VERTICAL EXPANSION OF EXISTING BUILDINGS USING
MULTI-STORY MODULAR CONSTRUCTION METHODS

A Thesis in
Civil Engineering

by
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ABSTRACT

The National Institute of Science and Technology (NIST) has recognized that the U.S. construction industry has room for large productivity gains. NIST has also recognized that the increased use of prefabrication methods in construction projects may be a way of improving productivity. This research is provided to investigate the potential productivity gains available with one type of prefabrication method, which is the construction of residential multi-story buildings using modular construction. Contained in the research is a detailed literature review, which identifies building uses such as social housing, micro-housing, military housing and other facilities where cellular arrangements are compatible with multi-story modular construction methods. Also identified in the research is opportunity for expanded use of non-structural modules in multi-story structures of the future, new structural module types and design philosophies, as well as, opportunities for modular manufacturers such as vertical expansion and custom one-off type projects.

The research identifies that multi-story modular construction techniques can be a suitable construction method for vertically expanding existing buildings and that modular construction can offer some advantages over competing site-intensive construction methods in some projects. The research uses the example of vertical expansion to discuss different types of structural modules suitable for this purpose, discuss appropriate means of conducting structural analysis, reveal design challenges associated with the use of modular construction and present design suggestions and resources that can be helpful. Analytical modeling is used to explore the feasibility of modular expansion and to reveal design aspects relevant to the task and important to structural engineers, developers and designers considering vertical expansion using modular construction. The analytical model of an existing building was created in software and structural analysis was performed to determine the effects of four different expansions on an existing building.

The research discusses the use of finite element modeling to perform structural analysis of modular vertical expansions. The finite element approach is useful to describe load transfer from the expansion to the existing building and to evaluate how the existing building will perform when subjected to expansion. Finite element modeling can also provide information such as story drift and diaphragm deflection, which is helpful in evaluating preliminary expansion performance and allowable heights. With some refinement, to current modeling presented, the approach can
potentially be used to preliminarily evaluate performance of modular expansion under seismic loading and could also be useful in determining loads for preliminary foundation design of newly constructed modular buildings.

The research uses the vertical expansion study as a tool that brings to light important design considerations that are not only relevant to vertical expansions but also to newly constructed modular buildings and prefabrication in general. The six-sided nature of modules was found to present challenges in structural modeling, questionable material efficiency and complications arising from building code requirements for non-combustible construction. An additional challenge that was identified for modular vertical expansion was finding a modular manufacturer who could economically deliver expansions of relatively low floor area.

In the research, corner-post bearing modules along with wood-framed wall bearing modules were found to be popular module types in the U.S. Both corner-post bearing modules and wall-bearing modules are defined and used to model one and two-story expansions. Wall-bearing modules are shown to have lighter weights, which is advantageous in vertical expansions, than the comparable corner-post bearing module, but may not be appropriate to use when non-combustible construction is required.

The research shows that modular projects require significant pre-planning to be successfully implemented and modular methods are not appropriate for all types of projects. The research also shows that additional complication arise when considering modular construction methods for vertical expansion. The primary considerations are showing that the building is capable of handling an expansion, determining the number of allowable stories to add and conceiving an appropriate load transfer strategy to ensure sound integration of new and existing construction. There are a significant amount of considerations in addition to structural considerations, such as building code issues, zoning restrictions, architectural considerations and mechanical equipment relocation that could affect feasibility of modular expansion. The research shows that ingenuity of design and combining expansion with other needed retrofits such as energy or cladding improvement can improve feasibility of modular building expansion.
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Chapter 1

Research Program Description

1.1 Research Background and Rationale

This thesis research was initiated in the fall of 2012 for the purposes of conducting a state-of-the-art review of multi-story modular construction methods, as they pertain to residential construction and to identify new application for this system of construction in the U.S. multi-family construction market.

From literature review of the topic, it appeared that multi-story modular construction methods were used more frequently in the U.K., in comparison to the U.S. Several journal articles, co-written by Mark Lawson from the university of Surrey, detailing the use of multi-story modular construction in the U.K. were initially reviewed (Lawson et al. 2012, Lawson and Richards 2010, Lawson et al. 2005), and it was not clear after reviewing these articles why multi-story modular construction methods are not more widely researched and used in the U.S given the documented benefits and productivity enhancements available through the use of these methods.

Following are two reasons why the timing and scope for this project was considered appropriate:

- The current construction industry interest in green building initiatives and Building Information Modeling (BIM) (McGraw-Hill Construction 2010)
- The increased demand for multi-family housing in the U.S. U.S. Census Bureau reports a 20.4% increase in construction spending on multi-family facilities for 2012 and forecasts a 35.2% increase in 2013 and a 40.2% increase in 2014. This increase, follows three straight years of decreased spending from 2009-2011 (“Construction Spending Ends the First Quarter on a Sour Note” 2013).
1.2 Research Goals

Prefabrication and modular construction methods have been identified by the National Institute of Standards and Technology (NIST) (MBI 2010) as a potential way to improve productivity in the U.S. construction industry. Multi-story modular construction methods, which are a subset of modular construction methods, are currently being utilized by developers to quickly construct cost-effective, multi-family housing. The goal of this research is to provide the construction industry with new knowledge that can be used to improve confidence in using one form of prefabrication as well as guidance to help define appropriate applications for this form of construction that could ultimately lead to increases in construction project productivity.

There is an increasing interest in using prefabrication in construction projects from those working in the industry, but there are also factors that could restrict the selection of a prefabrication method in a given project. A few examples of some of the influential factors are contractor resistance to new construction methods, social stigmas, aged manufacturing facilities, lack of training regarding the use of prefabrication and outdated construction management approaches (McGraw-Hill Construction 2011). Cost-effective implementation of prefabrication methods can be difficult for projects influenced by any of the above factors.

Academic research in the field of prefabrication is important because it produces information that is valuable to those members of the construction industry who wish to achieve productivity increases in their projects through the use of these methods. Research, such as this, can help increase confidence levels in using prefabrication methods, investigate barriers to implementation of the methods, define appropriate uses and project management considerations, as well as identify ways to refine current methods of using prefabrication techniques, such that they can be used more efficiently, economically and prolifically within the U.S. construction industry.

Prefabrication and modular construction techniques can be misused or improperly applied to a project such as with the Department of Housing and Urban Developments’ (HUD) Operation Breakthrough project (Staats 1976). This was an attempt to promote prefabricated housing that was largely unsuccessful due to a lack of preliminary planning and market research, which are typically keystones to the successful implementation of prefabrication on a project. Prefabrication methods can be mistakenly applied to projects that are not compatible with these methods and that do not take full advantage of the strengths of this form of construction. They can also be inadvertently used in markets that are not ready to accept this form of construction as in the case
of Operation Breakthrough. In these instances, the prefabrication methods may be associated with the project failure or inefficiencies in the project, which may discourage the use of these productive methods in other more suitable projects.

From literature review, it was apparent that there are productivity benefits available to users of prefabrication methods, but there also appeared to be risks and complexities associated with successful implementation of these methods. NIST identified (MBI 2010) that “Greater use of prefabrication, preassembly, modularization, and off-site fabrication techniques and processes” can potentially lead to productivity increases in the construction industry; however, from literature review, it was apparent that prefabrication is not suitable for all situations and proper selection of application is important to whether using a prefabricated method in a construction project will lead to any increases in productivity. It was also apparent that as the level of prefabrication increases, in a project, the level of pre-planning effort required by the design team also increases.

In this research, it is believed that projects involving a high level of prefabrication can fail or only realize partial success, not because of problems with the prefabrication method, but mostly due to misapplication of the considered method or failure, by the design team, to recognize how the decision to include a prefabricated component in the project impacts the design process. It is believed, as NIST suggests, that there are productivity increases possible through the use of prefabrication in a construction project but only if the prefabricated method is used for the proper application and the design team has a good understanding of the steps in the design process that are impacted by the introduction of a prefabricated component.

“Modularization” is one type of prefabrication technique mentioned in the NIST statement and multi-story modular construction is a form of modularization. This research is designed to take this one form of prefabrication and develop a strong understanding of it, such that proper applications can be identified and explore the important pre-planning aspects of the design process that can directly impact feasibility of implementation. Aspects that impact feasibility are assumed in this research to also impact productivity. Identifying factors that impact productivity and applications that are appropriate for this construction method can help to increase confidence in using this method and increase the chance of successful implementation.

This research has five main objectives, with the first being to provide the construction industry with a state-of-the-art review of multi-story modular construction, focusing on the background information that would be helpful to the project design team for the purposes of evaluating the feasibility of using this prefabrication method on their project. Secondarily, the
research seeks to define those applications that are appropriate for the construction method. The
third objective is to identify potentially appropriate applications for multi-story modular
construction that are either not currently used to a great extent in the U.S. or are new. The fourth
objective is to demonstrate the use of one of these in a conceptual engineering design scenario
and outline the design process. The last objective is to summarize the factors during the design
process that are thought to influence project feasibility and to distinguish those that are directly
related to multi-story modular construction.

Three concepts were identified during the course of research that were evaluated for use
as a design example. The first was an original concept called MODs. MOD’s was conceived
during the course of the research, but its originality has not been confirmed. MODs was
envisioned as a mobile residential module that could be placed and removed in an independent
structural frame. The module could be relocated as the purchaser relocates throughout their
lifetime and can be used in a communal structural frame or stand-alone setting. The second idea
was a multi-story modular residential core that could efficiently contain the majority of the utility
services in a home. This concept has been used in the past but is not common in current
residential construction. More information regarding MODs and the core concept is located in
Appendix A. The third concept identified was that of vertically expanding existing buildings
with residential modules. This concept has been used in the past in European countries but has
seen limited U.S. application (The Main SuRE-FIT Results 2009). After review, the vertical
expansion concept was selected for further research and to demonstrate application of multi-story
modular construction.

It is important to note that the literature review did not turn up a significant amount of
structural engineering design guidance relevant to multi-story modular construction projects in
the U.S. There is a limited amount of helpful design information available to designers who wish
to implement multi-story modular methods in the U.S. During the course of the research several
visits were made to modular manufacturing facilities in Pennsylvania (O’Hara 2013a),(Erb
2014),(Engle 2014),(Kline 2013). The visits were enlightening and tied together concepts read
about in literature review. Basic design information was shared, but the manufacturers were
unable to share detailed engineering design information due to proprietary interests. Most of the
public information available was from research conducted in the U.K. These sources are
enlightening, but not 100% applicable in the U.S. construction market.

The European Union (EU) has embarked on large scale research such as Robust
(“ROBUST - Renovation of Buildings Using Steel Technologies” 2013) and SuRE-FIT (The
Main SuRE-FIT Results 2009), which help to evaluate and promote applications for modular construction methods. The Steel Construction Institute also has several publications (R. M. Lawson et al. 1999, Lawson 2007, Gorgolewski et al. 2001) that provide information regarding engineering design to consider when evaluating the potential use of modular construction methods for a construction project. No comparable U.S. based guidance or initiatives have been identified in the literature review. This research is based chiefly on the European design guidance.

One of the unique aspects of this research is that it looks at the design of a multi-story modular buildings from an academic perspective and applies academic guidance taken from European sources and applies it to a U.S. design problem. As mentioned previously, site visits were made to four U.S. modular manufacturers to observe the construction of some typical U.S. modules. All manufacturers were courteous and positive about showing their product and production operation; however, it was clear that they were restricted from sharing any specific engineering design guidance by their proprietary interests. It was gathered from these four tours that engineering design for their modular projects was performed by a combination of factory employees and their contracted third party engineering firms, which use guidance developed and compiled by the factories.

The firsthand information gained from these visits was beneficial to the research and in the future it would be highly beneficial for academia to partner with multi-story modular construction industry representatives to share resources and lessons learned, as well as to help identify promising areas of research. Increasing academia’s presence in the engineering aspects of multi-story modular buildings, as well as other forms of prefabrication provides greater opportunities for significant advancement in these methods and the production of helpful literature that can ultimately benefit the construction industry.

1.3 Document Organization and Project Objectives Summarized

The document is organized according to the objectives of the research. Initially, the information obtained through literature review of multi-story modular construction methods is presented. Upon completion of those chapters the topic of vertical expansion is discussed. Examples of both traditional vertical expansions and of modular vertical expansions are presented for the reader. Following the expanded discussion on the vertical expansion topic, the analytical
modeling experiment is described and the results of this experiment are presented and discussed. Following is a summary of the objectives of this research:

2. Define residential applications that multi-story modular construction methods are currently being successfully used for in the construction industry.
3. Identify several promising residential uses for multi-story modular construction methods that are either not prolifically used in the U.S. or are a new idea.
4. Select one of the applications identified in objective 3 and apply to a design situation. Outline the design process and determine the factors/decisions that can impact feasibility.
5. Summarize the factors identified in objective 4 that are thought to influence project feasibility, distinguish those that are directly related to multi-story modular construction, discuss/show how the factors/decisions that had an impact on this project can be generalized to other similar projects and lastly summarize the lessons learned from the research.

1.4 Scope of Research

It is important to note that the scope of this research project is limited to multi-story modular construction of residential buildings. The use of single-family modular structures and townhomes is well documented in the U.S. and is not the focus of this report, nor is the use of 2D panelized construction. Construction of other types of multi-story modular structures may be discussed to support the discussion of the residential uses, but is not the focus of the research.

This research is primarily an investigation of the structural engineering design aspects of multi-story modular construction. Some attention is given to the architectural and architectural engineering design aspects, but the focus is primarily on the structural/construction system. The construction management aspects, MEP engineering design aspect and project economics can all affect the feasibility of a multi-story modular project, but are not the focus of this research; therefore topics relating to these subjects are discussed in a broad sense only.
Chapter 2

Literature Review

2.1 Introduction to multi-story modular construction

It has been previously noted that modular construction methods are sometimes used by developers to construct cost-effective multi-story residential structures and to increase the productivity of the construction process. Modular construction methods are thought to accelerate the delivery of a building, reduce construction costs, waste, design errors and community disturbance as well as enhance the predictability of job costs and deliveries (Lawson et al. 2012).

Residential structures in particular, due to their repetitive cellular arrangements are ideal candidates for modularization. Multi-family facilities such as apartments, condominiums, student housing, work-force housing, public housing, and care facilities are all good examples of structures that can be reasonably constructed using modular methods (Modular Building Institute 2013). Multi-story residential structures can be constructed quickly on small parcels of land with minimal site and community disturbance.

Multi-story modular methods show potential for increased use in the U.S. (McGraw-Hill Construction 2011), but significant research is needed in order to maximize their effectiveness in practice. The buildings shown in Figure 2-1d and 2-1f are examples of the current market penetration for mid-rise multi-story modular construction projects in the U.S while Figure 2-1e is representative of the current state-of-the art for high-rise multi-story modular structures in the U.S. Figures 2-1a, b, and c show comparable structures built previously in the U.K. Developers in the U.K have been using modular construction methods since the 1970’s to construct multi-story buildings, whereas modular methods in the U.S, have been primarily used to construct single-family dwellings, such as mobile homes, and more recently multi-story structures (Lawson et al. 1999). Table 2-1 show an approximate five to ten year lag in the construction of multi-story structures between the U.S. and the U.K.

Modular technology has been identified by research and interview by the National Institute of Standards and Technology (NIST) (MBI 2010) and McGraw-Hill construction
(McGraw-Hill Construction 2011), as well as others, as a potentially good way to improve the productivity of the U.S. construction industry. Accordingly, the construction industry has a positive attitude toward the increasing use of this technology and there is a desire within to develop and use this technology more frequently (McGraw-Hill Construction 2011).

The use of multi-story modular methods in the U.S. construction industry has slowly increased in the last decade. The resurgence in the use of these methods is largely in response to the demand for high performance buildings and construction cost savings (McGraw-Hill Construction 2011). McGraw-Hill construction suggests that modular construction methods can be better implemented today largely because of the current interest in environmental sustainability, advances in Building Information Modeling (BIM) and state-of-the-art manufacturing technology.
Table 2-1. U.S. – U.K. Multi-story modular building completion date comparison

<table>
<thead>
<tr>
<th>Project</th>
<th>Project Completion Dates</th>
</tr>
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<tbody>
<tr>
<td></td>
<td>Mid-Rise Structures</td>
</tr>
<tr>
<td></td>
<td>U.K</td>
</tr>
<tr>
<td>b</td>
<td>2000</td>
</tr>
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</table>

2.2 Survey Points to New Interest in Modular Construction

The use of modular construction has been gaining momentum and popularity in the construction industry slowly over the last decade. Large statistical reporting agencies such as FMI Corporation and McGraw-Hill Construction are promoting the modular construction industry as a growth opportunity in the short term (MBI 2011a). McGraw-Hill suggests that modular construction methods can be better implemented today largely because of the sustainability goals within the construction industry, advancements in Building Information Modeling (BIM) and improvements in manufacturing methods (McGraw-Hill Construction 2011).

McGraw Hill Construction conducted a study in 2010 (McGraw-Hill Construction 2010) in which 494 national construction industry representatives (architects, engineers, contractors, etc.) were surveyed to assess their current level of modular construction use and forecast the future role of BIM in achieving sustainable, high performance building construction goals. From the survey, it was clear that construction professionals expect model driven prefabrication (modularization of building components aided by computer BIM models) to make important contributions towards achieving green building goals in the future. At the time of the study, only 16% of the green BIM practitioners and 8% of Non-Green BIM practitioners were using model driven prefabrication in over half of their projects; However, when asked about expected future use, 42% of the green BIM practitioners and 29% of non-green BIM practitioners expected to be using it on more than half of their projects. Of the total surveyed building professionals, 68% reported money and time savings as their primary reasons for use of model driven prefabrication.

The significant interest expressed by building professionals in using model driven prefabrication to achieve sustainable building construction goals compelled McGraw-Hill to perform an additional study specifically pertaining to prefabrication and modularization (McGraw-Hill Construction 2011). This study was conducted in a similar manner to the previous one and supported the idea that the renewed interest in using prefabrication/modularization within building construction projects is largely the result of advancements in BIM, the popularity of green construction, and advances in manufacturing methods.

The second study reports several key findings that help illuminate the current state of the market regarding the use of prefabrication/modularization in the building construction industry. With almost universal interest, a remarkable 98% of the surveyed professionals expected to be using prefabrication/modularization on at least some of their projects by 2013. For those not currently using these methods, the primary reason cited was that they were not specified by the
architect. The primary reason architects reported not specifying the use of modular methods in their projects was either owner resistance to the idea or owner not requesting it

### 2.3 Modular Construction

In their 2011 annual report (MBI 2011a), the Modular Building Institutes (MBI) defines modular construction as follows: “Modular describes a construction method or process where individual modules stand alone or are assembled together to make up larger structures.” MBI divides modular construction into the following two categories:

“Permanent Modular Construction (PMC): An innovative, sustainable construction delivery method utilizing offsite, lean manufacturing techniques to prefabricate single or multi-story whole building solutions in deliverable module sections. PMC buildings are manufactured in a safe, controlled setting and can be constructed of wood, steel, or concrete. PMC modules can be integrated into site-built projects or stand alone as a turn-key solution and can be delivered with MEP, fixtures and interior finishes in less time, with less waste, and higher quality control compared to projects utilizing only traditional site construction. PMC structures are designed to be permanent structures.”

