 Discussion

Unlike simple modular interconnections, such as the one shown in Figure 4.1, which are assumed to not transfer lateral loads through module columns, rigid connections such as the VectorBloc may be modeled as part of the lateral force resisting system. Because the rigid modular connections did not reduce the building drift to within the serviceability criteria, a concrete core will be added to the building prototypes to create a dual lateral force resisting system. If the building prototypes with rigid connections are analyzed as cantilevered beams, their bending stiffnesses can be approximated as:

$$k_m = \frac{V_x}{\Delta_m}$$

Where:

K_{mx} is the stiffness of prototypes with rigid modular interconnections, where V_x is considered

V_x is the resultant wind force or base shear in the x-direction

Δ_m is the total building drift for the prototypes with moment resisting interconnections

Stiffness and drift values are calculated in the same manner in the y-direction. The total required stiffness of the modular building system, k_T, so that it satisfies the serviceability criteria of H/400 is:

$$k_T = \frac{V}{\frac{H}{400}}$$

To increase the stiffness of the building, the same reinforce concrete core which was incorporated in section 4.2.3 is added to the building prototypes with rigid connections. The prismatic 12' x 5' core is depicted in Figures 4.6 a and b. To simplify the analysis the core and building prototypes are assumed to behave as a composite cantilever beam. The required core stiffness, k_c, can be approximated as:

$$k_c = k_T - k_m$$

As detailed in section 4.1, the resultant wind force can be approximated to act at 2/3rd of the building height and so the required concrete core stiffness is estimated as:

$$k_c = \frac{6EI}{\left(\frac{2H}{3}\right)^2 * \left(\frac{7}{3}H\right)}$$

As in section 4.2.3 the compressive strength of the concrete core is taken as 5 ksi and the modulus of elasticity is 4,286 ksi. The required thicknesses, rounded to the nearest inch, of the concrete cores for the buildings prototypes are listed in Table 4.5. Figure 4.13 compares the required concrete cores for the building prototypes with pinned and rigid modular interconnections. On average the building prototypes with rigid connections required a concrete core which was 33% less thick compared to the cores required for the building prototypes which used only rigid connections. Therefore, while rigid modular interconnections may not always be capable of providing the necessary stiffness to a modular building so that it meets serviceability criteria, it can successfully be incorporated as part of a dual system.

TABLE 4.5 – STIFFNESS REQUIREMENTS OF MODULAR BUILDING SYSTEM WITH RIGID CONNECTIONS AND CONCRETE CORE

Number of Stories	Prototype Stiffness in x, k_mx (k/in)	Prototype Stiffness in y, k_my (k/in)	Stiffness Required from Core in x, k_cx (k/in)	Stiffness Required from Core in y, k_cy (k/in)	Minimum Required Core Thickness (in)
7	13.3	16.6	19.1	5.6	1
10	13.4	16.7	23.3	8.5	1
15	14.6	18.3	28.5	11.3	3
20	13.3	16.6	34.1	15.9	8

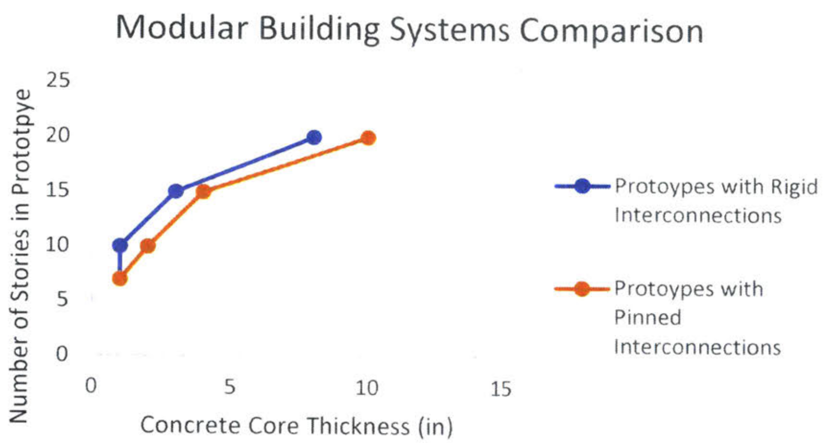


FIGURE 4.13 MODULAR BUILDING SYSTEMS COMPARISON

As depicted in Figure 2.2, rigid frames are most often used as lateral force resisting systems in conventionally built steel buildings up to 30 stories tall. Loads are resisted through rigid frame action where shear forces and bending moments are developed in the frame members and fixed or partially fixed joints. Moment frames are typically used as secondary systems in traditionally built high-rises because of their high costs and lack of rigidity. Figure 4.14 shows the moments developed in a rigid frame due to an applied lateral force. In braced frames lateral loads are transferred through axially loaded diagonal members. Figure 4.15 depicts two frames used in a cost comparison study between braced and rigid frames (Richard 2014). Both frames are 15 feet tall by 15 feet wide and are pin supported. The structural sizes were designed for a 10-kip lateral wind load per AISC LRFD and for serviceability to withstand L/400 lateral drift. Based on estimates from W&W Steel LLC, a steel fabricator located in Oklahoma City, the installed price of the moment frame including (materials, shipping, fabrication, and erection) would be 250% greater than the installed price for the braced frame. W&W Steel claims that the industry range for the installed cost increase between braced and moment frames is between 200% and 400%. Structural engineers therefore tend to pick reinforced concrete shear walls and braced-frames over of moment frames when designing a conventionally built lateral force resisting system.