“Re-locatable Buildings: A partially or completely assembled building that complies with applicable codes, and state regulations, and is constructed in a building manufacturing facility using a modular construction process. Re-locatable modular buildings are designed to be reused or repurposed multiple times and transported to different building sites.”

PMC methods can be subdivided into 2D panelized construction and modular construction. Panelized systems are complete flat assemblies that can be craned into place and set. The interior finishes and exterior cladding are not typically installed on panelized systems. Wall, roof, and floor systems are good candidates for panelization. Panelized systems are typically constructed from wood, light gauge steel framing, or concrete. Structurally Insulated Panel (SIP) wall and roof systems, panelized wood framed or light gauge steel walls, and panelized thin reinforced concrete foundation and above grade walls are all popular 2D assemblies used in modern residential construction. Modular construction comprises
prefabricated room-sized volumetric units that are normally fully fitted out in manufacture and are installed on site as load-bearing ‘building blocks’ (Lawson and Richards 2010). In this report, the focus will be primarily on 3D modular construction.

2.4 Volumetric Modules

A typical volumetric module used for the construction of a multi-family building is approximately 11’-14’ wide and 20’-30’ long and has a floor area of 270 ft² – 375 ft². One module is appropriate for a small single-person accommodation, two modules for a slightly larger 2-person apartment and three or four modules can be used for a family-sized accommodation (Lawson et al. 2012). There are three basic types of modules used in mainstream construction today (Lawson and Ogden 2008):

1. Wall-Bearing Modules: Load-bearing modules that use corner and intermediate posts (Figure 4-1a) to transmit gravity loads, which would align vertically throughout the building height.

2. Corner-Post Bearing Modules: Load-bearing modules that use their exterior walls to transmit gravity loads. The modules are stacked on top of each other, and the longitudinal walls are aligned vertically to form a continuous load path throughout the building height (Figure 4-1b).

3. Non-load-bearing units also called Pods, which are typically used as infill units for and set on a structural floor framing system.

Limiting heights for buildings constructed with wall-bearing modules is typically between six and ten stories; whereas, greater building heights can be achieved through the use of corner-post bearing modules due to their improved fire-resistance and more robust structural system (Lawson 2007). The structural limit-state for wall-bearing structures is generally the compression resistance of the wall studs, and the compression resistance of the HSS column members is generally the limit-state for those structures constructed from corner-post bearing modules (Lawson, et al. 2012).
Different lateral force resisting systems can be used to ensure stability of multi-story modular buildings of varying heights. The lateral force resisting system for buildings under six stories is typically contained within the individual modules and utilize diaphragm/shear-wall systems or in-plane bracing. Six- to ten-story modular buildings can require additional strengthening measures, which sometimes involve separate bracing systems located around a central access core and horizontal bracing accompaniments to help distribute the lateral forces from the modules to the central core bracing system. For taller buildings a central structural core and a steel podium are sometimes used to provide stability for the more demanding structural requirements (Lawson 2007).

![Figure 2-2. (a) Load bearing wall 3D module (image by Lawson and Ogden, 2008), (b) Corner-Post Bearing Module (image by Lawson and Ogden, 2008),](image)

### 2.5 Modular Construction Process

3D Volumetric Modules are typically constructed in a factory, stored on-site and shipped directly to the jobsite when required. The factory environment improves quality control, facilitates tighter construction tolerances, encourages material efficiency and promotes a safe and consistent labor environment (Modular Building Institute 2013). The modules are typically constructed by assembling 2D wood or light gauge steel panel elements in an assembly line fashion (Lawson and Ogden 2008). When complete the modules are packaged with a weather resistant membrane and stored until needed. When the modules are scheduled for use they are sent to the job site for a “Just-in-Time Delivery”. The modules arrive on site and are assembled as quickly as possible to minimize exposure to the weather and construction environment. Assembly is usually very quick and accomplished by a minimal labor crew with a crane.
The main benefits modular construction methods have over site-built methods is the significant time savings gained in the construction process. MBI discusses four major stages of a typical modular construction project (MBI 2011a) in which time savings is achieved in both the 2nd and 4th stage. Figure 2.3 shows that, for a typical modular construction schedule, the primary time savings occurs in the 2nd stage of the project. Building designs are sent to the manufacturer as soon as available and the construction of the modules can occur soon thereafter. While the modules are being constructed, the final permitting tasks, site improvement and foundation construction can be occurring simultaneously. Secondary time savings are often recognized in the 4th stage as shown in Figure 2.3. Because much of the construction of a modular project occurs off-site, the need for equipment, labor and stockpiling space on-site is drastically reduced. Many times, the availability of site space makes it possible to begin site restoration activities concurrently with the erection of the modules. If there is adequate access around the structure to maneuver modules and position crane equipment, the remainder of the site is typically available for site remediation efforts.

![Figure 2-3. Sequence of events for a typical modular construction project (Partially reproduced from reference MBI 2011a).](image)

2.5.1 VDC and cloud computing services

As mentioned earlier, the main benefit gained from modular construction methods is time savings. In order to fully recognize the time savings in the modular process, a higher level of communication, detailing and scheduling should be maintained throughout the life of the construction project (MBI 2010). Because of the required forethought and planning required for modular construction implementation, it is apparent that integration of modular construction methods on larger scale
projects can become complex and confusing without enhanced coordination. Traditionally the architect coordinates activities, but with increasing level of minitua and specialist design team members (i.e., security, telecommunications, fire alarm systems, advanced HVAC control specialist, interior designers etc.) involved in a modern day high-performance building construction project, the task can be overwhelming for the Architect.

To relieve some of the burden some construction managers are beginning to redefine the services they can provide on a project. Companies like ShoP construction (“SHoP Construction” 2012) can provide many virtual design and construction (VDC) services that can be critical to the success of a complex project, such as:

- BIM execution and modeling
- Basic Modeling
- 3D clash detection and coordination
- 4D scheduling and simulation
- 5D quantity take-offs and estimating
- 6D facilities management integration and As-Built modeling
- Sustainability tracking
- Logistics planning and coordination
- HD laser scanning integration
- Database integration and virtual tracking

Another tool available to professionals in the construction industry is cloud computing. Cloud services are internet-based services that can be utilized to enhance communication on a construction project. Sophisticated VDC services can be administered by a provider through a network connection. The provider of the service can host computationally-demanding software programs on their vast, sophisticated networks, rather than requiring individuals design team member to have powerful computer equipment and software on hand. The provider can offer the appropriate service access to subscribers. For the design team members, this alleviates the burden of maintaining design software and networks; for the users, it provides enhanced computing power and almost immediate access to the agreed upon services. The team members can access their relevant components on an as-needed basis through web-based interfaces and can potentially use any portable network ready device to manipulate or view aspects of the design. This can be beneficial because it instant updates and feedback from members of the design team can be acknowledged or implemented.
2.6 Benefits of Modular Construction Methods

It is important to note that modularization is not necessarily appropriate and cost-effective for all projects. Like most industrialized products economy is typically achieved primarily in terms of scale (Lawson and Ogden 2008). Large cellular building types such as multifamily dwellings (i.e., condominiums, dormitories, hotels, and apartments) educational, correctional and health care facilities are all prime candidates (Lawson et al. 2012). McGraw-Hill reports that building professionals are using modular methods for the construction of healthcare facilities (49%), college buildings and dormitories (42%), and manufacturing buildings (42%) (McGraw-Hill Construction 2011) and that future opportunity exists in healthcare facilities, hotels and motels and commercial warehouses.

Modular construction methods, in general, are used to improve productivity in the construction process. In a recent report, the MBI (MBI 2010) points out that almost every U.S. industry has experienced growth over the last few decades except the construction industry. The National Institute of Standards and Technology (NIST) recently formed a committee of experts to investigate ways of improving productivity and competitiveness in the construction industry. The committee identified the increased use of BIM and modularization as keys to increases in productivity. The U.S. Department of Housing and Development (HUD) has one of its mandates to support manufactured housing, and at a May 2012 meeting of experts in Washington D.C. organized by the HUD’s Office of Policy Development and Research, development of multi-story modular construction was identified as an approach toward creating affordable and energy efficient housing.

From reviewing the McGraw-Hill research it appears that integration of BIM on a modular project is likely to improve the chances of enhanced project productivity. BIM helps to organize project information and team member access to the information. The availability of the information can provide opportunity for early design team input, which can lead to the early detection of costly mistakes and the identification of optimization opportunities for the project (McGraw-Hill Construction 2010). Model-based prefabrication outwardly appears to offer benefits over traditional design and project management schema when discussing a modular project. It is McGraw-Hill's opinion that the required early decisions and design accuracy necessary to ensure successful implementation of modular methods can benefit from the extra effort required to implement BIM on a project. Productivity increases can typically be recognized in cost and/or time savings for the developer. When looking at productivity in terms
of cost and time savings, Lawson and Ogden estimate 10%-20% cost savings may be available and 30%-40% time savings may be achievable in larger modular construction projects (Lawson and Ogden 2008).

Time savings is usually the primary benefit of using modular construction methods (Rogan et al. 2000). Modularizing a project can potentially reduce construction schedules by 50% or 60% and provide early occupancy of a structure (Rogan et al. 2000). This is beneficial primarily because early profits can be generated and loan interest payments can be reduced. Time savings may also be beneficial for projects that must be completed within an envelope of time, such as student housing. The time saving aspects of a modular project are a direct result of the process. Construction schedules are shortened by the ability to concurrently construct modules during the site preparation, and the opportunity to begin remediation efforts early in the schedule. Additional time savings are gained in a modular project when various subsystems of the building are all constructed at the same time (i.e., floor system, roof components, and wall systems) (Modular Building Institute 2013). Time savings is an easily identifiable benefit of modular construction. The cost-saving benefits, on the other hand, are not so easily recognizable.

Lawson and Ogden estimated 10-20% cost savings can become available on larger modular projects, but it is somewhat unclear where these cost savings occur. All the building projects shown in Table 2-2 and Figure 2.1 reported significant time savings and enhanced project coordination, but very few reported tangible cost savings. The Appalachian State University (Figure 2.1d) reported considerable cost savings on material deliveries and the “Modules” project (Figure 2.1e) reported cost savings from the use of inexpensive offsite labor from outside the Philadelphia area (“Market Commercial Housing” 2014).

**Table 2-2.** Time savings benefits of modular construction.

<table>
<thead>
<tr>
<th>Figure 2.1 designation</th>
<th>Time Savings Benefits</th>
</tr>
</thead>
<tbody>
<tr>
<td>a (MBI 2012a)</td>
<td>Completed one academic year ahead of time</td>
</tr>
<tr>
<td>c (“Oea” 2012)</td>
<td>Completed one year earlier than a site built version</td>
</tr>
<tr>
<td>d (“Awards of Distinction - Entry Detail” 2013)</td>
<td>Completed in 9 month narrow time window</td>
</tr>
<tr>
<td>f (“The Modules” 2013)</td>
<td>Completed in 9 months</td>
</tr>
</tbody>
</table>

The majority of the cost saving features in a modular construction project most likely come from the potential reductions in “Soft Costs” and are hard to document (Smith 2010). Soft costs are those that the developer never incurs. They are the costs that are avoided through proper planning and implementation of a design. Many of the case studies reviewed reported enhanced
coordination on their list of reasons why they chose modular construction. Inherent to the factory pre-fabrication process is an increase in the amount of coordination and planning that is required in advance. Costly errors and bad design elements can potentially be identified in the early stages of a project and eliminated. Material deliveries and costs are more predictable in a modular project and construction schedules are less likely to be delayed or changed due to mistakes or fall backs on construction (McGraw-Hill Construction 2011).

During a podcast hosted by the Modular Building Institute, a panel of representatives from both the site-built construction industry and the modular construction industry discussed improving the integration of modular construction methods with site-built construction (Hardiman 2015a). The panel discussed that in order to maximize potential savings a project should be conceived as a modular project, not converted to a modular project after it has already been designed with another building system in mind. Also discussed, was when modular manufacturers are afforded the opportunity for early project involvement they can suggest design elements and philosophies that are more likely to be cost effective for the project. Another point brought up during discussion was that the manufacturer, most likely, can be more cost competitive when brought in during the RFP (request for proposal) phase rather than after the construction bids have been awarded. After bid award project managers already have identified the project with the construction type listed in the design documents and most likely will be resistant to a change in construction method. In general, one of the main take away points of the discussion was that when a project has already been designed for a given type of construction it may not be economical to modularize later in pre-construction phase of a project.

Regardless of any cost-savings achieved by selecting modular construction, there is still the improved quality assurance and consistency advantages provided by the factory environment and the pre-fabrication process to consider. Factory-built products benefit from the controlled environment of an indoor facility, which can be managed more efficiently than an outdoor jobsite. Strenuous tasks can be eased by machinery, worker safety can be enhanced and monitored, and unsafe tasks can be eliminated. Additionally, the materials used in the construction of the modular components can be state-of-the-art and optimized by protocol and sophisticated equipment. The industrialization of the process provides stability, which enables accurate forecasting of the quantities of materials needed on hand, and makes it possible to reduced material inventories. Indoor environmental conditions are seasonally constant and unaffected by adverse weather conditions. The stable, comfortable indoor environment provides for enhanced craftsmanship, while the sophisticated equipment allows for tighter construction tolerances.
Kullman (Garrison and Tweedie 2008), the SCI (Rogan et al. 2000), MBI (Modular Building Institute 2013) and Smith (Smith 2010) identify some benefits available through factory production. A summary of some of the main benefits they identified follows:

- Potential for efficiency gains by mass production procedures and through the implementation of “Lean Manufacturing” philosophies.
- Labor prices are typically more predictable.
- The indoor factory environment is typically safer than an outdoor jobsite.
- Conditions in a factory can be monitored closely and workers typically have the proper tools and safety equipment for their given task. The indoor environment is comfortable for the workers and there is no threat of adverse weather. Off-site construction not only enhances safety for the workers in the factories, but indirectly increases the safety at the project location by decreasing construction activities on site. After site improvement and foundation construction, major construction activities are minimized and often limited to the erection of the modules by a small crew with a crane (not considering indoor fit out activities). This limits the amount of equipment and labor on site and thereby decreases the risk of accidental collisions or injury from misuse of tools. Lastly, due to the lessened amount of construction activities and the reduced time of construction, the danger to the public from construction related accidents is typically reduced.
- Product quality can be increased by the capability of modular factories to warehouse modules and provide “Just-in-Time” deliveries. Construction materials used in a modular factory are typically stored in a protected location with stable environmental conditions, whereas job site materials are typically stored outside wherever there is stockpile space available at the job site. The job site materials are typically delivered well in advance of their use and often sit outside exposed to damage from changing weather conditions and construction equipment. Undamaged material is used for the construction of the modules. The completed modules are then stored in a safe location. When needed, they are packaged in a weather resistant membrane and shipped to the jobsite for immediate installation.
- Quality assurance and craftsmanship are often improved in a factory setting. Factories have increased opportunities to implement quality assurance programs that
can be used to ensure proper production of the module. With assurance programs integrated into the production process and the availability of state-of-the-art production equipment, improved construction tolerances are possible. The indoor factory environment promotes increased craftsmanship because of worker comfort and the availability of proper equipment.

- A modular manufacturing facility’s access to state-of-the-art material and equipment can make it easier for the facility to integrate unique component assemblies into module construction and successfully construct complicated details. Complicated interoperability issues can be worked out ahead of time before the construction begins. For example, architects might feel more comfortable specifying a non-traditional floor system, wall system, flashing detail, or perhaps an MEP designer might be able to specify a more complicated control system for HVAC arrangement knowing that the installation will occur in the factory. In summary, the potential for increased product quality can be advantageous in some construction projects.

Numerous advantages through factory production are available to users of modular methods; however, a disadvantage associated with factories production is sluggish response to design changes. It may be difficult or expensive to change a module design once the production of the modules is started (O’Hara 2013a). The production of similar modules offers the greatest opportunity for material efficiency and cost savings. If a project has many varying types of modules the benefits of factory production may disappear altogether and site built methods may offer clear advantages. The underlying economics is often linked to the design of the structure and project size (Lawson et al. 2005).

Developers, like the originator of the Lillie Road, Fulham project seen in Figure 2.6b (SCI 2003a), will select modular construction methods for the purposes of reducing community disturbance. Community disturbance is reduced because project schedule is shortened, often to a matter of days, and the stockpiling space required for materials, labor and equipment is also greatly reduced (Modular Building Institute 2013) limiting the effects of construction activities on the community in the immediate vicinity of the project. This is advantageous to projects constructed in areas that would be negatively impacted by delays or disruption of normal activities such as urban areas and universities.

Modular construction methods can offer speed advantages over site built methods when considering the construction of student housing on university campuses (MBI 2011a).
Universities typically have small windows of time available for the construction of student housing. Most universities operate on a fall and spring semester schedule. Summer semester typically operates at a fraction of the activity level of the other two semesters. The ability to construct the majority of the structure offsite, finish ahead of schedule, and erect the majority of the building quickly during the summer months offers a noteworthy benefit.

The Kullman architecture manual (Garrison and Tweedie 2008) discusses the recyclable nature of the modular construction. Modular components can be deconstructed, reused whole if possible or recycled. Many of the modules seen on the market today are constructed of cold-formed steel, which has a high recyclable content. If the module does not have another use or if the service life of the module has ended, the module can be taken offsite, demolished, and recycled without the pressure of meeting a construction schedule or limited storage space and any of the interior components that have not exceeded their usefulness can be removed, reused, or resold.

2.7 Challenges Facing the Modular Industry

The most significant challenge facing the modular industry is the social acceptance of modern modular products. McGraw-Hill points out that the primary reason cited by industry representatives for not using modularization in their projects was that the architect did not specify it; according to the architects, this is based on client preference (McGraw-Hill Construction 2011). One of the daunting challenges the modular industry faces today is overcoming social stigmas from the past.

Due to some poor implementation efforts of prefabrication in past projects people may associate prefabricated, in general, with unattractive, low quality, industrialized structures (Craig et al. 2000). It is possible that the term “Modular Construction” is psychologically linked to inexpensive, inferior quality (relating to their site-built counterparts) buildings like mobile homes or examples of poorly constructed wood-framed single-family dwellings. Although these types of single-family structures are useful, the technology used in producing them may be outdated and not necessarily representative of the technology level needed to produce high-quality multi-story modular structures.

One question that remains unanswered however is, “Are there enough U.S. modular factories able to interface and advise customers and produce these high-quality multi-story
modular units which can increase productivity in a project? The answer may be, “No”. Manufacturing facilities are expensive to update therefore are unlikely to change unless there is a definite demand for the product. MBI reported in 2011 that total PMC market was two billion dollars in 2010; whereas, total non-residential construction money put in place was 552 billion (MBI 2011a).

This would indicate that modular construction has approximately a 0.3% presence in the non-residential construction market. It is difficult to say whether this level of market demand is adequate to provide incentive for manufacturers to invest in their facilities to try and capture a portion of market. The technology and philosophies are likely to be available to produce cost-effective multi-story structures, but it is not clear whether potential users would have reasonable access to producers of these structures. It can be difficult for an architect to recommend a construction method to a client if the option does not exist in the locale or the nearest available modular facility is too far away (Olsen 2013).

Architects will be instrumental in initiating a strong modular integration effort in the main stream construction market (McGraw-Hill Construction 2011). If they accept and adopt modular technology, architects have the opportunity to influence clients when developing potential design options for a project. The availability of facilities and the advancement of organizational technologies such as BIM should make it more realistic for architects to propose modular solutions.

Model-driven prefabrication solutions can theoretically improve the productiveness of using modular methods to construct multi-story multi-family structures, but there are still issues that need to be resolved before this becomes apparent. McGraw-Hill identifies in its Green BIM report (McGraw-Hill Construction 2010) a definite interest in and recognition of the advantage BIM can offer but also recognizes that BIM is in its early stages of development and acceptance within the construction community. BIM has yet to be adopted on a large scale and professionals in the field are still struggling with communication standards, protocol as well as liability concerns. This makes it challenging for design teams to commit to a labor intense BIM strategy.

Modular methods have physical constraints that could limit the ability to modularize some projects and should also be considered during the needs assessment phase as well. The feasibility of modularization should be evaluated at the beginning of the conception stage of planning. 3D modularization will not be dimensionally appropriate for some projects. Modular assemblies initially face transportation restrictions, which limits the dimensions of modules
MBI identifies the typical width of modules to be between 8’-16’, maximum lengths of up to 70’ and heights of between 11’-6” and 13’ (MBI 2011b).

In addition to dimensional constraints, the site access is also a limiting design factor. If there is nowhere to store the modules when delivered or it is unfeasible to get crane access around the structure, then it may not be possible to assemble the modular units. In this case, site-built methods may be more appropriate. Finally, if there are not any modular plants within a reasonable distance of the project, the transportation costs of the modules may be restrictive. There are many aspects of modular projects that should be considered in the design phase. Another, often restrictive element of any building construction project is the code compliance.

Modular construction projects face additional challenges within the building construction regulatory environment. Typical building codes and many local zoning codes are often convoluted and difficult to understand for many site-built construction projects, but can be restrictive for modular projects. Modular construction projects not only contend with building codes written for site-built construction methods but have the additional burden of abiding by state and federal transportation regulations.

The prescriptive nature of most building codes does not promote innovation within the design community. Prescriptive codes can hamper new ideas and make it difficult to integrate much needed new building design philosophies. Design codes, such as the current versions of the accepted national building codes, follow primarily a prescriptive-based approach in regulating designs. Performance-based code regulation allows for greater flexibility in design. Some performance-based specifications, especially in the energy codes, are written, but many are difficult to implement and tend to be outdated.

For example, the international code council (ICC) updates their building codes every three years and it often takes states up to a year to adopt them. By the time the code is being used by the designers, it can be up to four years old. In many instances, the referenced publications in the ICC codes already have new updated publications that the designers are not legally allowed to use in design because only the older version is the state approved. Prescriptive codes allow for little innovation and impede designers in many ways. In order to engage the creativity and professional training of the design engineers and lessen the effects of time lags, it would be advantageous for states to move toward performance-based criterion and allow the designers to use their professional judgment.

Lastly, it is important to briefly mention the redundant nature of modular construction. Due to the six-sided nature of the construction method inherent redundancy exists with adjacent
wall assemblies and floor/ceiling assemblies. The redundancy can bring the structural efficiency of this type of construction into question. Additionally, complications can arise when dealing with building height restrictions and ceiling height restrictions due to redundant floor ceiling assemblies.

2.8 Multi-Story Modular Construction

Both the benefits and challenges associated with multi-story modular construction are numerous. The maximization of the benefits however is often associated with the selection of the appropriate use for the technology. As discussed previously, factory produced housing makes sense when discussing many repeatable units. Multi-story, Multi-family buildings are good candidates for modularization because of the cellular, repeatable nature of the typical floor plans. In the next section, some common multi-story construction techniques will be discussed and some example presented.

2.8.1 Stackable modular units

The most common construction technique used to construct low-rise modular structures is stacking. Similar to building blocks, load bearing modular units (Modules) can be stacked and bonded to form a complete structure. Modules are typically arranged in a story floor plan such that they border a central corridor or common area. This allows easy service connection and common access for maintenance of modules or module connections (Lawson et al. 2012). When a building has multiple stories, the subsequent stories are arranged such that modules are aligned vertically and bonded to provide load path continuity. Load-bearing modules, as mentioned earlier, are typically stable on their own up to six stories.

Structures that exceed this height typically have additional superstructure elements to stabilize the building and transfer the increased design loading. The structure shown in Figure 2.4 is an example of a typical low-rise modular building. This particular project, owned by Panoramic Interests of Berkeley, Ca., was constructed in San Francisco to provide small space living accommodations (“LEED Urban Prefab SoMa” 2012). This type of housing offers small 300 ft² units commonly called micro-apartments.
2.8.2 Concrete core construction

Concrete cores are commonly used within structural systems to resist lateral loads in modular buildings exceeding six-stories. An example of a 19-story modular high-rise concrete core construction is shown in Figure 2-5 (MBI 2013b). Cores can be constructed onsite with reinforced concrete or can be prefabricated and assembled onsite.

Concrete cores are used to transfer lateral loads and provide story access in mid-rise and high-rise structures. Modules are typically arranged around a core in one of two ways. They can be clustered around the central core, with modules attached to the core via embedded connections, or they can be bordering a common corridor and attached via bracing elements. Typically, gravity loads are transferred through the modules. Module connections and bracing elements are designed to transfer lateral loads from the module to the core or corridor (Lawson et al. 2012).
2.8.3 Hybrid modular, panel and primary steel frame

A hybrid modular design incorporates the benefits of a primary steel frame with the benefits of 2D and 3D modular components. The primary steel frame is typically used as the stabilizing structure and provides the designer flexibility when planning internal spaces. 2D modular panels can be incorporated to make up open areas in the floor plan, and the 3D volumetric modules can be used for the core use spaces or highly-serviced spaces such as bathrooms. Two generic forms of construction are typically used with a hybrid modular design (Lawson et al. 2005):

Podium Structure – These structures are intended for mixed commercial/residential use. The first one to two stories are steel or concrete framed and typically support commercial occupancies. Load-bearing modules are then stacked on top of the podium and used for residential units.

Skeletal Structure – This type of structure is used to provide flexibility of floor planning to the owner. A steel skeleton is used for the superstructure and to frame out any intended open areas. Modules are then placed as needed. Both load-bearing and non-load-bearing modules can be used with this type of design.

The student housing project, shown in Figure 2.6a is an example of a hybrid structure. This 7-story building was constructed with a primary steel frame and a two-story podium. The first story was constructed below grade for parking. The second story has retail occupancies, and
the remainder of the stories contains 3D modular student housing units. A total of 1425 modular units were used for the construction of this building. The Steel Construction Institutes (SCI) claims a construction time reduction of 60%, for this project, over site-intensive construction methods (SCI 2003b).

The building shown in Figure 2.6b incorporated light steel framed 2D wall and floor panels with 3D modular bathroom units. The apartments are a maximum of six stories, and 16 weeks were saved from the overall construction period of 68 weeks (SCI 2003a).

![Figure 2-6. (a) Podium design Manchester University (Image by Lawson and Ogden, 2008), (b) Hybrid structure, Lillie Rd. Fulham (Image by Lawson and Ogden, 2008).](image)

### 2.8.4 Open building systems

The concept of “open building ” originated in the Netherlands in the 1960’s, generally as a result of the discontent of the public with the mass produced housing at the time (Cuperus 2001). The housing failed to suit the needs of the individual homeowner and customers felt as if they were not a part of the construction process and thereby disconnected from their residences.

The philosophy of open building systems strives to decouple the base-building (support) and fit-out (infill). This concept can be seen in the construction of modern-day leasable office space. Many times, the developer of the building does not know who the future tenant will be, so they provide flexibility in the floor plan leaving it open to the input of the future occupants. The developer provides the superstructure of the building, the functions necessary to satisfy local ordinance, and the mechanical systems to service the building. The functionality of the space is subject to the end user’s needs.
The end goal of an open building system is to provide a structure that can adapt to fit the needs of different users throughout its lifespan. An open buildings philosophy will incorporate protocol such that manufacturers of building components can produce standardized components for use as infill that is compatible with the base building. In this way, flexibility to the end user can be maintained, and connections can be made with different building elements from different manufacturers (interoperability). Protocol is necessary for individual manufacturers to successfully provide services as needed, directly to the customer without the need to consult the structure owner.

The School of Architecture and Planning at Massachusetts Institute of Technology (MIT) hosts an organization called the Open Source Building Alliance (“MIT House_n” 2012). MIT embraces open building system concepts with its “Open Source” philosophy. In a recent publication, the alliance summarized the benefits of “Open Source” as follows:

“Open source allows independently developed modules to work together in a larger system due to mutually agreed principles. It ensures integration and interoperability because: The system as a whole provides a framework –an architecture- that allows for both independence of structure and integration of functions. Modularity encompasses physical components, electronics, software, and services from design through use.”

Figure 4.6 shows an example of an MIT open-source modular solution proposed to a developer as an alternative to a traditional multi-family complex in Cambridge aimed at young urban singles and couples. MIT suggested that the developer provide the stacked open-sided modules that can be seen in figure 2.7a for the main structural elements, leaving the interior space fit out seen in figure 2.7b open to end-user input and development.
Many conceptual models are being developed under similar principles and show great promise for the future of building design and modular construction. Architectural firm ANDO has an innovative idea labeled by architects as “the mutant vertical city” (Parkins 2012). The project called NODO is a proposal for super dense housing in Beijing. This concept combines residences with public space and commercial lease space. Modules for this project are not stationary. The idea is that the modules can be relocated and rearranged as necessary. Figure 2.8 shows a rendering of the concept.

Open building philosophies show great potential for future use considering the advancements in BIM technology, manufacturing methods, and general advancement of internet access and computing power. Modern technology is making the design and construction of complex building systems more achievable.
2.9 Multi-Story Modular Construction for Multi-Family Buildings

The modular construction techniques discussed previously are those that are primarily used to construct multi-family, multi-story buildings. Multi-story modular construction has yet to be adopted on a large-scale in the U.S; however, reports from McGraw-Hill and similar organizations indicate that interest in this construction method is increasing. Residential multi-family facilities such as apartments, condominiums, student housing, work-force housing, public housing, and care facilities have been identified as ideal candidates for modularization due to their cellular repetitive nature.

The UK is currently an industry leader in the use of modular methods for the construction of multi-family residential buildings. Most of them are used for student housing, social housing or apartments with mixed commercial components. The Lillie Road Building (Figure 2.6b) is an example of a social housing project built in the UK, while the Manchester University project (Figure 2.6a) is an example of a mixed commercial, student housing project. Seen in Figure 2.9b is the Murray Grove Project constructed in London. The structure was designed for the accommodation of young singles and couples. The high-rise building shown in Figure 2.9c was designed to provide student accommodations. While UK dominates the landscape, there are also large-scale residential projects in the US; Figure 2.9a shows the proposed B2 high-rise slated for construction in Brooklyn New York. This building will be the tallest modular high-rise in the world if constructed and will be used for a combination of affordable and market rate housing units.

Figure 2-9. (a) B2 modular high-rise in Brooklyn, NY (Image by ShoP Architects) (b) Murray Grove housing project (CABE 2012) (c) Wolverhampton student housing project (MBI 2012a).
A noteworthy form of affordable housing gaining popularity in larger U.S. cities such as New York City and San Francisco is the small apartment unit (approximately 300 ft²), discussed earlier, and known as the “micro” apartment. Figure 4-9 shows a rendering of a sample floor plan for such an apartment. This particular unit will be located in the Soma studios project shown in Figure 2-4. The housing units are once again aimed at young singles and couples. Micro apartments are good candidate for modularization due to their small, easily transportable, size and repetitive floor plan.

![Figure 2-10. 300 ft² Soma Studio micro-apartment interior rendering (Courtesy of MBI).](image)

2.9.1 Appropriate Multi-Family Uses For Multi-story Modular Construction

The literature review has led to the identification of several application compatible with modular construction methods that are currently being used in the building construction market. The identification of these applications fulfills the 2nd objective of the research. In this section, the residential applications appropriate for multi-story modular construction methods are summarized. This ends the initial literature review portion of the research. Following this section is a discussion on some interviews that were conducted to support the literature review and general commentary/summarization of important points/questions gathered from this review.
2.9.1.1 Affordable housing for young urban singles and couples

A large percentage of the modular multi-family buildings discussed in this report are designed for occupancy by members of the urban young single and couple demographic group (i.e., student housing, affordable housing). One or two-person families typically do not have need for large storage space, do not typically spend the majority of their time occupying their dwellings and are likely to be a part of the middle-income tier. Young singles and couples are likely to accept smaller accommodation if the cost is fair and the units are located within close proximity to their jobs and entertainment. Therefore, it is possible that smaller housing units can be acceptably designed and marketed to this group.

From statistical data, the young singles and couples demographic group appear to be growing in America, especially in urban areas. Young college graduates tend to move to urban areas in search of job opportunities. Urban living is expensive and housing costs can be restrictive. The statistics indicate that there is a need for affordable housing in this category. A recent Pew research (Pew Research Center 2012) study based on a survey of 1287 middle-class adults and supplemented with statistical data from the 2010 U.S. Census reported the following enlightening statements (in comparison to the previous census taken in year 2000):

- 85% consider it more difficult to maintain their standard of living now versus a decade ago.
- Middle-tiered household income dropped from $72,956 annually to $69,487 annually (incomes in 2011 dollars).
- The net worth of individuals in this income bracket plummeted from $152,950 to $93,150, a level last seen in 1983.
- 61% of adults were in the middle tiers income in 1971, whereas 51% were accounted for in 2011. The missing 10% was basically split and distributed amongst the lower and upper income brackets.

In the following, the U.S. Department of commerce reports several interesting statistics that indicate growth in urban one and two-person families:

- US Census Bureau’s 2010 Population Report (U.S. Department of Commerce 2011) indicates that the age of first marriage for men increased from 26.8 in 2000 to 28.2 in
2010 and from 25.1 to 26.1 for women. The percentage of adults married in 2000 was 57.3 and the percentage in 2010 was 54.1.

- The average household size declined from 2.62 in 2000 to 2.59 in 2010. Part of this reason was the increase in single person households which increased from 25% in 2000 to 27% in 2010. The 27% rate is nearly double the 13% rate observed in 1960 (U.S. Department of Commerce 2012).
- The percent population living in urban areas increased 10.8% to 87.3% measured from year 2000 to 2010 (U.S. Department of Commerce 2011).

### 2.9.1.2 Social and supportive housing

The U.K. has successfully built low to mid-rise examples of social-housing. Low-income social housing projects may be an appropriate use for low-rise and mid-rise structures. The Lillie Road Project previously mentioned was a model example for future British social-housing.

### 2.9.1.3 Work-force housing

Markets can exist for low-rise modular workforce housing in locations such as the Marcellus shale-rich, rural areas of Pennsylvania. The influx of residents due to the development of natural gas wells has overwhelmed the ill-prepared rural areas; as a result, housing is in demand. HUD specified these areas as having a housing shortage (HUD 2012). The quick construction of attractive, high performance, comfortable housing units could be appealing to both community and end-users in these areas. The re-locatable buildings defined earlier in the paper also offer an additional non-permanent solution.

### 2.9.1.4 Disaster-Relief and military housing

Multi-story modular buildings both re-locatable and permanent are prime candidates when considering disaster-relief housing and military housing. Both forms of housing are required to be constructed and deconstructed quickly in many cases. Often the construction
environment may be hostile and lacking in resources. Modular buildings are able to be pre-designed, constructed quickly with minimal construction equipment and can be deconstructed and reused if necessary for another project. Modular construction methods have clear advantages over site-built methods in this category.

### 2.10 Interviews with professionals

In order to partially confirm the sentiments toward modular construction and uncover any additional issues, not readily apparent through literature review, a set of four phone interviews were conducted. The interviews were focused on design professionals since they are typically the members of the design team that has the most influence on the selection of a building system. The interviews included two State College principal structural engineers (Davis 2013), (Schneider 2013), one State College principal architect (Olsen 2013) and one New York City project structural engineer (Mugford 2013). A transcription of the full interviews is located in Appendix B. The interviews were fruitful and indeed brought to light some considerations that did not turn up in the literature review. Some important insights obtained from the interviews are listed below. The interviews were part of the initial state-of-the-art review and will not be expanded on in later research.

Literature review is a valuable first step in any research project, but has its limitations. The interviews were successful in identifying issues that were not able to be brought to light by simple literature review. The literature review provided extensive information and understanding, including some of the challenges facing the modular industry, but the few phone interviews revealed additional issues that may affect the use of modular construction in the modern multi-story market. Summarized below is a few of the important points made in the interviews:

- All the interviewees were open to the idea of using modular methods but are currently either not considering its use, have not had requests for its use or have found it to be not cost effective.
- None of the interviewees had any relationship with local modular manufacturers.
- All the local interviews have only seen the use of modular methods for single-family dwellings and townhomes. The New York City engineer is familiar with multi-story modular construction.

- None of the local engineers are using BIM, but the New York City engineer is using BIM.

- Block and plank construction is a competitor to multi-story modular construction.

- All interviewees considered modular construction as a way to save time, improve project management and reduce community disturbance.

- Contractors are resistant to the use of new types of construction methods.

- Trade unions may deter to the use of modular construction methods. An advantage of using off-site construction is it can provide a mechanism for general contractors to circumvent prevailing wage requirements on some projects.

2.10.1 Questions and comments generated by the interviews and literature review

The literature review points toward a market for modular construction methods within multi-story residential building construction category and the possibility of short term increases in productivity gained with its use. The interviews, however point to the relatively undeveloped use of the technology in the construction industry.