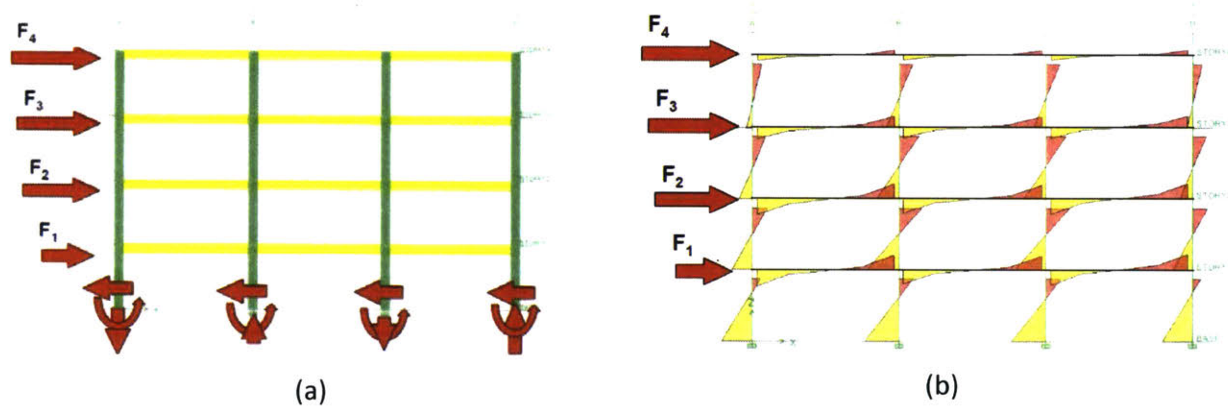


FIGURE 4.14- MOMENT FRAME REACTIONS. (A) RESULTANT FORCES (B) FRAME MOMENT DIAGRAMS (PETROV 2017)

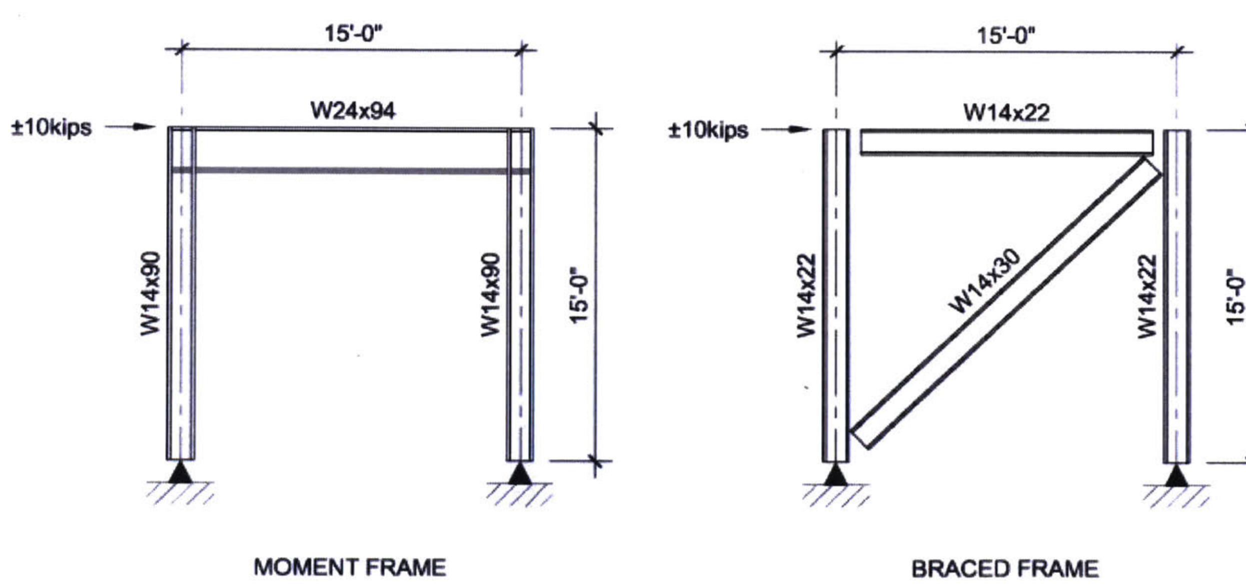


FIGURE 4.15 - COST STUDY BETWEEN MOMENT FRAME AND BRACED FRAME (RICHARD, 2014)

Moment frames allow for more open floor plans to be developed. Braced frames and shear walls may limit a buildings architectural organization; reinforced concrete cores are often placed near the center of a building to limit torsional stresses and displacements while braced frames are often placed on the perimeter of a building where they have the largest moment of inertia but block window views. For architects or owners who desire to have an open space floor plan with minimal obstructions; rigid beam to column connections in moment frames provide for the most unrestrained designs.