There is some evidence from literature review and interviews that the use of multi-story modular methods and BIM are beginning to be used in the major U.S. cities like New York. This is not surprising because the need for housing, construction time savings and reductions in community disturbance typically exists in dense cities such as New York. In less urbanized area, such as State College, PA, where 3 of the 4 interviews were conducted, multi-story modular construction methods do not appear as if they have infiltrated the building construction market. The work conducted for this review generates a few questions:

1. Do we currently have enough modular facilities in the U.S. capable of producing high quality, cost-effective multi-story modular units?

2. Are the challenges and constraints of using modular construction sufficient to restrict increased future use of this construction method?
3. Is state-of-the-art equipment established in the modern modular facilities or is the majority of the current equipment outdated in most factories?

4. Is there enough market to incentivize modular manufacturers to invest in technology?

5. Are there additional markets for alternative multi-story modular philosophies?

It is relatively clear that modular construction methods currently being used in building construction have a niche market and definite benefits associated with its use if used appropriately, it is not clear, however, if the industry is prepared to use this technology efficiently moving forward. It is also not clear if the multi-story modular methods being used currently are appropriate for buildings of the future. McGraw-Hill (McGraw-Hill Construction 2011) notes that, “Building information modeling (BIM), modern manufacturing methods, sustainability goals and recognized productivity gains rejuvenate century’s old-construction process.”

Productivity gains and sustainability goals are fairly well recognized but BIM and modern manufacturing methods seem to be the uncertainties in this statement. The preparedness level of the modern manufacturing facilities to develop multi-story modular technology is questionable. The acceptance level of BIM within the AE community is questionable as well.

BIM and companion technologies such as cloud services coupled with modular construction methods (model-driven prefabrication) should theoretically produce maximum gains in productivity. BIM is a good companion technology for modular methods but is not firmly established in the AE community, the benefits of using this philosophy are still being debated and BIM standards are just beginning to be developed. It is probable that the AE community will eventually accept BIM philosophies, but it may be some time before this occurs.

2.10.1.1 Comments on efficient use of modular construction methods

Efficient use of modular technologies should lead to maximum cost-savings. The following is list of good practices that has been learned from the literature review and interviews that should lead to the efficient use of modular construction methods:

- Modular methods are used for a suitable project, such as those that have repetitive, cellular elements,

- Modular methods should be most efficient when the project is conceived as a modular project and manufacturers brought in early in design process for advice,
BIM and model-driven prefabrication can improve productivity,

- Social problems are not dominant in a project (Unions, contractor resistance, perspective, etc.), and,
- Local labor prices are high relative to those at a nearby off-site facility.

### 2.10.1.2 Comments on cost-savings features of modular construction methods

The literature review suggests that cost-savings should occur as a result of direct reductions of building construction soft-costs, but insufficient evidence was discovered in the review to verify the cost-saving aspects of multi-story modular construction methods. All the case studies reviewed reported time savings and improvements in project management, many reported reductions in community disturbance as well, but few concrete cost-savings elements were discussed. The two cost saving elements reported, other than mass production benefits, were as follows (“Market Commercial Housing” 2014):

- ASU dorm project shown in Figure 2.1d reported reductions of material transportation unit costs,
- Philadelphia “Modules” project shown in Figure 2.1f reported saving in labor costs due to offsite construction.

The cost-saving features of modular construction appear to be debatable and often dependent on the project type and level of management quality. They may be difficult to identify and document or they may be minimal in comparison to other site-intensive construction methods. The interviews suggest that site-built methods may be more cost-effective given an unlimited time period. On the other hand, factory production should ultimately produce cheaper products. The following may be possible:

- The current modular construction methods are dated and unable to produce the full cost savings available with off-site factory production,
- Projects just have to be large in order to recognize the economy of off-site production,
- There are inefficiencies in the current modular design philosophy.
One such inefficiency identified through interview is the concept of a 6-sided volumetric module. The redundancy in floor / ceiling, and wall elements may reduce the cost-effectiveness of the construction method in some instances. Other potential inefficiencies in the current modular philosophies may exist but have not been identified yet. In any regards, it is worth mentioning that the cost-saving features of modular construction methods are not clear for small to medium size projects and there may be inefficiencies present in modern methods.

2.11 Commentary on Literature Review and Interviews

In general, considering the literature review, factory tours and discussion with the interviewed, there appears to be positive sentiment regarding the use of modular technology within the building construction industry; there also seems to be opposition from some members of the construction community and challenges to overcome. The main challenges identified are the questionable readiness level of the U.S. factories, the inexperience of construction industry with this technology, the general unawareness of its existence, social stigmas regarding the term, “Modular”, contractor and trade union resistance to new forms of competing construction, BIM readiness level, lack of design guidance and questionable cost-saving features of the technology. Continuing basic research is needed to address some of these questions and present guidance for the construction industry, so that improvements can be made to multi-story modular construction methods to a point where industry can have confidence with the use of this form or prefabricated construction.

The development of modular construction methods can be important for prosperity in the short-term building construction market and equally important, if not more so, for the buildings and cities of the, not so distant future. In the short-term modular stacking philosophies can be used to quickly provide cost-effective, high-quality structures that might serve to satisfy current affordable multi-family housing demand, work-force housing demand in areas such as the Marcellus-shale boom areas of the northeast U.S, and disaster-relief housing needs such as those that might occur in the hurricane stricken regions of the U.S. east coast. In the long-term outlook, modular technology might be used for many of the same types of structures, but the use of it may be expanded to include the production of modular units that will serve in modifiable buildings such as those that can be seen near the top of Figure 2.11.
**Multi-family buildings in 50-100 yrs.**

Living buildings begin to emerge. Super-structure and in-fill are likely to be separate and In-fill constructed with modular components that can be updated and re-arranged. Buildings are likely to have a combination of occupant, private and public owners.

**Near future development:**

Semi-permanent modular structures will emerge, based on open building principles, in which the units are relocatable.

**State-of-the-Art Application:**

A few permanent modular high-rise building are being designed and constructed.

**Emerging Application:**

Modular low-rise multi-family construction is beginning to emerge and develop.

**Current Application:**

Modular technology is used mostly for manufactured homes and single-family dwellings.

**Figure 2-11.** The figure depicts the use and progression of modular construction as the industry becomes more advanced and building design philosophies change over time (Images by ARUP, DesignBuild Source, ShoP Architects, MBI).
Figure 2.11 illustrates the conceptual development of multi-story modular methods in the U.S. as they pertain to multi-family housing. A full commentary on multi-family buildings looking forward is located in Appendix C. The arrangement of structures was developed from information learned in literature review. As building design philosophies and requirements of buildings change over time the role of modular construction can change as well. In the U.S, modular construction methods are largely being used to construct single-family dwellings (Lawson et al. 1999).

Multi-story modular complexes are beginning to be constructed in dense urban areas such as New York. A singular example of a modular high-rise is currently being constructed in Brooklyn, New York. As the industry progresses, it is possible that non-structural modular units will be constructed and used as infill for modifiable buildings. It is also possible that as building design advances the buildings will perform multiple functions for the community such as those outlined in the multi-family commentary in Appendix C. In this scenario, modular infill units may become a key infill element for these types of building in which they can be removed for service, repair and rearrangement.

The multi-story modular construction industry has the potential to become a strong high-tech manufacturing field with some support and development. The U.S. manufacturing sector has suffered in recent years due to the more favorable business environments in other countries. President Obama in his recent state of the union address given on February 2nd 2013 pointed out that we, as a nation, need to revitalize our manufacturing sector in order to remain competitive in the global economy. Having a strong manufacturing sector is important in many aspects. It not only provides stable jobs and wealth to the community, but also helps a nation mobilize in times of need. It would be in the direct interest of the U.S. and in agreement with executive opinion to foster and promote promising technologies, such as multi-story modular construction.

The use of prefabrication methods is forecasted to increase in the future construction market, but the proper application and design methodologies of these methods has not yet been clearly defined. There are positive attitudes and hopes regarding modular technology, but there is also resistance in the building construction industry to this technology, inefficiencies and competing construction methods in the multi-family market. It may be typical resistance to new ideas, but it also might mean either the right applications have not been found for it in the U.S. or the methods need revision and update. Multi-story modular construction methods are relatively new to the U.S. but the stacking concepts were developed primarily in war-ravaged European countries decades ago to accommodate housing shortages (Lawson et al. 1999). Perhaps, moving
forward we need to investigate whether these concepts are still applicable and if so, should they be modified to better fit the needs of this generation. Overwhelmingly, many think that conceptually, the use of modular technology for the construction of buildings should improve productivity in the construction industry, but how to properly implement it and achieve those improvements is the question.

In summary the concept of multi-story modular construction seems to be aligned with the goals of the construction industry and does show definite potential if used for the right application. The U.K. has been using this construction method for some time now with success and there has been some success with these types of structures in the U.S. as well. The industry as a whole seems positive about the use of these methods, but competition and social resistance do exist and should be recognized. As mentioned before, basic research is needed to provide the construction industry with the tools it needs to utilize these methods and review is necessary to identify proper application and ensure what methods are appropriate for the next generation of buildings.

Our urban areas are growing and becoming more populated and our natural areas face increasing pressure from the built environment. It is well accepted that we need to become a more sustainable society, but how do we get there is the question. Fostering the development of sustainable technology such as modular construction can help the U.S. become a world leader in a high-tech manufacturing industry.

2.12 Summary of Important Points from State-of-the-Art-Review

The state-of-the-art review incorporates knowledge gained from the initial literature review, an interview session with professionals in the building construction industry, attendance of a conference sponsored by the Modular Building Institute (MBI) and tours of four Pennsylvania modular manufacturers. There has been much learned during the review, but there are a few points that stand out from the others. Following is a summary of the important points that have influenced the selection of the proposed applications defined in Section 2.13 and helped to shape decisions made throughout the vertical expansion research process:

1. The U.K. is a leader in the multi-story modular construction industry.
   a. Using multi-story modular construction methods before U.S
   b. Documented research regarding design guidance
c. Available case studies
d. Many different types of modular manufacturers and modular systems historically.

2. U.S. based modular design guidance is largely held by modular manufacturers’ or their third party engineers. Manufacturers are hesitant to share due to proprietary interests.

3. Cold-formed steel modules are widely used in the U.K. but not in the U.S. U.S. manufacturers tend to use either wood-framed modules or corner-post bearing modules. Moment framed modules are used infrequently in both countries.

4. Time savings and reduction in business operations or community disturbance are the primary reasons for opting for modular construction methods.

5. Design guidance in the U.K. is not directly applicable to U.S. projects due to differences in design codes.

6. Tools like BIM, VDC and cloud services have the potential to ease the implementation of prefabricated components into a construction project.

7. Modular construction serves a limited market.

8. The use of modular construction is restricted by geometric limitations.

9. There is material redundancy present in the floor-ceiling assemblies and wall assemblies in all 6-sided modular approaches.

10. The attention to planning in the design phase and involvement of a modular manufacturer early in the design process can influence the success of a modular project.

11. Design changes are hard to make and can be costly once production of the modules have begun.

12. A modular manufacturer must be within range of the project in order to consider modular construction.

13. Because of factory overhead modular manufacturers have a break-even point. The break-even point varies between manufacturers and module type.

14. Manufacturers have different labor management approaches:
   a. Maintain staff of full time labor
   b. Sub-contract most labor, only hire when needed

15. Modular construction is ideally suited for floorplans that are regular in dimension and can be broken down into repeatable cellular units. Residential uses can be ideal. Benefits of using modular construction decrease as the complexity of the floor plan increases and the ability to repeat units decrease.
16. Modular construction is subject to many codes and it is not clear which govern in some cases:
   a. HUD guidelines in the factory
   b. Building Code in the field
   c. Transportation Codes in transport
      i. Federal
      ii. State
      iii. Local

17. Modular construction is well suited to produce dwellings for the singles and couples demographic group. Census data indicates that the singles and couples demographic group is increasing in the U.S. Census data also indicates that U.S. population is trending toward cities.

18. The demand for multi-family housing is increasing and is forecast to continue increasing in the near future.

19. Interest in sustainability is increasing in the construction industry.

20. Modular construction can be used to move complex operation or high levels of detailing into a controlled factory environment.

21. It is possible that Multi-family buildings of the future will serve more functions in society:
   a. Public and private ownership
   b. Self-sustaining, performing functions for the public good such as air-filtration
   c. Vertical development
   d. Separation of structure from infill:
      i. Replaceable units
      ii. Modifiable floor plans
      iii. Use of non-structural modules

22. The majority of the application of modular construction and BIM is seen in the larger cities such as New York City.

23. U.S. Modular manufacturers appear to rely more on labor than equipment.

24. Interviews identified:
   a. Potential union resistance
   b. Advantages of modular construction in prevailing wage jobs
c. Block and plank as a strong competitor to modular construction for the construction of apartment buildings.

25. Modular construction has advantages for jobs in areas where off-site labor wages are lower than local labor rates.

2.13 Selection of Under-Utilized or New Use for Multi-Story Modular Construction

The information gained from the state-of-the-art review was utilized to conceive/find a selection of applications for multi-story modular construction that are either underutilized in the U.S. or new to the multi-story modular building construction market. In this step lessons learned regarding modular construction methods and current successful applications of it were used to identify applications that could be good matches for the construction method. The rationale is that because multi-story modular construction techniques serve such a limited market identification of potential uses for the techniques would benefit the modular industry. Additionally the design guidance and process identified in the research will be useful to those who are considering the application or similar.

As mentioned in the discussion regarding project goals, there were three application that were considered; MOD’s, CORE and Vertical Expansion. A brief summary of each follows:

1. MOD’s – an original concept that conceives the use of non-structural modules to provide mobile dwellings for the young singles/couples demographic group. The key concept behind MOD’s is that it is a dwelling that the purchaser can own and take with them as they move from place to place, over the years; much like a modern day mobile home. The difference is that MOD units can be used in either a stand-alone ground mount setting or inserted in a multi-story structural frame. Points 8, 15, 17, 18, 21 and 19 from Section 2.12 influenced the conception of MOD’s. The MOD’s concept was in line with current applications, the units would be able to be easily mass produced, and the idea seemed to fit the vision of multi-family buildings of the future described in Appendix C. Additionally, the concept could be shaped to appeal to the younger generation whose demographic group was identified to be increasing in the U.S. Final points on this concept are that the provision of housing for young couples and singles have been identified in the literature review as an ideal use for modular construction and the demand for multi-family housing is increasing
currently. More information regarding the MOD’s concept can be found in Appendix A.

2. CORE – Multi-story utility core for residential buildings. This concept has been used in the past, but is not greatly used in the U.S. building construction market. Points 4, 15, 19, and 20 had influence on the decision to consider this application. Modular construction is currently successfully being used in the U.S. for the provision of multi-trade racks for commercial projects. Also, interest levels in sustainability and high-performance housing appear to be on the rise. Utility cores can be used to consolidate MEP equipment in a home to improve the energy efficiency of the home. The time consuming, complex arrangement of MEP equipment can be moved to the factory environment, increasing the chances of proper installation and set-up. Standardized cores can be reproduced in a factory environment. By choosing to use a CORE system the homeowner can gain both energy efficiency savings and construction time savings. More information regarding the CORE concept can be found in Appendix A.

3. Vertical Expansion - Vertically expanding existing commercial buildings with residential dwellings using modular construction methods. This idea has been found to be used in the U.K. for single-story expansions in the past. Points 4, 15, 17, 18, 22 had influence on the decision to consider this application. Vertical expansion was thought to be good use for modular construction because the building owner could add additional income generating dwellings onto their existing building. Modular construction would benefit the owner because the units could be installed quickly with minimal disturbance to both business operations and community and the owner could see an early return on investment. Modular construction would be beneficial in an urban environment where most suitable buildings would be located. Additionally from review, it appear there were ample low-to-mid-rise building available in the U.S. that could potentially be candidates for an expansion.

The decision was made to further investigate the 3\textsuperscript{rd} application, which is the idea of vertical expansion. Further detail will be discussed regarding this concept in the next chapter.

Vertical expansion was selected because it was thought to be the choice that could be implemented the quickest in the building construction market and also because
it appeared to be well-suited to the modular construction techniques used currently by manufacturers.

The idea of MOD’s is well-suited for modular construction but is conceptual and would require significant development that could cause the research to stray from the original goals. Research for MOD’s would likely require the development of alternative modular production techniques, investigation into advanced materials and development/identification of appropriate structural systems for both the MOD units and the independent structural framing system. This type of research is much needed, but is out of the scope of this project. Although the idea could produce a new modular market, the application did not appear to be adaptable to use in the near future.

CORE was not selected because it seemed to have limited impact on the modular construction market. Although it could be a niche market in itself and provide a useful product, the use of the product seemed to serve a limited market. Also, the CORE concept would require significant development during the research that could cause the research to stray as in the case with MOD’s. Utility cores have some application in the U.S. building construction market, but new ideas are needed in this field to make the cores more useful to builders, designers and homeowners.
Chapter 3

Vertical Expansions of Existing Buildings Using Modular Construction

Modular Construction is appropriate for the construction of buildings with repetitive floor plan elements. Residential structures such as apartment buildings, student housing and workforce housing tend to be ideal candidates for modularization. The projects that are highly compatible with modular construction methods tend to be those that would significantly benefit from off-site construction, construction schedule time-savings, and reductions in community disturbance or business operations.

Renovation projects, particularly those planned for congested urban areas, can potentially take full advantage of these benefits. Initially, by choosing to renovate a building versus constructing a new one, owners can preserve the historic nature of their building and its relationship with the surrounding community, as well as take advantage of the existing embodied energy, avoid expensive foundation and site activities, and eliminate land purchases.

Vertical expansion is an approach that can be used to add roof-top apartments to buildings that are able to accept expansion (conceptual expansion shown in Figure 3.1). Vertical expansion, if feasible for a given existing building, can provide the financial benefits gained from rental or sale of the new units as well as be a part of a more comprehensive roof renovation plan that would not only add more square footage to the building but can simultaneously replace aging roof components and improve the energy performance of the roof system.

*Figure 3-1.* (a) Rendering of an existing commercial building (b) Vertically expanded building.

The modularization of an expansion can introduce the benefits of off-site construction, such as lower wages, high quality components and just-in-time delivery schemes. The benefits of modular construction can have value to a building owner who desires to accomplish renovations
quickly, while maintaining the operation of an existing business. The Steel Construction Institute (SCI) suggests the following advantages modular construction may have if applied to an expansion project (Lawson 2008):

- New facilities are added cost-effectively
- Construction is rapid, which minimizes costs and disruption
- High-quality can be achieved by off-site manufacturing
- Delivery of modules can be timed to suit local conditions
- Light-steel constructed modules may not over-load an existing building
- In some projects it is not necessary for the occupants to move out during renovation

3.1 Rationale

Many of the markets that multi-story modular construction methods are used in have site-built competitors, which can lead to social resistance from contractors that make their livelihood from using these traditional construction methods. In these markets it may be challenging for modular construction methods to establish a significant presence and find dedicated users. It would be much easier for modular construction methods to gain a foothold in markets that have less competition, social resistance and those that take advantage of the strengths of this construction type.