Although rigid frames may be more expensive than braced frames or shear walls in conventionally built buildings, it does not necessitate that the same be true for modular buildings. From his comparison study with W&W steel, Richard (2014) concluded that the factor which had the greatest impact on the price disparity between the installation costs of braced and moment frames is erection costs. One of the most expensive components of installing a rigid frame is extensive welding due to the high cost of labor. However, while the cost per hour of labor is almost identical between on-site and off-site construction, the productivity per man hour is not. Figure 4.16 depicts a comparison of cost break downs between site intensive construction and modular construction. The economic benefits of modular construction arise from higher productivity in manufacturing and less work required on site leading to less labor costs per unit of work (Lawson et al. 2014). The (United Kingdom's) National Audit Office, (NAO), found that modular systems saved 7 to 8% over traditional construction practices just due to the speed of construction. The cost of welding does not change whether it occurs in a factory or on a construction site, however productivity of the welder increases dramatically. Therefore, while moment frames may require more labor hours than braced frames to install on site, that is not necessarily the case in a factory with a controlled setting. Moment resisting interconnections such as the VectorBloc are welded to steel modules at a factory, the modules only then requires 4 bolts to be screwed in when they are assembled on site.

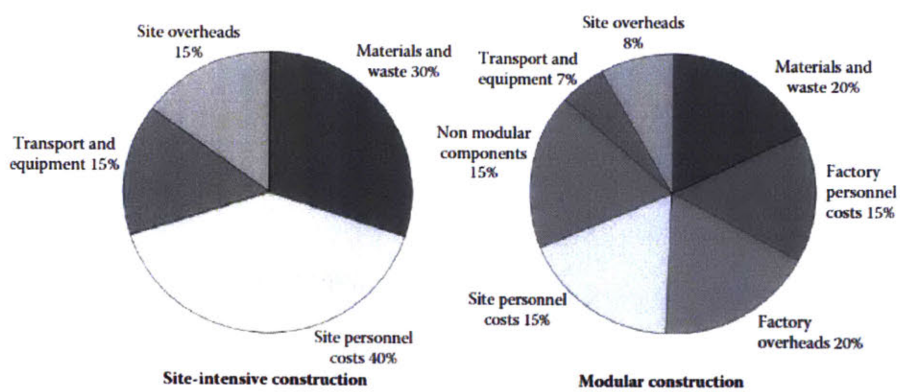


FIGURE 4.16 SITE-INTENSIVE AND MODULAR COST COMPARISON (NATIONAL AUDIT OFFICE, 2005)

4.4 Post Tensioned Intermodular Connection

In conventionally built buildings post tensioning has been used in a variety of applications. It may be used to help counteract external loads, reduce required structural member sizes and achieve longer spans. While post tensioning was first used in concrete floor systems, it has expanded other areas such as masonry walls, foundations, and moment frames.

The use of high strength post-tensioned steel strands may be used to create self-centering steel moment resisting frames (SC-MRF). As shown in Figure 4.17 a system that includes post-tensioned strands develops a moment resisting connection by compressing the beam flanges against the column flanges (Herning et al. 2009). Compared to typical welded seismic connections, SC-MRF systems require no field welding, are more ductile, and are able to re-center a frame which is shifted by an earthquake. The system is composed of: self-centering post tensioned strands, top and seat angles which dissipate seismic energy, shim plates that create a firmer contact between the angles and the column and a floor system which transmits earthquake inertial forces to the SC-MRF. Redundancy of the connection in shear is created through the bolted angle connections and the friction created by the post tensioned strands compressing the beam and column flanges.

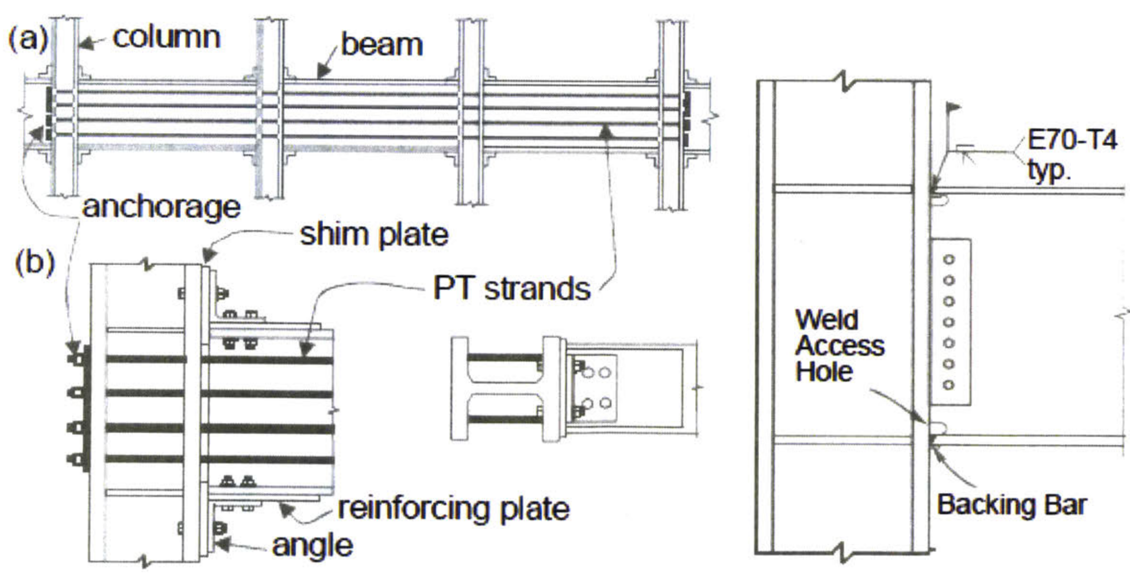
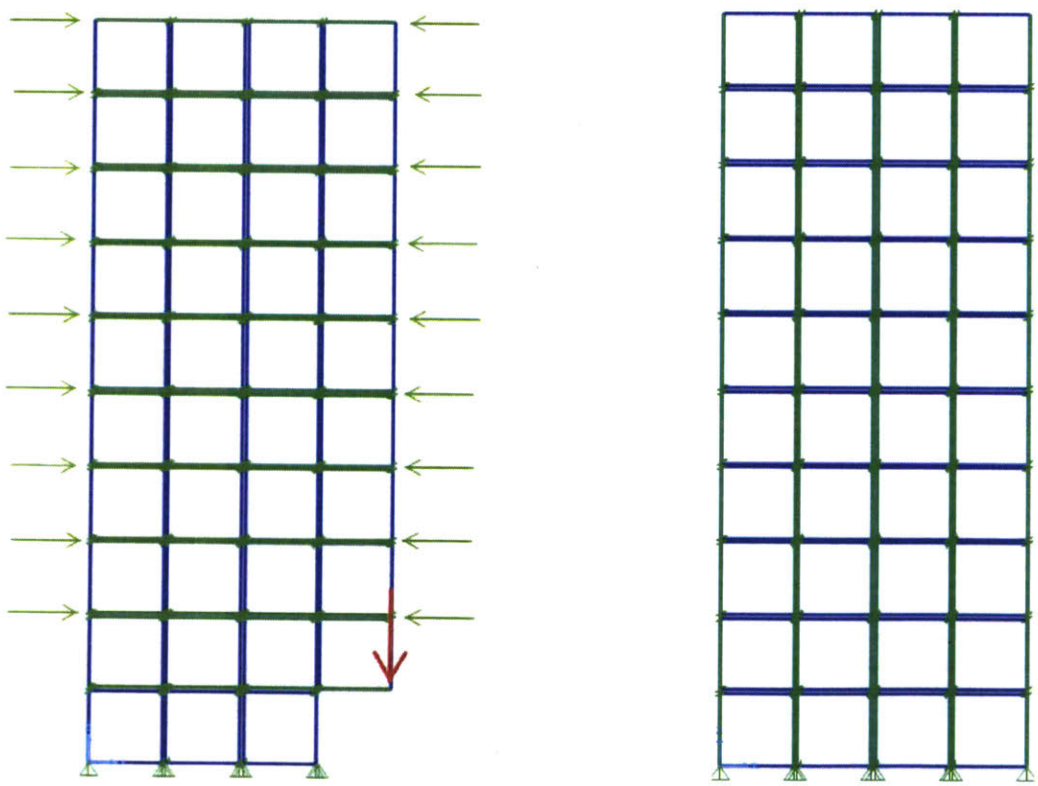


FIGURE 4.17 - (A) SC-MRF ONE LEVEL ELEVATION (B) CONNECTION AT EXTERIOR COLUMN (HERNING ET AL. 2009)

The majority of modular interconnections are constructed with mechanical fasteners such those shown in figures 4.1 and 4.4. However, the commonly used simple pin interconnection is not able to provide the required lateral force resistance and structural integrity requirements needed for taller buildings. Moment resisting interconnections improve the robustness of modular buildings but are currently only employed alongside other lateral force resisting system. A steel modular system which incorporates post tensioned steel strands as a lateral force resisting system is worth investigating under seismic and structural integrity loading scenarios. Horizontal post tensioning is included as shown in Figure 4.18 to reduce the displacement of the modules which cantilever over the notionally removed corner module. The post tensioned tendons prevent the modules from rotating as shown in Figure 4.4a. Although, horizontal post-tensioned systems are rare have not yet been incorporated for use modular buildings, vertical post tensioning systems, such as the one depicted in Figure 4.4b have been researched for use in seismic resisting system modular systems such as the one proposed by Zheng et al. (2012)