It is important to recognize the strengths and weaknesses of modular construction methods, in order to take full advantage of the strengths and avoid the use of these methods when the project requirements would expose the weaknesses. As discussed in the literature review, modular construction methods have distinct advantages over site-built construction methods when utilized appropriately. The projects that are highly compatible with modular construction methods are those that would significantly benefit from off-site construction, construction schedule time savings, and reductions in disturbance of community or business operations.

Modular vertical expansions can take advantage of the strengths of the construction methods. In particular, adding modular residential units on top of existing commercial buildings. The use of modular construction for vertical expansion of existing buildings has several potential advantages over site-built construction methods and can become a niche market that is well suited
for modular technology. Following is a discussion regarding the suitability of multi-story modular methods for use in the construction of vertical expansions.

In some instances, expanding an existing building vertically may not be practical with any other construction form of construction. A modular expansion to an existing building can be performed quickly with minimal disturbance to the business operations and the community. This is a clear advantage over any comparable site-built method, which would require much more time, on-site equipment, labor, construction traffic and materials to construct the same expansion.

An off-site construction method, such as modular construction, is well tailored for this type of project and can reduce the actual construction time. The use of off-site construction allows all the modules to be completed ahead of time and stored until a convenient time is selected for erection. The erection is accomplished with a small crew and a crane. This drastically reduces construction time and deliveries, thus reducing disruption of the day-to-day activities of an existing business.

Initially, factory conditions allow for tight tolerances and high quality control, which can provide designers with an increased comfort level specifying complicated details and alternative building materials that normally might not be practical in a site-built situation. Additionally, a project owner may benefit from the differences in labor costs between the off-site location and the project location. This may be an important benefit, especially for urban or union projects where the labor rates can be prohibitively high.

Secondly, there should be little competition and social resistance in this market from site-built competitors. For reasons discussed earlier, it is not likely that a vertical expansion of an existing business using site-built construction methods would be able to compete with a well-planned modular solution on the same level as that of the construction of a new multi-family building. Excessive down-time and disruption could be prohibitive to functioning businesses. Two stories of modular residential units can be erected in a few days where as constructing two stories using traditional site-built methods may take a months. Modular construction is the clear winner when discussing minimizing down time, which can be important to project owners that are trying to expand a building while running a business simultaneously.

When the competition is lessened in a market so follows most of the social resistance. Social resistance to modular construction can come from the contractors and unions that make their living using site-built construction methods or past stigmas attached to modular construction. If using site-built methods is not an option then resistance from these groups should
be minimized and architects should feel more confident recommending it as an option to a potential client.

Finally, it is likely that a light-framed modular addition will weigh considerably less than some site-built solutions (typically structural steel framing). This is an important point because dead-load considerations are of primary importance when considering a vertical expansion; less weight means less stress to the existing structural system. This could allow for additional stories to be considered and could be a decisive factor in determining the feasibility of an expansion.

In general, using modular construction methods to vertically expand a commercial building with residential units vertically, on the surface, would seem to have merit and reduced competition from site-built competitors. The addition of a few stories of residential units could likely provide adequate compensation for the investment if construction costs are reasonable and business operations are not terribly disrupted. In cases where the existing building elements are able to resist or be easily modified to resist the additional loading, then vertical expansion may prove to be a rational and economical option that could increase the buildings earning potential with a modest investment. In the following sections, the portion of the research designed to evaluate the feasibility of using modular methods for vertical expansions is explained.
Chapter 4

Modular Vertical Expansion Research Study Design

To satisfy objective three, of the overall research, a new application for multi-story modular construction methods was to be selected that was identified, through literature review, as potentially appropriate for the U.S. construction industry. By selecting a new application and performing research in this area, new information can be added to the U.S. prefabricated construction methods knowledge base. The research conducted on the topic of vertical expansion is design to satisfy the 4th objective of the overall research.

It is proposed that, based on what was learned from literature review, that expanding buildings vertically using residential modular units could be an appropriate use for modular construction methods. This use, seemingly, has potential for application in the current building construction market, but needs research to identify items that will affect the feasibility. The review indicated that modular vertical expansion is enacted in Europe as a renovation technique, but its use in the U.S. is not well documented. Construction market conditions and building codes in the U.S. are different than in European countries. In order to identify factors that affect feasibility of using this renovation approach in the U.S. it is important to study the approach using common U.S. modular construction techniques and evaluate the expansions using accepted U.S. engineering design practices.

The research will be conducted by accomplishing tasks that involve additional literature review, U.S. design code review, case study selection and the analytical modeling of several modular vertical expansions on a case study building.

4.1 Research Study Methodology

Case study and analytical modeling were chosen as the research methods for the study. Case studies are generally used to bring about an understanding of complex issues in a situation. In this research the complexities of implementing the prefabricated method of multi-story modular construction is the object of study. A case study is necessary to bring to light the factors
that will affect the feasibility of implementing this particular type of prefabrication. It is important to apply this application to a design situation involving a real example. Many factors that affect feasibility cannot be revealed until the application is put into practice. It is necessary, when performing a case study to fully develop the process. In this manner, important elements can be observed from the various parts of the design process, which might get overlooked in a less rigorous process evaluation.

The 2nd method used in the research is analytical modeling. It is necessary to create an analytical model, such that analyses can be conducted to produce results that will affect decisions during the process. These decisions are what is important to the research and should be brought to light in order to discuss how they may impact feasibility. Additionally, the focus of this research is the structural engineering aspects of modular construction. It is necessary to accurately reflect the existing and proposed conditions with an appropriate analytical model of the existing building and proposed expansions. The analytical model allows for structural analysis to be performed and commentary to be made on how the expansions will affect the existing building. Making judgments on how a proposed expansion will affect the existing structural system is of top priority in a vertical expansion project.

4.2 Goals of the Vertical Expansion Research Study

The goals of the study were developed to satisfy the 4th project objective. Upon completion of the study the 4th project objective will be satisfied and the results will be used to complete the 5th objective. The goals of this study are:

1. Describe/develop a design process for vertically expanding an existing building.
2. Identify factors, learned from the design process, that affect feasibility of vertically expanding an existing building using modular construction. Distinguish those that are relevant to modular construction.
4.3 Study Design

Studying the design process for this prefabricated method is important to the success of the research and achievement of the overall goals. Because of the high-level of prefabrication involved in a modular construction project many decisions regarding design and construction of the building must be made early on in the design process. In these early stages of a project design/conception, critical details that can affect project success can be easily overlooked. Missing detail in these stages can negatively impact productivity gains anticipated by including prefabricated components in a project design. This is why it is important to study the design process to identify those factors that can affect the feasibility/productivity gains anticipated and report them to potential users of modular construction.

The study has been designed to identify a design process for vertical expansion and review the process. The case study is used to explore the factors (within the scope) that could have an effect on the feasibility of vertically expanding existing buildings using modular construction. In the study an analytical model of the case study building is created and structural analysis is performed on the model to develop baseline information regarding reserve capacity of the existing building structural system. The analytical model is used to develop an appropriate vertical expansion concept for the building.

Two modules types, identified in the literature review, that are commonly used in the U.S. are wall-bearing modules and corner-post bearing modules. In order to develop an understanding how module type influences feasibility, analytical models are created for both modules types and expansions are conceived using both types. One- and two-story expansions are model using both styles of modules for a total of four expansions. The factors that found to affect the feasibility of this specific case study are generalized, such that they can be applied to other modular vertical expansion projects.

4.4 Research Study Scope

The scope of this project is limited to the vertical expansion of an existing building with residential units using stacking modular construction methods. The following is included in this study:

- A single case study (HMAC)
- Analytical modeling of the case study adequate to identify factors that affect feasibility of expansion.
- Development of a design process adequate to discuss the factors that affect feasibility of expansion.
- Definition of analytical models for both a wall-bearing and a corner-post bearing module that are adequate to discuss the factors that affect feasibility of expansion.
- Addition of four separate expansion to the analytical model. A one-story and two-story expansion with both the wall-bearing and corner-post bearing modules.

Structural remediation options for existing building structural elements may be discussed in general terms during the course of research, but it is not within the scope of the project to design these or elaborate on them. The research concentrates on investigation of the concept of modular vertical expansion as it applies in the U.S., defining situations where it might be able to be successfully implemented, evaluating the general structural performance of several modular vertical expansions and provide commentary regarding the use.

This study is primarily concerned with the structural design characteristic and relevant architectural information needed to determine the appropriate selection of modular construction method and identify those factor that can affect feasibility of expansion. The economy of an addition may be discussed in general terms, such as those that might prohibit the use of this concept or encourage the use of another, but a detailed analysis from a construction management point of view will not be provided. Additionally, detailed structural analysis and design of the modules is out of the scope of the project and will not be conducted. Structural analysis is limited to that needed to discuss the effects of the expansion on the original structure.

4.5 Case Study

An existing building in which a vertical expansion has already been planned was provided for case study. Preliminarily the provided case study appears to be a building which might be a likely candidate for a vertical expansion. As will be discussed, the building appears to be located in an area that could absorb new dwellings, the building appears stoutly constructed and the likelihood of the structural system being capable of handling the expansion is good, the building is located in an urban area, which would increase the likelihood that a prefabricated
construction method would be of benefit and the building is also of a mixed use containing several business operations that could benefit from the limited disruption. Further information regarding case study evaluation can be seen in Chapter 7.

4.6 Research Study Objectives

To accomplish the goals of the study and ultimately comment on the factors that affect the feasibility of using modular construction methods for vertical expansion, several objectives have been defined. The objectives are used as guides to help develop the design process. The following is a list of the project objectives:

1. Identify design criteria, governing codes and develop an understanding of how buildings are expanded using both site-intensive construction methods and modular construction.
2. Create the analytical model of the existing building and perform structural analysis to determine baseline structural capacity information.
3. Define modules and expansions. Analyze the effects of the expansions on the existing structure.
4. Evaluate the design process for factors that can affect feasibility and summarize those factors.

4.7 Research tasks

Initially, further literature was required to help define the design environment, design methodology as well as structural criteria necessary to model the expansions and extract useful information from the models. One of the factors that is influential in evaluating any vertical expansion would be the regulatory environment.

Module selection for any specific vertical expansion is likely to be highly dependent on local building code requirements and the estimated height of the expansion. Often when the building height increases, building code requirements become stricter. The requirements regarding structural design, construction type, egress, accessibility, and fire-protection are those that will directly affect the feasibility and cost of a project. These same requirements are those
that can be used to initially determine the type of building modules that are best suited for a given expansion. In addition to the building code requirements, the structural and geometrical limitations of a particular module type might restrict its use as story height and external loading increases; therefore, it is equally important to define the structural performance characteristics of each class of module.

The tasks relating to the first objective lay the groundwork for the remaining objectives to be completed. The applicable building codes are researched and are related to the different module classes. During this stage, the design module candidates are described and the structural systems are discuss.

As mentioned previously, an analytical model of the existing building will be created based on existing plans or site measurements. The existing buildings will be analyzed to determine baseline load effects in structural elements.

Upon completion of the initial building modeling, a plan will be developed to vertically expand the buildings by adding residential modular units. These expansions will be added to the initial models and analyzed to determine design challenges, design methodology recommendations and likely effects on the existing building. Comparisons will be drawn to discuss the expansion effect on structural elements, potential remediation requirements, and suggestions to improve the design process or construction method (i.e., manufacturing improvements, better selection of existing module type, alternative structural design for module, alternative module construction materials. etc.).

4.7.1 Description of Tasks

Objective #1

Identify design criteria, governing codes and develop an understanding of how buildings are expanded using both site-intensive construction methods and modular construction.

1.) Tasks:

a.) Review examples of vertical expansion of existing buildings; both those constructed using site-intensive construction methods and those using modular methods.
b.) Identify applicable building codes relevant to vertical expansions.
   - Sub-task:
     - Review IBC and IEBC requirements for pertinent structural design requirements.
     - Review ASCE documentation for appropriate external loading, applicable design criteria and guidance.
     - Literature review to define similar structural design precedents.

c.) Conduct IBC assessment to document how the following requirements within the building code change as story height increases and discuss how changes might affect module selection.
   - Example topics:
     - Construction type
     - Building height and area
     - Fire protection
     - Egress
     - Accessibility

d.) Document non-structural considerations that might affect the feasibility of a modular vertical expansion.
   - Example topics:
     - Mechanical system routing
     - Access to new addition
     - Building enclosure concerns

e.) Document Structural design considerations
   - Loads assessment
   - Existing building evaluation
   - Structural analysis and design approach
   - Load transfer from proposed to existing structure

f.) Identify an appropriate design strategy for the vertical expansions and describe expansion considerations that would be useful to structural engineers or architects.
g.) Identify module types, commonly used in the U.S., to be used for the expansions.

**Objective #2**

Create the analytical model of the existing building and perform structural analysis to determine baseline structural capacity information.

1.) Tasks:
   a.) Perform site visit to document existing building geometry and structural system.
   b.) Define existing external loads.
   c.) Create an analytical model of the original building and perform structural analysis to assess demand on structural system components due to existing conditions.
      • Sub-tasks:
        – Model the existing building in ETABS.
        – Define the building structural system.
        – Apply the appropriate loads and load combinations.
        – Define materials and structural shapes.

**Objective #3**

Define modules and expansions. Analyze the effects of the expansions on the existing structure.

1.) Tasks:
   a.) Determine external loads for the new expansion.
   b.) Select appropriate module types to be used in the expansions.
   c.) Define the structural system of the modules, load path and connections.
   d.) Define the connection between existing structures and new additions.
   e.) Model each of the four expansions and perform structural analysis on each new model.
f.) Select a representative sample of structural elements and determine their structural capacity.

g.) Compare the results of the existing building structural analysis to those of the expanded building for each instance.

**Objective #4**

Evaluate the design process for factors that can affect feasibility and summarize those factors.

2.) Tasks:

a.) Potential discussion topics.

- For the case study; discuss advantages, if any, of modular construction in vertical expansion based on research.
- Discuss advantage and disadvantage in comparison to competing site-intensive methods such as cold-formed steel and wood platform framing and structural steel framing.
  - Weight advantages
  - Speed advantages
  - Business operation disruption reductions
- Discuss the upper bound threshold of economy and how that affects selection of construction type in each case.
- Discuss maximum achievable story heights for the case study and describe the limiting factors. Also discuss how module type selection may have changed as story height increased.
  - Discuss if it is possible to increase amount of stories with a modular solution in comparison to site-intensive methods.
  - Make recommendations on particular module types that should be considered for different scale additions.
- Discuss any limiting factors of each vertical expansion example and discuss changes, that could be made to either module selections or module production process that might improve feasibility or increase maximum story height.
– Discuss any new complete module design that was conceived during research that may improve the feasibility of a vertical expansion.
  – Weight reduction
  – Increase strength
– Discuss any modification to the existing types of modules that was conceived during research that may improve the feasibility of a vertical expansion.
– Discuss any modification to the current modular stacking method that was conceived during research that may improve the feasibility of a vertical expansion.
• Discuss any new modular construction methods that were conceived during research that may improve the feasibility of a vertical expansion.

4.8 Process Map

In this section a process map is illustrated that helps to summarize the design process of a vertical expansion. The process map is used as a guide for the case study and will assist with identifying those factors that impact project feasibility. The shape that are relevant to modular construction are shaded for easy identification. Based on the initial evaluation shown in Figure 5.8 the process was divided into four phases:
  1. Pre-planning Phase
  2. Structural Evaluation of the Existing Building Phase
  3. Expansion Conception Phase
  4. Expansion Analysis Phase
Process Map Legend:

- **Start/end**
- **Process**
- **Input/output Data**
- **Off-Page Connector**
- **Decision**
- **Delay**
- **Document**
- **Or**
4.8.1 Pre-Planning Phase

(d) Safety/Access

(a) Motivations

(b) Constructability

(c) Interfases

(See 5.2.1)

(1.1) Conceptual Evaluation

Conceivable?

No

Terminate

Yes

Start Vertical Expansion

(1) Candidacy Evaluation

Initially feasible? Benefits apply?

Yes

No

Consider Site Intensive Methods

F

(1.2) Modular Application Evaluation

F

(a) Transportation Regulations

(b) Geometry

(c) Amount of Planning Time Available

(d) Manufacturer input

(e) Factory Location

(f) Site Access

(g) Factory Break-Even Points

(h) Project Specific Modular Benefits
4.8.2 Structural Evaluation of the Existing Building Phase

(a) Plans/Specifications → (b) Engineering Calculations → (c) Permits

(2) Design Document Assembly

Yes

A

Design Documents Available?

No

C

As Built Drawings

(3) Simplified Building Assessment

(a) Condition of Existing Structure

(b) Field Visits (Verify Plans)

(4) Rigorous Building Assessment

OK to Proceed?

Yes

B

Re-evaluate

No

Terminate
8. Define the Existing Structural System

9. Determine External Loads
   - (a) Load Reduction Opportunities, See 5.3.2.1 (Thorton)

10. Develop Analytical Model
   - (a) Method of Analysis
   - (b) Existing Structural System
   - (c) Detail Level of Analysis

11. Structural Analysis

6. Structural Assessment of Building

7. Evaluate Archaic Material
   - Yes
   - No

8. Define the Existing Structural System
   - No Archaic Materials?
   - Yes

C, J, D

H

(a) Assumptions
(b) Literature Review
(c) Testing
(d) Discussion with Professionals

Expect Strengthening Measures

Terminate
(12) Building Code Review

Regulatory Solution for expansion?

Yes

E

No

D

Terminate

(5) Regulatory Assessment

Variance required? Public disagreement? Rights Purchase?

Yes

(13) Zoning Review

(a) Historic Structure Regulations

(b) Local Amendments

F

B

No

(14) Incentive Program Review

(a) Financial Incentive Programs (Historic renovation grants, energy retrofit, tax credits, etc.)

(b) Special Rating Programs (i.e. LEED)

P
4.8.3 Expansion Conception Phase

(a) Allowable Heights and Construction
(b) Fire-protection Requirements
(c) Access and Egress Requirements

(15) Generate Module Options

(c) MEP Connections

(a) Bearing Requirements

(On Next Page)

O

I

(17) Structural Load Calculation

(19) Preliminary Structural Analysis

Reasonable?

Yes

No

(b) Roof Type

(c) Roof Spans

(d) Connection Points

Terminate

Yes

Consider Site-intensive Methods

E

N

Feasible Options?