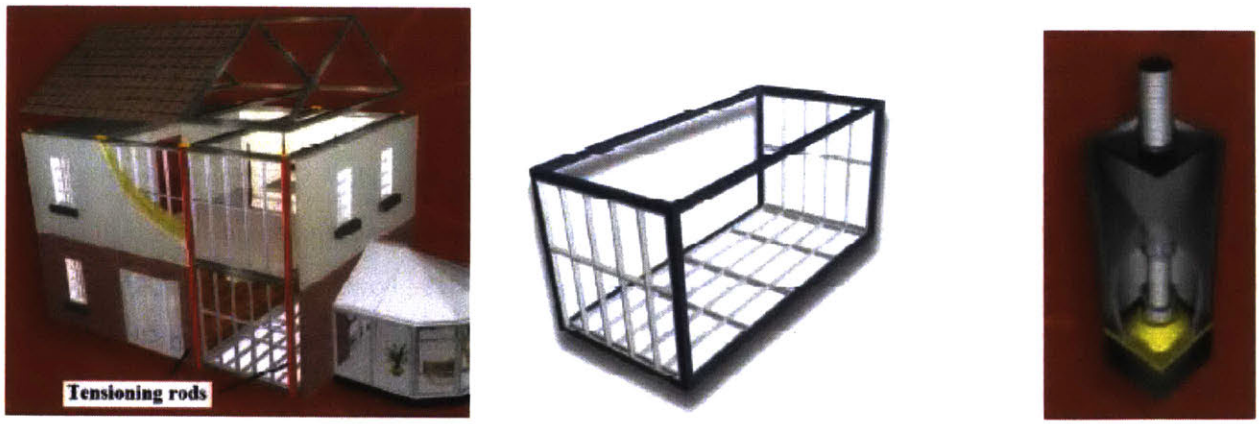
(a) (b)

FIGURE 4.18 - 10 STORY PROTOTYPES (A) HORIZONTAL POST TENSIONING (B) VERTICAL POST TENSIONING

Post tensioning has never been incorporated in any actual modular buildings. However, as the use of modular construction expands to taller buildings and regions with high seismic forces, post tensioning may provide a promising solution to resist the increased lateral loads. Zheng et al. (2012) investigate the seismic response of a modular building outfitted with a post tensioning system that is incorporated into a Powerwall building system depicted in Figure 4.19. Powerwall is a manufacturer of modular buildings in the United Kingdom, however they have not yet incorporated post tensioning into any of their designs. The post-tensioned modular system (PTMS) is formed by assembling the individual modules shown in figure 4.19b through the connectors, shown in Figure 4.19c, at the floor levels and then tensioning the rods. The pre-stressing achieved via the tensioning rods provides a mechanism for lateral load resistance. The individual modules are composed with hot rolled rectangular HSS members. As modules are installed on top of each other, tie rods are passed vertically though the tubular columns and attach to the connector with a lock nut at each story. Final tightening is applied thorough post tensioning at the top when the whole structure is erected. Figure 4.20 illustrates the force paths in the post-tensioned Powerwall modular system. The lateral movement of modules is resisted thorough shear provided by the connectors and friction between the modules.

Stresses induced by applied wind or seismic loads are redistributed between the modules and the tensioning rods. Three distinctive failure modes may occur in a post tensioned modular structure when it is subject to lateral loading (Zheng et al. 2012):

1. The tension rods may fail due to improper locking with the nut
2. Modular connections may fail under shear or tension
3. Modular frame member such as a column of beam may either buckle or fail due to excessive bending.



(A) (B) (C)

FIGURE 4.19 – (A) POST TENSIONED POWERWALL HOUSE (B) INDIVIDUAL MODULE (C) PT CONNECTION (ZHENG ET AL. 2012)

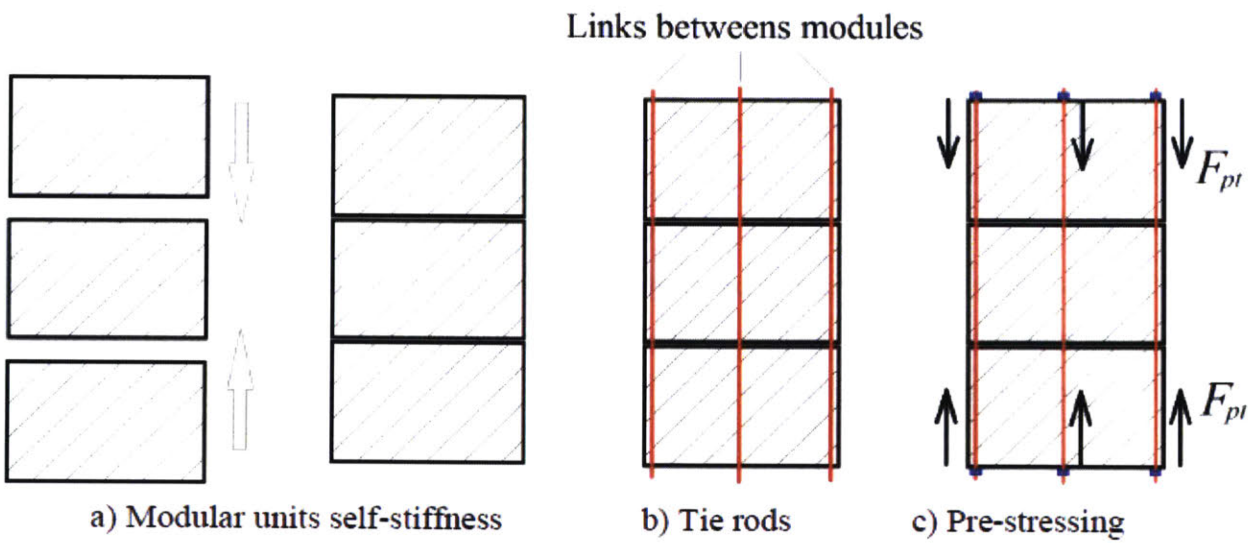


FIGURE 4.20 - SCHEMATIC OF VERTICAL PT SYSTEM (ZHENG ET AL. 2012)

4.4.2 Results

During seismic events, the post tensioned strands of the SC-MRF are elastically stretched and then pull back on the structural steel frames which reduces lateral displacements and re-centers a building. If this effect could successfully be implemented in a modular steel building then it could increase the structural integrity of a module structure by elastically pulling back on modules which are cantilevered over a damaged or removed module (Figure 4.21). The results of the modular integrity study listed in Table 4.6 show that horizontal post tensioning can reduce the displacement caused by notional module removal. The displacement due to the notional removal of a ground floor corner module which is only interconnected with other modules with a rigid connection is 5.81 inches in a 10-story modular building (Figure 4.12). If horizontal post-tensioning was incorporated in the 10-story modular building the module which was displaced 5.81 inches would now only be displaced 3.97 inches, a 32% difference. Horizontal post tensioning is found to be at least 15% more effective than vertical post tensioning or rigid interconnections at reducing the vertical displacement of modules caused by notional module removal.

Without any post tensioning or rigid interconnections, modules resist displacement through the tying action developed by tension in the modular interconnections and the rigidity of the diaphragms as depicted in Figure 4.11a. Modules with moment resisting interconnections also resist displacements through the rigid connections which prevent independent modular rotation. As shown in Table 4.4 the tying action is more prominent for smaller building heights (less than 10 stories), but the 20 story building prototypes with rigid connections had Δ_1 and Δ_2 value which were 65% less than that for the prototypes with pinned interconnections. As shown in Table 4.6, post tensioning has the greatest impact on the structural integrity of a modular building at each building height as the tying action developed by tension in the module interconnections acts in the same direction as the horizontal force created by the post-tensioned tendons.

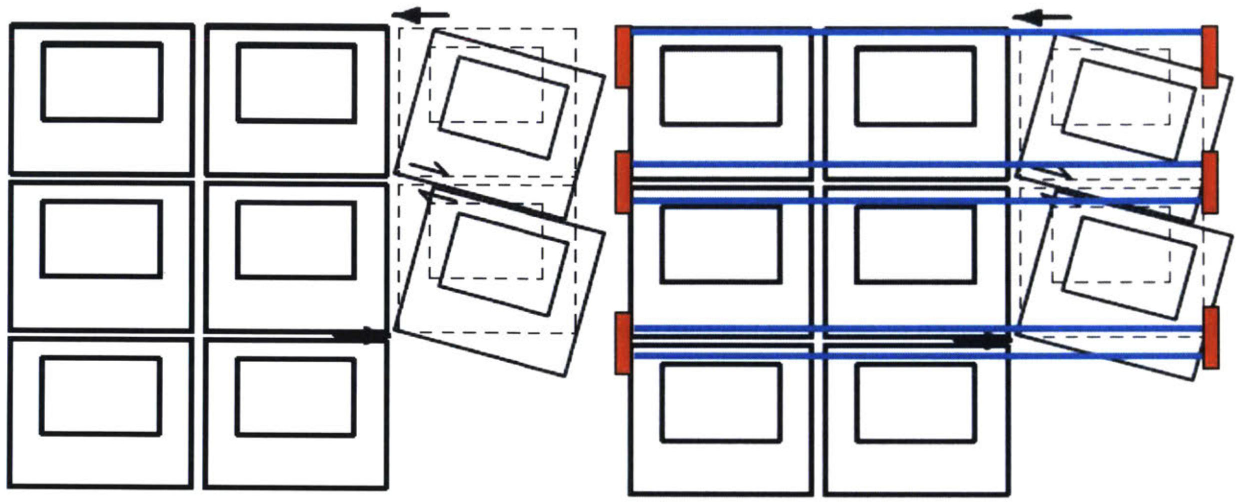


FIGURE 4.21 - MODULAR BUILDING WITH HORIZONTAL POST TENSIONING (ORIGINAL FIGURE FROM GORGOLEWSKI ET AL. 2001)

Vertical post-tensioning also has a positive impact on the structural integrity of a modular assembly. Table 4.7 shows that vertical post tensioning reduces the vertical displacement due to notional removal of intermediate modules in 15 story pin connected buildings by 67%. However, vertical post tensioning can potentially be difficult to install in high rise modular buildings. Zheng et al. (2012) discussed providing vertical post-tensioning in a two story modularly constructed house. For a 20-story modular building, construction tolerances can easily add up creating a misalignment between the modules. This would cause the resultant force of the vertical post tensioning to be off skew which could potentially create shear forces between the modules. The imperfect alignment of modules causes additional inter-module stresses and global instability. Out of verticality is caused by imperfect off-site manufacturing, deformation during transportation and installation inaccuracies (Lawson 2011).