Yes

No

M

L

Preliminary Concept

Achievable Break-Even Point for Mfg.

(a) Floor plan geometry

(b) Estimate of addition square footage and height

No

Yes
Can MEP equipment be re-configured? (O) Yes

Terminate

Feasible to expand? (N) No

Terminate

Consider Site-intensive Methods

Yes

(b) Estimated # of Stories

(a) Module Selection

Yes

(20) Expansion Planning

Conflicts with Modular Methods? (Yes/No)

Preliminary Concept

Yes

(f) Building Enclosure requirements

(g) Building Performance requirements

(d) Other Designers

(c) MEP

(b) Architect

(a) Client Needs

Yes

(21) Final Regulatory Review

OK to proceed? (Yes/No)

Terminate

No
4.8.4 Expansion Analysis Phase

Does IBC Section 3403 Apply?
- No
- Yes

Can detailed structural analysis be avoided?
- No
- Yes

Generate Expanded Analytical Model

Develop Method of Modeling
- (a) Load Path
- (b) Connection Details

Available Analytical Tools
- (a) Load Path
- (b) Design Codes

Budget and Schedule
- (b) Budget and Schedule

Project Goals
- (c) Project Goals

Manufacturer Role and Input
- (d) Manufacturer Role and Input

Existing Structural System OK?
- Yes
- No

Modules OK?
- Yes
- No

Structural Analysis
- (24) Structural Analysis
- (a) Load Management
- (b) Design Codes

(See Table 5-2)

Design Strengthening Measures
- (25) Design Strengthening Measures
- (a) Load Path
- (b) Connection Details

Field Visit
- (a) Field Visit

Reduce stories?
- Yes
- No

Use lighter modules?
- Yes
- No

Strengthening measures?
- Yes
- No

Final Design
- (25) Design Strengthening Measures

Terminate
- (22) Generate Expanded Analytical Model

L
Chapter 5

Example Case Studies of Modular Vertical Expansion

In this chapter the findings of the expanded literature review specific to modular vertical expansions is presented. This step was necessary to further define appropriate modular construction methods for the use and to define the considerations important to the designer of a modular vertical expansion. The discussion will begin with traditional vertical expansions that are constructed using site-intensive construction methods.

5.1 Traditional Vertical Expansions

A few examples of traditional vertical expansions are presented in this section. The examples are presented to introduce the concept of vertical expansion and identify some of the challenges presented by the task.

5.1.1 Example 1: 2040 Market St. Philadelphia, PA (Squitiere and Vacca 2013)

The building shown in Figure 5.1 is an example of an adaptive reuse project located in Philadelphia, PA. In this case the developers decided to add eight stories to an existing vacant five-story concrete office building. The use was changed from office to mixed office residential. The steel-framed addition was attached to the existing concrete structural system by means of specially designed HSS transfer truss concealed in the first story of the addition. Structural steel is a popular material for traditional vertical expansion because of its high strength to weight ratio.
5.1.2 Example 2: Tufts University School of Dental Medicine (Hines et al. 2012)

The building shown in Figure 5.2 is an example of a vertical expansion being used to facilitate growth of a business. The original precast concrete exterior can be seen in the bottom right portion of the photo. Five stories were added to the existing ten-story building. The new space was needed for lab and office use. The original building was designed to accommodate future stories but the design was still difficult due to change in wind and seismic regulations over time. Structural investigation was necessary to verify foundation details and sophisticated computational modeling was required to meet the structural requirements.
5.1.3 Example 3: Tacoma Parking Garage Addition (Putnam 2012)

The building shown in Figure 5.3 is another example of an urban adaptive reuse project. A structural deficient parking garage, slated for demolition, with some imagination, was able to be retrofitted for seismic deficiencies and transformed into this modern looking mixed use building with first floor retail, parking and office space in the top stories. The parking garage had originally been designed for additional stories but had fallen into disrepair over the years and still required structural upgrades. In the end, it was decided that the structure could support three stories of steel-framed addition.
5.2 Roof-Top Extensions

Modular construction, along with light-steel framing and panel construction, is used by members of the European Union (EU) to add roof-top extensions to existing buildings, in particular, older masonry and concrete apartment buildings that were constructed between 1950 and 1970 (W/E Consultants 11/07). Figure 5.4 shows an example of a concrete building, in Denmark, extended with cold-formed steel (CFS) modules to create communal space.

Lawson points out (Lawson 2008) that many buildings of this construction type were initially built to house the post-world war II homecoming. A large stockpile of these buildings exists in the EU. Lawson goes on to say that many of the buildings are aging and are currently
due for either renovation or demolition. He also points out that modular construction, when used for renovations to this type of building is generally used to accomplish the following:

- Expand building horizontally or vertically
- Add bathroom, balcony or stair modules
- Upgrade façade to improve aesthetics or building energy performance

### 5.2.1 Renovation of Buildings Using Steel Technologies (ROBUST)

In order to address the problem of the aging buildings research was conducted to determine renovation alternatives. Two notable research projects that, in part, investigated the benefits and challenges of using modular construction for roof-top extensions are briefly reviewed.

ROBUST, one of the research projects, was conducted between 2007-2010 by a consortium of representatives from the European steel industry (“ROBUST - Renovation of Buildings Using Steel Technologies” 2013). The project focus was on the use of Cold-Formed Steel (CFS) construction methods in renovations. CFS Modules were reviewed, as an option, and information regarding their use in renovation is presented in the resulting third work package (WP3), which investigates the use of steel-intensive technologies for building extensions and conversions.

In WP3 roof-top extension design considerations are reviewed. WP3 also contains research regarding the use of portal moment frames to stabilize rooftop extensions. Although this is more relevant to framed light-steel extensions, there is still important information contained in the document pertaining to roof-top extension connections to existing masonry and concrete that could be relevant to modular extension connections as well. Two publications, which were part of WP3, point out some important design issues. The first publication points out constructability, safety and technical issues with general roof-top extensions (Lawson et al. 2013) and the second points out some specific issue with using modular construction for roof-top extensions (Lawson 2008). Below are a few important points identified by the authors of the reports (Points 2m and 4g were not identified by Lawson but are applicable):

1) Motivations for extending buildings
   
   a. Create more space
b. Change of use
c. Energy efficiency improvements
d. Upgrades to new regulations
e. New lift, stairs or balcony required
f. Conservation of historic property
g. Deterioration of existing building

2) Constructability
   a. Will the project be economical?
   b. What are the township and zoning regulations and what are the aesthetics and visual integration requirements?
   c. What are the characteristics of the building? Is it suitable for extension?
   d. What are the technical issues in regards to structure, thermal insulation and fire safety?
   e. Will the extension infringe on the neighbors natural light access?
   f. Are there historic building restrictions?
   g. Will modular construction methods be able to be successfully used?
   h. Can strong points be identified in the existing structure, for module attachment, to ensure stability?
   i. Is the cladding of new structure compatible with that of the existing structure?
   j. Does light-weight façade materials need to be attached by sub-frames to the modular units or to the existing building?
   k. Will the modular units have adequate bearing?
   l. Will the foundation system have adequate excess capacity, if needed?
   m. Can mechanical equipment for the existing building be relocated or worked around?

3) Interfaces that require special attention
   a. New structure/old structure interface
   b. New cladding/old cladding interface
   c. Expansion joints

4) Safety and access issues
a. New egress routes, additional occupant load to existing egress routes
b. Change of fire resistance ratings of building elements such as doors or roof
due to the addition of the roof-top extension
c. Fire load characteristics of the new envelope must reduce the risk of fire
propagation
d. New requirements for fire-fighting access brought on by increase in building
height.
e. Addition of elevator with only one additional level
f. Difficulty of providing necessary assembly fire ratings with modular
construction. Will fire protection requirements influence module selection?
g. Accessibility regulations

5.3 Design Considerations for Modular Vertical Expansions

As mentioned previously, there are many items to consider when approaching a vertical
expansion design. In the next few sections, the items will be divided into two categories; non-
structural design considerations and structural design considerations.

5.3.1 Non-structural considerations associated with modular vertical expansions

The non-structural design considerations generally discussed in this research are
economic considerations such as economy of scale and time savings, regulatory considerations
and the interesting consideration of the advancement of air rights philosophies.

5.3.1.1 Economic considerations

Projects that involve a high level of off-site manufacturing (OSM) are generally more
cost-effective with larger projects. Fixed overhead factory costs and transportation costs are large
in comparison to the overall budget in smaller projects, but conversely, smaller projects can be
economical if they are repeated several times. The economics of OSM in smaller project may be
improved in the future by the integration of numerically controlled machinery and integrated CAD/CAM software (Lawson and Ogden 2008).

Modularization of a project usually involves a break-even point. This is the point (usually measured in square footage or units produced) where it becomes economical to choose modular construction over a competing site-intensive construction method. One New York City modular manufacturer of corner-post structural steel modules estimates their break-even point around 20,000 ft² (O’Hara 2013b). In other words, the manufacturer’s experience shows that in order to achieve economy, the project size should be larger than 20,000 ft². Manufacturers of all wood or CFS modules may have a lower break-even point. Lawson points out that typically corner post bearing modules are more costly to manufacture than an all light-steel product (Lawson et al. 2012).

The primary benefit of using modular construction is time savings. The time savings can provide the benefits of reduced interest charges from outstanding loan balances, early rental income and also less disruption to the existing business (Lawson et al. 2012). When assessing the economics of a modular project, these benefits as well as others are often weighed against the production costs of the modules. Other less tangible benefits can include fewer call backs due to higher quality product and gains from material efficiency.

Local labor rates can affect the economy of a modular project. The Building Industry Association of Philadelphia shows that considerable cost savings can be achieved through modularization in locales where the labor rate is high (Black 2010). Labor rates in Philadelphia, for example, are 39% higher than the national average and construction costs are 18% higher than the national average. The report shows that, due to reductions in labor costs achieved by using off-site construction, a modular single-family row home (one example only) constructed in the city can cost 20% less than an identical home constructed by on-site wood-framed construction.

5.3.1.2 Regulatory considerations

Local zoning code and building code regulations have significant effect on the feasibility and cost-effectiveness of a vertical expansion. According to the ROBUST report (Lawson et al. 2013), the following zoning issues can have influence on the design.

- Local regulations may impose limitations on aesthetics, height, shape of roofs, as well as type of use.
- Height is also connected to the natural lighting issue. The geometrical arrangement of the new building has to preserve natural light for the neighbors.
- The building can be registered as a historical site. In this case, the project has to take into account the constraints on the appearance of the façades and the roof.

In addition to the zoning regulations, the building code has a large influence on a design. The International Building Code (IBC) is the governing document adopted by a large percentage of municipalities across the U.S. The 2009 IBC (International Code Council 2009) has many regulations that could significantly affect the feasibility or heavily influence the choice of building materials for a specific project.

Most vertical expansions would be categorized as an addition per the IBC definition. They would follow the regulations either in IBC Chapter 34 Existing Structures, or the most recently adopted version of the International Existing Buildings Code (IEBC). Chapter 34 requires that any addition causing greater than a 5% stress increase to elements within the gravity load system or 10% increase to elements part of the lateral force resisting system be altered to resist the increased load. Another relevant point in chapter 34 is section 3409, which states that the provisions of the IBC are not mandatory for historic buildings judged by the building official to not constitute a distinct life safety hazard.

Allowable building heights and areas prescribed in chapter 5 affect the choice of materials used for the expansion. Structural steel and CFS modules can be used in non-combustible construction applications, whereas wood framed modules are combustible and are restricted to the requirements for Type V and Type III construction.

Apartments are semi-permanent dwellings and are categorized as an R-2 use group according to section 310. Table 503 allows for a maximum building height of 50’ (max. three stories) with Type 5A construction and 40’ (max. two stories). Type III construction allows for a maximum height of 65’ (max. four stories) and 55’ (max. four stories), respectively, for Type A and B construction with the provision of a two-hour fire rated exterior wall according to Table 601. Section 504.2 allows for an increase of one story and 20’ if an automatic sprinkler system is installed, but at the same time restricts the total increase to 60’ and four stories.

The IBC allows combustible construction to be set on a non-combustible Type 1A podium, maximum one story, with a 3-hour fire resistive barrier between the two (with special restriction on podium occupancy and other prescriptive requirements). In this manner, the
amount of allowable stories and building height for wood construction can be increased by the podium height.

The IBC maximum building height restrictions will typically limit the use of wood-framed modules to vertical expansion no greater than four stories and 60’ unless special provisions are followed or local exceptions pertain. According to Cheung (Cheung 2010), some locales such as Portland, Tacoma and Seattle allow for the construction of 5 and 6 story wood framed buildings (with some restriction). The 2006 Seattle building code has allowed, in the past, for two-story non-combustible podiums beneath five stories of combustible wood framing (Cheung 2010). In general, building height regulations with podium construction consideration can affect material selection for modules and also will determine whether the construction type of the existing building is adequate for expansion.

Fire protection requirements of the IBC should be considered early on in the design process or feasibility analysis. Initially, the addition of even one story of residential occupancy brings a requirement for an automatic sprinkler system in accordance to NFPA 13 or 13R if under four stories (Section 903). In addition, according to Section 905, Class I or III standpipes are required for buildings that have any floor level greater than 30’ above fire department vehicle access height. Lastly, buildings having an occupied floor greater than 75’ are considered high-rise according the IBC and are subject to the requirements of section 403. Increasing the height or changing the construction type of the building can require a higher degree of fire-protection for the whole building. This can greatly affect the feasibility or cost-effectiveness of a vertical expansion.

Separation of occupancies and dwelling units is another component of fire protection that must be considered. Both non-separated and separated occupancy classifications can be considered for an expansion if the occupancies in the expanded building differ. Depending on the particular project, one classification may offer advantages over the other. If the building is evaluated as non-separated, then the whole building is subject to the most restrictive occupancy related to height and area according to Table 503. If the building is considered separated, then a horizontal assembly would be required between the proposed expansion and the existing building according to Table 508.4. Each occupancy will then follow the height and area restrictions pertaining to their individual use groups and the construction type of the building. The exception being that a particular use group cannot be located on a story higher than its allowable amount of stories or height according to Table 503 unless section 509 special provisions is followed and a podium design is constructed as discussed earlier. This may allow for more overall stories to be
constructed but may not be relevant if the developer is considering wood-framed units on the top stories.

In addition to the building separation requirements, the separation of the residential units should be considered. This can be a deciding factor in module selection. Depending on the IBC requirements, a structural steel module, may end up costing less because the fire resistive detailing is easier to implement than other module types. Group R-2 occupancies are required by section 420 to have fire partitions, per section 709, separating the units on a floor, and horizontal assemblies, per section 712, providing the story to story separation.

Accessibility and egress should be given consideration during feasibility analysis. Initially, access must be provided to the new floors, by either stair or elevator. In addition to access, the egress must be provided per chapter 10. Additions must meet the IBC requirements for new construction and therefore must have accessible egress according to section 1007. If the accessible floor is above four stories, then an elevator is automatically required, with some exceptions.

Section 1107 has requirements for accessible dwellings. When residential units are added to the top of a building, it is likely that section 1107 will require that at least the bottom floor of the expansion have Type A or Type B accessible dwelling units unless the building being expanded already has adequate accessible units on lower floors. In this case, some of the general exception in section 1107.7 may apply. In any regards, consideration should be given to the IBC accessibility and egress requirement because it may turn out that adding just one floor of residential units to the existing building can require the installation of an elevator or lift, which can be cost prohibitive to smaller projects.

5.3.1.3 Consideration of Air Rights

The high cost and scarcity of land in dense cities along with the existence of sprawling low-height transportation systems and short buildings in urban areas make vertical development in dense cities a reasonable alternative for developers to consider. Air-rights provide incentive and a framework to develop vertically. Air rights describe the vertical property rights of a landowner. According to Goldschmidt (Goldschmidt 1964) the landowner owns as much of the space above the ground as he can occupy or use in connection with the land. This of course has limitations set by aviation regulations. The first air rights construction project was in New York
over the New York Central Terminal where a street, apartment buildings and an office building were constructed over the railroad track.

Air rights can be transferrable rights in which the land owner can sell the rights to another party to develop the space above their property. The space usually involves a set horizontal division at some agreed upon elevation. New York City has provisions in the zoning code to define air rights within the city. From the definition of development rights, the air rights are associated with the maximum allowable building area set by zoning. If the building is smaller than the maximum allowable, by zoning code, then the unused portion of this amount can be considered developable and transferable (“NYC Zoning - Glossary” 2014). Additional Air rights can also be obtained through lot mergers or transfers of development rights from neighbors.

5.3.2 Structural considerations associated with modular vertical expansions

The potential to vertically expand an existing building is highly dependent on the condition and capacity of the existing building’s structural system. The amount of site-investigation and structural analysis will vary among projects. The structural engineer is the member of the design team that determines the extent of structural investigation required for the specific project and ultimately provide recommendations on the capability of an existing building to accept expansion and maximum achievable heights.

5.3.2.1 General Concerns

The primary objective for the structural engineer employed to design or evaluate the potential of a vertical expansion is to assess the structural capacity of the existing building system and determine how many stories can be added to the existing structure and what, if any, modifications are required to the existing system.

In general, the structural engineer will accomplish this by conducting an investigation and developing an assessment of the condition of the existing structure. The engineer will conduct structural analysis to determine the capacity and reserve capacity of the structural system and use the analysis to make prudent recommendation regarding the maximum amount of stories
that might be added and the appropriate structural systems that might be used for the addition.

Vertical expansion can be grouped in three categories:

- **Category I** - This type of expansion was previously planned for when the existing building was first designed. The original plan set is readily available and foundation and structural systems have been designed to support a designated amount of additional stories. Minimal structural analysis and investigation is necessary in order to proceed with design.

- **Category II** – In this case, the structure has not been originally designed with the intent of future vertical expansion. The original plan-set or as-built drawings are available and reliable. Only minor investigation of existing structural elements is necessary to verify accuracy of drawings and condition of the structure. Structural analysis is required to assess the feasibility of the addition.

- **Category III** – In this case, the structure has not been originally designed with the intent of future vertical expansion. No drawings are available and significant structural investigation and analysis is necessary to assess the condition and capacity of the existing structural system.

The level of difficulty, in evaluating a vertical expansion will often increase, respectively, from a “Category I” to a “Category III” expansion. The availability and trustworthiness of the original design documents can greatly affect the amount of initial structural investigation that is required for analysis, thereby affecting the cost of evaluation. If a building has already been designed for a future vertical expansion, very little investigation and analysis may be required unless building codes have significantly changed between original design and newly proposed addition. If no design documents are available, a full building structural investigation is often necessary, which most likely will be costly and time consuming.