Post tensioning would effectively reduce the amount of construction tolerances in a group of modules by squeezing them together similar to how the SC-MRF systems created a moment frame by compressing the beam and column flanges together. Post tensioned modular systems would potentially have better structural integrity than modules which are connected just at their corners, as the post tensioning will effectively connect a whole row of modules instead of just adjacent ones.

TABLE 4.6 – STRUCTURAL BEHAVIOR OF MODULAR PROTOTYPES WITH HORIZONTAL POST-TENSIONING

Number of Stories	Total Building Drift (in)	Module Displacement due to Notional Module Removal of Corner Module, Δ_1 (in)	Module Displacement due to Notional Module Removal of Corner Module, Δ_1 (% Change from Rigid Condition)	Module Displacement due to Notional Module Removal of Intermediate Module, Δ_2 (in)	Module Displacement due to Notional Module Removal of Intermediate Module, Δ_2 (% Change from Rigid Condition)
7	5.13	3.4	5.56%	2.9	12.12%
10	5.98	3.97	31.67%	3.2	39.85%
15	6.98	4.63	50.59%	3.95	53.96%
20	8.14	5.4	64.26%	4.6	66.79%

TABLE 4.7 – STRUCTURAL BEHAVIOR OF MODULAR PROTOTYPES WITH VERTICAL POST-TENSIONING

Number of Stories	Total Building Drift (in)	Module Displacement due to Notional Module Removal of Corner Module, Δ_1 (in)	Module Displacement due to Notional Module Removal of Corner Module, Δ_1 (% Change from Rigid Condition)	Module Displacement due to Notional Module Removal of Intermediate Module, Δ_2 (in)	Module Displacement due to Notional Module Removal of Intermediate Module, Δ_2 (% Change from Rigid Condition)
7	5.22	3.7	2.78%	3.2	3.03%
10	7.02	4.98	14.29%	4.31	18.98%
15	9.45	6.7	28.50%	5.79	32.52%
20	12.72	9.01	40.37%	7.5	45.85%

4.4.3 Discussion

The post-tensioning systems have a potential to reduce the total drift that the modular buildings experience. Figure 4.22 compares the drift of the prototype buildings modeled with horizontal and vertical post tensioning against the desired maximum total building drift, H/400. Although neither post-tensioning system satisfies the serviceability criteria, both exhibit less drift than the modular buildings with rigid interconnections did.

The horizontal post-tensioned systems are shown in Figure 4.22 to approach the H/400 line as the building height increases. The taller modular prototypes have a larger aspect ratio which causes them to displace larger distances just like a slender cantilever beam experiences more deflection than a stocky one. It is possible that if more pre-stressed tendons were added to the modular assembly or larger post-tensioning forces were applied to the threaded bars then the horizontal post tensioning would satisfy the serviceability criteria. To reduce the drift of the post-tensioned modular prototypes, effects of adding a concrete core were calculated just as in section 4.2. Similar to the building prototypes with rigid connections, the buildings with horizontal or vertical post tensioning required a minimal thickness of 2 inches for the concrete cores for the 15 story prototypes. The 20 story prototypes the horizontally and vertically post tensioned buildings respectively required a minimal of a 6 and 7 inch thick concrete core. For smaller buildings whose core thickness is governed by fireproofing requirements rather than stiffness, a braced steel core may be a possible substitute. Figure 4.23 a depicts a steel module outfitted with a staircase. The module columns are made of hot rolled HSS's and the beams are composed of cold formed channels. A potential bracing system, such as the one shown in Figure 4.32 b could be incorporated into the steel module. The V or X- braced staircase could then eliminate the need for a concrete core as long as the stair case was properly fireproofed.

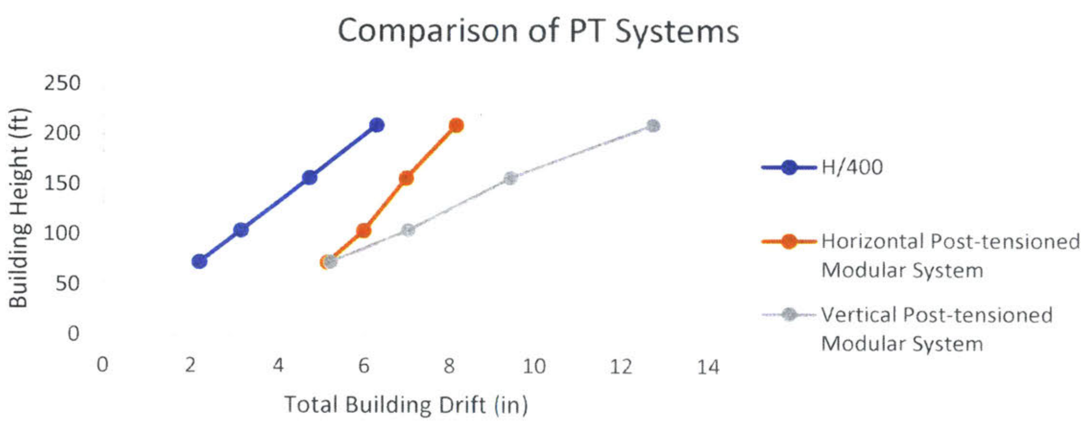
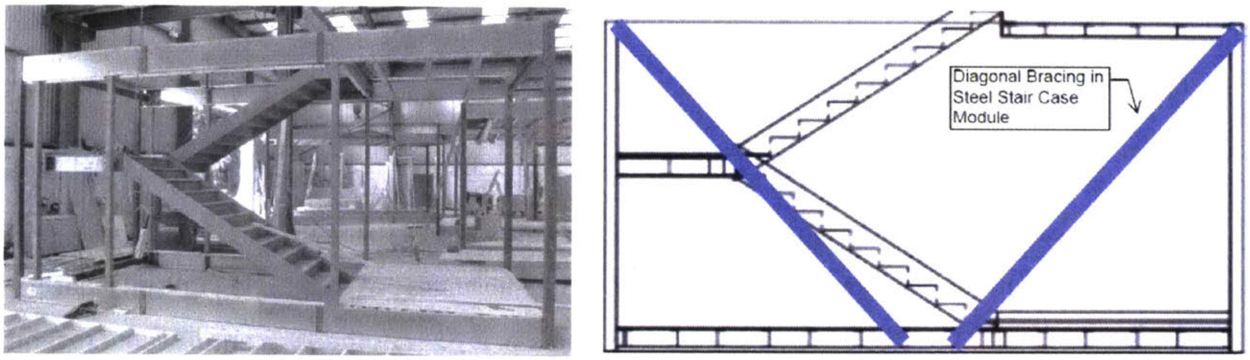


FIGURE 4.22 – COMPARISON OF POST-TENSIONED MODULAR SYSTEMS



(a) (b)

FIGURE 4.23 MODULE STAIRCASES (A) STAIR MODULE WITH CORNER POSTS (LAWSON ET AL. 2014) (B) PROPOSED STEEL MODULE STAIRCASE WITH DIAGONAL BRACING

Ch 5. Conclusion

By using off-site construction processes a project's cost and schedule can be reduced through standardization and pre-fabrication. Although pre-fabricated buildings like the London Crystal Palace proved the advantages of using panelized structural systems in as early as the 19^{th} century, until recently the majority of modular construction has accounted for less than 3% of the total US construction market. There has been a surge in the number of new buildings built using off-site construction processes over the past couple years due to the sustainability and cost saving benefits.

High-rise Buildings, such as The Atlantic Yards Tower, are being built with modular construction every year. However, as proven by the missteps of the 32 story tower, more research and detailed case studies are required to better understand the behavior of modular buildings. The performance of connections between modules largely affects the global behavior of a structure. Special care must be given to the design of interconnections as they transfer lateral loads between modules and affect the robustness of a modular building.

In this thesis, rigid modular interconnections were explored as a potential substitute for the conventional bolted plate connection used in most modular buildings. Moment resisting connections were shown to significantly decrease the maximum vertical displacement experienced by the modular assemblies when selected modules were notionally removed. The building prototypes which were modelled with rigid modular interconnections were shown to have better structural integrity compared to building prototypes modeled with only simple pin connections, as shown in Table 4.4. Including rigid connections reduced the total building drift of the modular assemblies but the prototypes still did not meet the desired serviceability criteria without providing additional lateral force resisting systems. However, the stiffness of the modular prototypes could potentially be greater if elevator shafts, exterior walls, partitions or staircases were included in the models. To increase the stiffness of rigidly connected modular assemblies, a concrete core can be incorporated. Similar to traditionally built buildings which incorporate dual building systems, a pre-fabricated building may use moment resisting connections like the Vector Block to resist part the lateral load, in additional to using braced frames or prefabricated steel cores. Engineers are often hesitant to use moment frames in traditionally built building s because they often cost 200-400% more than comparable braced frames. However, the inherent advantages in manufacturing moment resisting connections in a controlled environment with automated systems, like robotic welding arms, could potentially make it cost effective to incorporate rigid connections in pre-fabricated buildings. Therefore, while they are currently rarely used in practice, there is a potential to incorporate rigid modular connections into future modular buildings.

In section 4.4, post-tensioned tendons were proposed to be used as a lateral force resisting system in steel modular buildings. The post-tensioned modular systems were found to have a impact on the structural integrity and building drift of the studied modular prototypes. Modular assemblies with horizontal post tensioning systems were found to experience at least 15% less vertical displacement due to the notional removal of module elements compared to the other interconnections studied in this thesis. Rigid modular connections and post-tensioned systems allow for more open floor plans with less obstructions to be developed. A possible design which uses post-tensioned tendons and rigid connections in the direction of the building's strong axis along with inverted V bracing in the direction of the building's weak axis can be a focus of a future study to develop a hybrid modular building. Such a design would give architects and owners the flexibility to create open space floor plans and expands potential future applications of prefabricated construction.

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