Many of the buildings being considered for vertical expansion are historic and should be reviewed carefully because the building codes, material strengths, occupancy and building construction methods are likely to be different than today’s standards. Thornton (Thornton et al. 1991a) lists the areas below that should be researched when evaluating the feasibility of the vertical expansion of a building:

- Review of as-built drawings, comparison of drawings to field observations and measurements of the existing structure
- Comparison of the analysis and design methods in use at the original time of design to present practice
- Comparison of the requirements of the prevailing codes and standards in effect at the time of the original design to the present requirements
- Comparison of code provisions for live load reduction at the time of the original design to the present requirements
- Review of the changes in functional use within the building

Gustafson suggests (Gustafson 2007) that the building materials of the period be considered. He points out that, in particular, steel design and composition has had many changes over the years and that AISC Iron and Steel Beams, Design Guide 15 and Appendix 5 of the AISC Specifications for Structural Steel Buildings have good reference information for evaluating existing structural steel framing.

Schwinger mentions (Schwinger 2007) that the building should be carefully evaluated for any damage and emphasizes the importance of a thorough building examination. He points out the following items to look for:

- Framing damage
- Corrosion
- Signs of modification to structure or the addition of heavy mechanical equipment that may have been conducted or installed without engineering review
- Unusual deflection
- Foundation settlement
- Cracks in slabs

Structural design methods have matured over recent decades and have led to more efficient use of structural building materials. A better understanding of live loads and lateral loads have led to more accurate and often times smaller design loading over the years. Often older building were designed much more conservatively and have significant structural capacity (Thornton et al. 1991a).

Thornton points out (Thornton et al. 1991b) some ways that the changes in building code and design methodology have made it possible to design a cost-effective vertical expansion for the B. Altman building in New York city. The building was constructed in the early 1900’s and the following changes in methodology and code were taken advantage of:
• Allowable steel stress at the time was 16 ksi, and in 1991 the allowable stress was 0.66fy=0.6*33ksi=22ksi, which gained the designers 35% more steel strength.
• 22 kips per floor structural capacity was gained through changes in occupancy loads.
• The application of live load reduction reduced design live loads for columns and foundations up to 60% in some location.
• Heavy roof cinder was removed and a lighter concrete floor deck was used. This provided extra structural reserve capacity.

In addition to the techniques used for the B. Altman building expansion, structural engineers prefer to specify the lightest possible structural elements in their designs to reduce stress on the existing structural system. An eight-story vertical additions was added to an existing office building in Philadelphia, PA. The structural engineer specified an innovative light-weight composite joist floor system and a bearing steel wall panel assembly to increase the maximum achievable height of the building. (Squitiere and Vacca 2013).

5.3.2.2 Weight of the modules

Modular construction can offer a light-weight alternative to structural steel framing in some settings. The three most common modules used for multi-story modular construction are show in Figure 5.5. Figure 5.5a shows a corner post bearing module or open sided module. Corner post bearing modules are typically constructed with HSS corner and intermediate columns, CFS non-bearing in-fill walls, structural steel perimeter framing and either light steel or concrete floor systems. Loads are transferred primarily through the HSS columns. These modules are typically used in applications where wider spaces (Lawson 2007) are required or situations that require higher strength structural steel components (Lawson and Richards 2010). Corner-post bearing modules are typically stable for no more than 2-stories and require additional bracing from diaphragm action or braced core.

Figures 5.5b and 5.5c show wall bearing modules constructed from all CFS or all wood, respectively. These modules are used for cellular structures up to eight stories. Wall bearing modules are traditionally stand-alone and typically transfer both vertical and horizontal loading through continuous wall bearing and diaphragm action within the wall system (Lawson and Richards 2010).
The weights of each of the modules are shown below in Table 5-1. The weights reflect typical module construction considering only the framing components and gypsum board. Structural steel construction is listed in the table as a point of comparison to site intensive construction methods.

**Table 5-1.** Weights of typical modules used in multi-story modular construction in comparison with structural steel framing (See Appendix D for detailed calculations)

<table>
<thead>
<tr>
<th>Construction Type</th>
<th>Weight (lb/ft²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Corner-Post Bearing</td>
<td>57.5</td>
</tr>
<tr>
<td>CFS Wall Bearing</td>
<td>36.8</td>
</tr>
<tr>
<td>Wood Wall Bearing</td>
<td>37.7</td>
</tr>
<tr>
<td>Structural Steel Framing</td>
<td>61.2</td>
</tr>
</tbody>
</table>

**Figure 5-5.** (a) Corner-post bearing module (image by Lawson and Ogden, 2008), (b) CFS wall bearing module (image by Lawson and Ogden, 2008) (c) Wood wall bearing module (image by Modular Building Institute).

### 5.3.2.3 Transfer mechanisms and structural remediation

Both gravity and lateral loads must be transferred from the proposed expansion to the existing building and the existing structural components strengthened if they do not possess adequate capacity. Often the structural system proposed for a new expansion is not the same as that of the original building. Often large transfer beams or trusses can be required to transfer loads. In the case of the Philadelphia office building renovation, mentioned earlier, the engineer specified custom trusses constructed from HSS steel members to transfer the loads to a concrete column grid spaced at 27’, below the expansion. Large steel tie-downs constructed of plate steel and rod were fastened to the existing columns to resist the large uplift forces imposed by the new expansion.
The university of Plymouth used modular construction to add 28 roof-top bedrooms to an existing four-story steel-framed building (SCI 2001). The extended building is shown in Figure 5.6. The engineer specified a grillage of structural steel to transfer the loading from the proposed expansion to the existing structure.

If the structural system or component within the structural system does not have adequate capacity, then remediation is required to resist the new loads. Schwinger points out (2007) that there are generally two options for the remediation of a floor system. Either new framing could be added to distribute the increased loading or the existing framing could be strengthened. He suggests, that often it is more economical and easier to strengthen the existing construction. Schwinger also discusses that column strength is typically dictated by the slenderness of the column and if added capacity is required, he recommends stiffening the column weak axis with plate steel in an efficient manner. Lastly, he recommends welding new steel to existing steel if possible, because it is easier and requires less precision than field drilling bolt holes.

5.3.2.4 Structural design of modules

Structural design of modules is typically accomplished by the modular manufacturer and reviewed by a third party structural engineer or designed by a structural engineer and reviewed by the manufacturer. The external loads to a modular building are derived in the same manner as any other site-constructed building. Loads can be determined from provisions in ASCE/SEI 7 or prescribed by local building code and zoning regulation.

Chapter 16 of the IBC regulates the structural design criteria for most construction projects in the U.S. Some criteria, such as load combinations, are specified directly in the text but
most are referenced from reliable design codes and sometimes modified partly by language within the IBC. Table 5-2 lists design codes referenced by the 2009 IBC that are applicable to modular design.

Table 5-2. IBC referenced codes applicable to modular design.

<table>
<thead>
<tr>
<th>Structural Material</th>
<th>Referenced Standard</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Structural Steel</strong></td>
<td>AISC 360-05</td>
</tr>
<tr>
<td><strong>Cold-Formed Steel</strong></td>
<td></td>
</tr>
<tr>
<td>Composite Slabs:</td>
<td>ASCE 3</td>
</tr>
<tr>
<td>Non-Composite Floors:</td>
<td>ANSI/SDI-NC1.0</td>
</tr>
<tr>
<td>Framing Members:</td>
<td>AISI 100,200,210,211,212,214-07</td>
</tr>
<tr>
<td>Lateral Design:</td>
<td>AISI 213-07</td>
</tr>
<tr>
<td><strong>Wood</strong></td>
<td></td>
</tr>
<tr>
<td>Framing Members:</td>
<td>AF&amp;PA NDS-05</td>
</tr>
<tr>
<td>Lateral Design:</td>
<td>AF&amp;PA SDPWS-08</td>
</tr>
<tr>
<td><strong>Concrete</strong></td>
<td>ACI 318-08</td>
</tr>
</tbody>
</table>

Modules must be structurally designed for different stages of construction. Smith points out (Smith 2010) that a module must be hoisted onto a truck for shipping, transported to a building site, hoisted off a truck, maneuvered around the site, and finally placed into service. Smith goes on to say that often times dynamic loads placed on the prefabricated element are often the largest that the element will experience in its lifetime and that at times the overdesign of the structural elements for this stage can be a deterrent to using modular construction for a project. The following is a list of items that require design by an architect or structural engineer:

- Structural design of the gravity system
- Structural design of the lateral force resisting system
- Stability of structure under lateral loading
- Connections
- Cladding
- Interface with other modules or building elements
- Robustness in taller buildings
- Fire-safety
- Acoustic Performance
- Durability
• Airtightness and thermal performance

The module is the basic element of a modular building and consists of beams, columns, braces and stressed skin structural elements. Modules are typically categorized as either a wall bearing module in which loads are transferred through the side walls, a corner-post bearing module in which loads are distributed horizontally through edge beams and transmitted vertically through corner or intermediate columns and lastly non-load bearing module commonly called a pod.

The selection of module construction type is generally governed by the required building construction type, economy of design, structural capacity requirements and the availability of modular manufacturers.

Lawson summarizes the limits of each module type and the general load resistance strategy as discussed in the following paragraphs (Lawson and Richards 2010).

Wall bearing modules constructed of CFS or wood framing are used for structures between four and eight stories in height. The compression resistance of the wall elements usually limits the story height. Some variation of a corner-post bearing modules are used for structures of greater height. In this case, the compression resistance of the corner-post governs the design. Square HSS sections are used commonly because of their high resistance to buckling. Lateral loads, such as wind or seismic are resisted by one of three methods:

• Diaphragm action of boards or bracing within walls of the modules; appropriate for four to six story buildings
• Separate braced structure using hot-rolled steel members located in lifts and stair area or in end gables
• Reinforced concrete or steel-plated core; suitable for taller buildings

In taller modular buildings structural integrity is a design consideration. Robustness is provided by ties between the modules (Lawson 2007). The ties help distribute the load to other modules in the event of a module within the system being destroyed. The interconnection and load sharing between modules help prevent a total building collapse.

Module to module connections typically involve a bolted connection and steel plates. The connections can be made at the corners of the modules where structural steel is typically present. Figure 5.7 shows an example of a typical CFS steel module to module connection. The detail can be repeated at the top and bottom and the modules can be connected both vertically and horizontally with the same detail.
5.3.2.5 Discussion on module selection and design approach

Figure 5.8 presents a list of suggested steps to follow when considering a vertical expansion. Keep in mind that a thorough review of building codes and zoning regulations should be conducted along with a detailed evaluation of the existing building in all cases. The construction type of the module should be carefully selected and the construction type of the existing building be defined to verify allowable heights and areas per IBC table 503. The most economical construction type is likely to be different for each project and the use of wood-framed modules will be restricted to lower expansions in most cases.

Wood framing can be an economical choice for vertical expansion if allowed. Wood-framed and CFS modules are comparable in weight and both are lighter than structural steel framing and corner-post bearing module construction. It is possible that the break-even point for wood-framed is lower than that of the corner-post bearing modules, due to the industry familiarity with the material. If this is true, then wood would be the ideal material for smaller vertical expansions involving less square-footage.

All CFS modules could be a logical choice in cases were non-combustible construction is required along with light-weight construction. However it appears that all CFS module construction is not popular in the U.S. Of the manufacturers reviewed in Pennsylvania only corner-post bearing modules were currently being used for multi-story modular construction. It is possible that the strict U.S. building code provisions make it economical to use this type of construction for multi-story projects.
The heavier weight of the corner-post bearing modules and the large break-even point make it questionable whether this type of modular construction would be the more appropriate for vertical expansion. However, the use of some structural steel in an expansion is unavoidable. Structural steel will be needed in most cases were large openings are required in the floor plan and most likely will be used in the transfer mechanism as well.

Non-structural requirement such as the addition of elevators or a sprinkler system is likely to control the feasibility in smaller expansions. These costs can make it difficult to bring economy to a project.

**Figure 5-8.** Suggested approach to feasibility analysis of a modular vertical expansion.
Chapter 6

Examples of Commercially Used Modules Suitable for use in Expansions

6.1 British Module Types

The U.K. expanded the use of multi-story modular construction methods in the mid 1990’s. The construction method gained popularity due to ability to complete building construction projects in less time than traditional construction methods, thereby providing for early return on investment. The methods were used primarily for residential buildings such as hotels, student accommodations, and apartments. Modular construction also found use in the social housing construction sector because of the economy of scale benefits as well as the reduced community disturbance attributes (R. M. Lawson et al. 1999).

The amount of information and case studies available in the UK regarding multi-story modular construction is greater than in the U.S. The U.S. has a well-developed modular construction industry but the market is primarily focused on single-family dwellings and one- or two-story multi-family dwellings. Multi-story modular construction is just beginning to become used in the U.S. Case studies and design information is limited due to the relative immaturity of the industry. Professional organizations, in the UK, such as the Steel Construction Institute have had the opportunity to study buildings constructed by modular methods and have produced publications that include design guidance and case-study. In order to develop an understanding of what module types may be best used for vertical expansions in the U.S. a few of the popular British multi-story module types will be discussed.
6.1.1 Example #1: Murray Grove Project

Figure 6-1. Erection of Yorkon modules at a Murray Grove project in Hackney, North East London (Courtesy Kingspan).

Building Statistics:
Use: Residential; key workers, couples, affordable housing
Modules by: Yorkon
Height: Five Stories

Murray Grove was the first major residential multi-story modular project in the UK and can be seen in Figure 6.1. The building contains one and two bedroom apartments. Modules with one side open were assembled side-by-side to form larger apartment sizes. The one bedroom apartments are composed of two 8m long x 3.2m wide x 3m high modules, while the three bedroom apartments used three of the same modules (Lawson et al. 1999).

Figure 6-2. Modules constructed from two-dimensional light-steel framing have been used in the UK for the construction of multi-story buildings.
Modules were provided by Yorkon Ltd. Yorkon uses a light-steel module similar to the ones represented in Figure 6.2. The module uses a panel system assembled with 355mm deep light steel floor beams and 150mm deep light-steel floor joists. Four 100mm square SHS members are used at the corners of the units and light steel walls studs are used for infill. Modules can be assembled together to form larger spaces by adding an additional interior SHS column along the longitudinal side to create a larger opening in the wall (Lawson et al. 1999). Modules were clad on site with a clip-on terracotta rain screen system on one side and a cedar wood siding on the opposite. Rails for the terracotta clip-on system were installed offsite at the factory.

6.1.2 Example #2: Raines Court, North London

![Raines Court, North London](image)

**Figure 6-3.** Raines Court, North London (Courtesy OpenHouse AB).

**Building Statistics:**
- **Use:** Social Housing
- **Modules by:** Yorkon
- **Height:** Six Stories

This affordable housing project, seen in Figure 6.3, is constructed from 127 light-steel self-supporting corner post bearing modules that were installed in a four-week period. Modules were similar in construction to the Yorkon modules discussed in Example #1. Eight 3.8m modules were used at the base to make up apartments while five stories of multiple module two and three-bedroom apartments were stacked above. Modules ranged from 9.6m to 11.6m in
length and all were 3m tall with 600mm floor/ceiling assembly. Stability of the building was provided by the braced walls of the modules along with supplemental support module groups attachment to the Steel framed access core (SCI 2003c). The units were clad with a ship-lap profiled zinc panel system attached to a factory-installed module sub-frame.

6.1.3 Example #3: Mixed Use Modular Building Constructed for Manchester University

![Podium Design, Manchester University (R. M. Lawson and Ogden 2008).](image)

**Figure 6-4.** Podium Design, Manchester University (R. M. Lawson and Ogden 2008).

Building Statistics:

- **Use:** Mixed; student/social housing with commercial components
- **Modules by:** Rollalong using the Ayrframe system
- **Height:** Seven stories plus below grade parking and ground floor retail

This mixed use building, seen in Figure 6.4, was designed using a podium system. A below grade parking area and grade level retail space was constructed using steel I-beam framing and composite slab/steel deck construction (SCI 2003b). In podium construction, the site-built steel frame is traditionally constructed and braced to resist lateral loading while the group of modules are stacked on top of the podium. The modules typically resist gravity loading and are typically stabilized through a separate bracing system usually located around a core (R. M. Lawson 2007). In this case, however, the stability was provided by the Ayrframe system and transferred to the steel moment frame below.

Seven levels of self-supporting modular housing units were then stacked on top of the podium. The corridor system was incorporated into the modules, which reduced build time and
improved weather resistance. The composite slab system supports loads that do not directly align with the steel framing system. Modules were constructed using the Ayrframe system, shown in Figure 6.5, which is a unique light-steel framing system that uses a grid of C-shaped and furring sections to create a rigid thin walled module (SCI 2003b). The Ayrframe system uses moment-resisting connections and wall studs that are connected by longitudinal runners that provide for stability. No additional bracing is required (Lawson et al. 1999). This building also used a clip-on terracotta rain-screen system as well as aluminum siding in areas.

Figure 6-5. Ayrframe structural system for modular construction (Courtesy Mark Lawson).

6.2 U.S. Multi-Story Modular


Although used to a lesser degree in earlier decades, modular construction was mainstreamed in the U.S. to meet a housing demand created by the post-World War II homecoming. In the years following the war, mobile homes and manufactured housing and modular homes developed a significant presence in the housing market, due to their affordability and quick construction (Smith 2010).

Multi-story modular construction in the U.S. has seen a rise over the last few years due to the increasing urban populations, owner’s awareness of the time-saving benefits of the techniques and advances in key technologies such as BIM and CAD/CAM integration. The U.S. permanent
modular construction market grossed roughly $2.7-$3.0 billion in 2012, which is about 1.2 to 1.3 percent of the overall non-residential building construction market (Modular Building Institute 2013). Although, still a relatively small share of the market, modular construction is beginning to develop presence in appropriate markets such as multi-family, student housing and work-force housing construction.

According to the literature review and factory tours, modular manufacturers in the U.S. tend to favor either wood wall-bearing modules or structural steel modules. Light-framed wood construction is a popular and familiar construction method in the U.S., the material is readily available and the combustible construction type can be economic for both residential and commercial structures up to four- or five-stories in height. For non-combustible modular structures U.S. manufacturers prefer structural steel modules. New York manufacturer CAPSYS suggests that the structural steel modules work better in the U.S. because modules in the U.S. are generally longer and wider than those manufactured by the British manufacturers. Assembly method make structural steel construction cost-competitive with light-framed cold-formed steel modules. Additionally, the robust all steel construction can provides better resistance to wind loading and makes it easier to achieve required IBC fire resistance ratings (O’Hara 2013c).

6.2.1 Wood-framed modules

The construction methods used for wood-framed modules are similar to the platform light-framing construction methods used by contractors to construct sight-built wood-framed structures. Modules are constructed similar to the image shown in Figure 6.6. The floor and roof systems are constructed, as panels, from joists typically ranging between 2x6 and 2x10 members placed at spacing’s of 16” or 24” on center. Wall panels are constructed from 2x4 or 2x6 members. The wall panels are typically set on the floor system and the ceiling set on top of the walls. Gravity loads are transferred through longitudinal walls and lateral loads are transferred through flexible floor diaphragms and shear wall panels. Double rim boards can be provided in the floor system to both stiffen modules for transportation and improve bearing conditions through the floor.
U.S. Manufacturer Britco Structures has used wood-framed modules on many of their projects. Whistler's Athlete Village, shown in Figure 6.7, was constructed using a similar type of wood-framed module. The 70,000 ft² housing facility was constructed in 392 days to house visiting Olympic athletes.

6.2.2 Structural steel modules

Two rolled structural steel framing systems have been identified from literature review and factory tour. Structural steel modules are largely seen constructed as traditional corner-post bearing modules, similar to the British types, or with more slender steel element assembled as trusses or moment frames.
Figure 6.8 illustrates a typical U.S. corner-post bearing module. Manufacturers such as CAPSYS, NRB and Deluxe Homes use modules similar to this to construct their non-combustible structures. A college dormitory constructed by NRB using corner-post bearing modules can be seen in Figure 6.9. The structures generally have floor systems constructed from W-shape or C-shape members and a light-weight reinforced concrete membrane, either with or without decking. Shear studs can be added for composite action with the frame. Although not common in the U.S., cold-formed steel W-shapes or C-shapes could also be used. HSS corner and intermediate columns are used as the primary vertical load transferring members. Cold formed steel studs that range between 3”-6” in width are used as infill walls. The walls do not typically transfer vertical gravity loads, but can be sheathed in gypsum or wood to add lateral resistance. X and K type bracing are also used to resist lateral loads in the wall system. The rigid concrete floor is used as a diaphragm to transfer lateral loads within the floor system. Connections between individual steel members can be either bolted or welded.
The other type of structural steel module that is commonly seen in the U.S. are those similar to the Kullman Framing System (KFS) shown in Figure 6.10. This type of arrangement of structural steel elements provides more flexibility to architects in regards to shape of module and opening positioning. The KFS is constructed from square or rectangular HSS members welded to form rigid moment frames. Both gravity and lateral load resistance are primarily provided by frame action. Diaphragm action and cross-bracing can be utilized as necessary. The KFS system is based on the creation of a Vierendeel truss within the module, which transfers load and requires connection only on the ends. Kullman claims the advantages of this system are compact member sizes, minimum welding, high rigidity, and the fewest possible column and connection points (Garrison and Tweedie 2008). The slender elements can reduce concerns generated by redundant structural elements, such as increased story heights and wall thicknesses.
6.3 Summary of Vertical Expansion Review

This concludes the research necessary to fulfill the 1st objective of the study. The tasks outlined in Section 4.7.1 have been completed and enough information has been assembled to proceed with next step of the research. In Chapters 5 and 6 the following have been identified to be used in design process mapping and the remaining vertical expansion research:

1. Examples of vertical expansions constructed using site-intensive methods and modular methods,
2. Applicable regulations and design codes,
3. Structural and non-structural considerations,
4. Outline of preliminary design process shown in Figure 5.8 which led to the design process map shown in Section 4.8,
5. Modules types that would be appropriate for vertical expansion.

In the next section the case study will be introduced and the design process validation outlined in Section 4.8 will commence.
Chapter 7

Case Study – Harrisburg Midtown Arts Center (HMAC)

A unique building in the heart of downtown Harrisburg, PA was provided for the research to be used as a case study. The case study was preliminarily evaluated early on in the research according to Section 4.5. Upon commencement of the vertical expansion research study the criteria pointed out in the pre-planning phase of the process map illustrated in Section 4.8 was used to evaluate the candidacy of the building. Based on an initial evaluation of the building, it appears that HMAC is a candidate for expansion and it is reasonable to proceed with the study. The building appears suitable to consider modular construction methods, although it is not possible to determine whether they would be the best construction method for this project at this early point in the design process. Some influential points and considerations that would likely need review, in a design situation, are displayed in Table 7.1:

Table 7-1. Candidacy evaluation criteria.

<table>
<thead>
<tr>
<th>Identification Number From Process Map</th>
<th>Evaluation/Assumption</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.1 Conceptual Evaluation:</td>
<td></td>
</tr>
<tr>
<td>(a) Motivations:</td>
<td>1. Add income generating apartments, possibly cladding/energy retrofit.</td>
</tr>
<tr>
<td></td>
<td>2. Located in down-town area, with good view of capitol and water-front, likely to attract renters; Dwellings could be used as primary housing for the owners.</td>
</tr>
<tr>
<td></td>
<td>3. Building plans available and building has provisions for expansion.</td>
</tr>
<tr>
<td></td>
<td>4. Conceptual renovation plans exist to expand building.</td>
</tr>
<tr>
<td>(b) Constructability:</td>
<td>1. Assume zoning OK and project will be economical.</td>
</tr>
<tr>
<td></td>
<td>2. Assume interfaces will not present an excessive amount of problems. It should be fairly easy to match the existing brick cladding.</td>
</tr>
<tr>
<td></td>
<td>3. Trusses appear to be robust and have good connection points. Steel frame and masonry appears to be in good condition.</td>
</tr>
<tr>
<td></td>
<td>4. Building on historic registry; may need consideration.</td>
</tr>
</tbody>
</table>
5. Preserving and renovating historic buildings may be appealing to zoning board.
6. Will have to consider light infringement.
7. Foundation capacity is initially questionable.

(c) Interfaces:
1. Old/new building interface will need consideration of the parapet.

(d) Safety/Access:
1. Elevator installed is beneficial, rear stairshafts are beneficial.
2. Concrete slab on roof good for fire separation purposes or new floor.
3. Need to consider fire-fighting access and addition of automatic sprinklers.

1.2 Modular Application Evaluation:

(a) Transportation
1. Should not be an issue

(b) Geometry
2. Roughly square, truss geometry suitable for grid that would fit typical module sizes.

(c) Planning Time
1. Assume no restrictions.

(d) Mfg. Input
1. Should get one on-board

(e) Factory location
1. At least four within range.

(f) Site Access
1. Questionable. Potential access in the rear, would have to block alley (rear) for crane access. Stock-piling space is limited.

(g) Break-even points
1. Would need to be considered

(h) Modular specific benefits
1. Time savings and reduced community disturbance and business operation would apply.

The case study building is called the Harrisburg Midtown Arts Center and is located at 110 North 3rd Street, Harrisburg, PA (location shown in Figure 7.1).

Figure 7-1. Location map; 110 North 3rd Street Harrisburg, PA (image by Google maps).
The HMAC building is circled in Figure 7.2. The building is located in an urban setting and surrounding buildings are used as either residences or contain both residential uses and light commercial uses such as convenience store or professional offices. The original building, with Mansard roof and brick quoins, as shown fronting 3rd street was constructed of mostly timber-framing and structural brick. The date of construction of the original structure is unknown. According to the design drawings, in 1927 the two-story beige addition was added to the original structure by the Fraternal Order Orioles. The addition has a story below grade housing a swimming pool and previously a bowling alley. The first floor was primarily used, in the past, as a dining area and the 2nd story is voluminous space boasting 18’ ceilings that appeared to be set up as an auditorium, at present.

![Figure 7-2. The Harrisburg Midtown Arts Center (HMAC).](image)

Currently, the building is mostly vacant except for a small bar in the rear of the addition. However, plans are anticipated for a future renovation. As depicted in Figure 7.3a, 7.3b and 7.3d, the conceptual renovation plans include elevator access to the roof area and the placing of two modular residential units on the roof to be used as living quarters for the owners of the establishment. According to the owners, the building was originally designed to accommodate an additional story. As can be seen in Figure 7.3c, the structural steel columns were stubbed past the roof truss, which would appear to be in preparation for a future story column connection.
Figure 7-3. Assortment of HMAC photos; (a) Elevation view showing proposed access to roof (image by OPA architects) (b) Plan view of roof showing proposed residential modular rooftop units (image by OPA architects) (c) Existing steel columns have stub for anticipated future connection of additional story (d) Elevator shaft added to existing building as part of roof access.
The HMAC building is an ideal candidate building to be used for this research project, and meets the criteria set in the beginning of the chapter. In addition, the building appears to have been originally planned for vertical expansion and the owners anticipate this addition option as part of the future renovation plan.

For this research project, the rear addition only, constructed in 1927, is modeled for the case study. The rear addition has a roughly regular shape and an identifiable structural system, which simplifies the analytical model. Simplifications, which will be discussed later, are made to the building and the building modeled in ETABS. In order to meet the objectives set for the project the architects proposed modular unit addition will be disregarded and new conceptual modular arrangements, suitable to the research will be used.

7.1 Description of the Building Structural System

The original addition design drawings were provided by OPA architects, who are the architects that are working with the current owners on renovation plans. The design drawings provide detail on structural member sizes and some connection details, but lack detailed drawings of connections and material specifications. The design drawings used for the research are located in Appendix E. Assumptions are made throughout the research regarding material engineering properties, based on guidance regarding common period construction materials. A site visit was made to document existing conditions and pictures taken to describe the structure.

A structural steel frame with simple connections functions as the building’s primary gravity load transferring system. The floor system consists of open-web joists bearing on steel I-shaped girders. A typical floor framing plan is shown in Figure 7.5a and a picture of the framing shown in Figure 7.4c. A two inch concrete topping poured over draped lath, shown in images 7.4c, d and f, is used as the floor surface. The slab was assumed to act in a non-composite fashion with the floor system. Large rivet connected trusses, constructed from steel angles and plates, span the exterior walls are used to provide open auditorium space below and support roof loads above. The truss detail is shown in Figure 7.4a and some typical connections are shown in Figure 7.4e and f. Orthogonal to the large trusses, spaced at intervals are smaller trusses, which function as diaphragms to help share loads between the larger trusses and provide framing for the barrel vaulted ceiling in the auditorium. The roof plan can be seen in Figure 7.5b, in which one row of the diaphragm trusses is circled.
Figure 7-4. HMAC structural systems (a) Main roof trusses (b) Diaphragm trusses (c) Steel girders framing into columns which are integral with the exterior wall system (d) Structural clay-tile masonry (e) Typical truss to column riveted connection (f) Typical truss rivet connection.
Cross-bracing is used between the bays made up by the intersection of the two trusses to improve stability and provide erection bracing. In the analysis, the diaphragm trusses are assumed to transfer shear loads only and not vertical gravity loads from the roof. Both the girders from the floor system and roof trusses frame into spliced steel columns located within the exterior wall assembly. The columns transfers loads to reinforced concrete pad footings in the basement.

The exterior walls are constructed from 8” structural clay-tile masonry (assumed to be unreinforced), which can be seen in Figure 7.4d and e. The masonry is arranged in running bond using side-construction. The units appear, from their dimensions, to be end-construction units; however, Stang makes no distinction in engineering properties between end units and side units used in side-construction (Stang, Parsons, and Foster 1926). The exterior walls, along with the concrete floors, are used as the primary lateral load resistance system. The concrete floors act as rigid diaphragms transferring load to the clay-tile shear walls and continuing to foundation. The exterior walls bear on concrete strip footings. The wall footings are assumed to transfer the wall self-weight only. The floor and roof load is transferred via column to their respective pad footings.

**Figure 7-5.** (a) Typical steel floor framing design (b) Roof framing plan; Four large trusses clear span from exterior wall on the left to exterior wall on the right. Four rows of diaphragm trusses are orthogonal to the large trusses. Rod cross-bracing is installed in the bays formed by both trusses.
7.2 Engineering Properties of Materials

The majority of the structural materials specification used for the construction of the HMAC building are considered archaic in nature and no longer are used in modern construction. Therefore, the availability of design properties for the materials that were used in construction is limited. Determining the engineering properties for these archaic materials proved to be challenging. The engineering properties were necessary, however, for the structural modeling of the existing building, as well as the analysis of the expanded structure. The three main materials that required definition were the concrete used in the foundation elements and floor slabs, the structural clay tile and the steel used in the framing system.

Concrete was used for the basement foundation wall as well as the pad and strip footings and the floor slabs. To simplify the analysis, the same type of concrete was assumed to be used throughout the building. In an article written for the journal of the American Concrete Institute it is stated that 2,000 PSI concrete was used almost universally during the time period when HMAC was constructed (Kerekes and Reid Jr. 1954); therefore the concrete in the building was defined as an isotropic material with a $f'_c$ value = 2,000 PSI.

The structural clay-tile used in the building was much more difficult to define. Only the surface of the masonry could be seen in areas, so it was impossible to tell whether any reinforcement was present without additional testing, which was out of the scope of the project. For this project, it was assumed that walls do not contain any reinforcement. This was a reasonable assumption since the walls were side-constructed with the cells running horizontally along the length of the wall, laid in a running bond pattern (see Figure 7.4d and e). It would have been difficult to place any reinforcing in this situation.

Ultimately, the engineering properties for the structural clay tile were taken from an article published in the Journal of Structural Engineering. In the article a series of side-constructed clay tile prism tests were conducted to determine the compressive properties of those prisms and compared them with previous tests completed in 1926 (Bennet et al. 1997). In the article Bennet reports an average prism strength ($f'_m$) of 812 PSI, whereas Stang et al. reported (Stang et al. 1926) prism strengths ranging between 232-363 PSI. To be conservative, it was decided to use the upper end of Stang et al. testing for modeling and set $f'_m = 363$ PSI. In the same article, Stang et al. reported the tested modulus of elasticity at $\frac{1}{2} f'_m$ to be 464 KSI and a Poisson’s ratio between 0.14 and 0.16 for prisms tested at zero degrees relative to horizontal. The masonry was defined as an isotropic material having a Poisson’s ratio of 0.15.
The final archaic material to define the material properties for was the structural steel used in the framing. The definition of both the mechanical properties of the steel and the section properties were taken from the AISC Design Guide 15 (Brockenbrough 2002) and the earlier AISC reference (Ferris 1954) that the design guide was chiefly based on.

Design Guide 15 (DG15) lists, in Table 1.1a of the DG, that steel specified for use in buildings between the years of 1924 and 1931 was primarily A9 with tensile strengths = 55 KSI minimum and 65 KSI maximum. The yield of the material is listed as ½ the tensile strength >= 30 KSI. The steel in the model was defined as isotropic with a minimum tensile strength of 60 KSI and a yield stress = 30 KSI. The remainder of the mechanical properties were assumed to be similar to modern steel.

In addition to the steel mechanical properties, the section properties also required definition. Most of the steel shapes specified in the column and beam schedule were no longer included in the current AISC Steel Construction Manual. DG15 and its predecessor were used once again to determine the section properties. The shape in the building were compared with those in the DG and appropriate section properties were selected based on year of production. Table 7-2 lists the information provided on the plan beam or column schedule and the steel shape identified in DG15. Those shapes, whose engineering properties were able to be taken from the AISC Steel Construction Manual 14th edition are not listed in the table.

Table 7-2. Historic steel shape cross-reference table

<table>
<thead>
<tr>
<th>Shape Listed on Schedule</th>
<th>Design Guide 15 Shape</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Column Schedule:</strong></td>
<td></td>
</tr>
<tr>
<td>6”-H26.5#</td>
<td>H 6x26.5</td>
</tr>
<tr>
<td>10”-H72.0#</td>
<td>H 10x72</td>
</tr>
<tr>
<td>12”-H126.5#</td>
<td>H 12x126.5</td>
</tr>
<tr>
<td>8”x61/2”-27.0#</td>
<td>H 8/6.5x27</td>
</tr>
<tr>
<td>10”-H49.5#</td>
<td>H 10x49.5</td>
</tr>
<tr>
<td>8”-H44.0#</td>
<td>H 8x44</td>
</tr>
<tr>
<td>6”-H20.0#</td>
<td>H 6x20</td>
</tr>
<tr>
<td><strong>Beam Schedule:</strong></td>
<td></td>
</tr>
<tr>
<td>8” I-beam 18.4#</td>
<td>S 8x18.4</td>
</tr>
<tr>
<td>9” I-beam 21.8#</td>
<td>S 9x21.8</td>
</tr>
<tr>
<td>12” I-beam 31.8#</td>
<td>S 12x31.8</td>
</tr>
</tbody>
</table>
12” I-beam 40.8# S 12x40.8  
15” I-beam 42.9# S 15x42.9  
15” I-beam 50.0# S 15x50  
15” I-beam 60.8# S 15x60.8  
18” I-beam 54.7# S 18x54.7  
18” I-beam 65.0# S 18x65  
20” B.G. Beam 107.0# G20  
24” I-beam 79.9# S 24x79.9  
28” B.G. Beam 175.0# G28  
30” B.G. Beam 200.0# G30

7.3 2009 IBC Assessment

The information contained in this section is referenced from the ICC 2009 International Building Code (IBC). Not all the IBC requirements will be discussed, but just the important considerations that could affect the feasibility of a vertical expansion. Throughout, the appropriate sections and chapters of the building code will be referenced.

Dimensional information for the HMAC building is provided in Table 7-3. As mentioned previously, the original occupancy, according to the building code, is primarily Assembly (A) throughout. The proposed renovation plans by OPA architects show mixed occupancy including business and mercantile uses; however, the majority of the building would still be classified as an Assembly occupancy (See Appendix E for proposed renovation plans). Considering the proposed uses of the building, it would most likely be classified as A-2, according to section 303, which is reserved for uses such as banquet hall, night club, restaurant or bar. The vertical expansion would be classified as R-2 occupancies, which is reserved for semi-permanent or permanent residents.
Table 7-3. HMAC height and area information

<table>
<thead>
<tr>
<th>Description</th>
<th>Measurement</th>
</tr>
</thead>
<tbody>
<tr>
<td>Building Height:</td>
<td>44.0 ft. (w/parapet)</td>
</tr>
<tr>
<td></td>
<td>40.8 ft. (w/o parapet)</td>
</tr>
<tr>
<td>1st floor building area:</td>
<td>11,046 ft²</td>
</tr>
<tr>
<td>2nd floor building area:</td>
<td>7,928 ft²</td>
</tr>
<tr>
<td>Roof buildable area:</td>
<td>5,272 ft²</td>
</tr>
</tbody>
</table>

7.3.1 Building height and area limitations

Because of the relatively small areas allotted to uses other than Assembly, it would be reasonable to categorize the 1st and 2nd floor as a mixed use, non-separated according to section 508. When a portion of a building is categorized as mixed use, the strictest requirements of Chapter 9 (Fire Protection Systems) of all the uses included in that portion shall apply to the whole portion of the building. Chapter 9 requires an NFPA 13 automatic sprinkler system to be installed throughout the building in A-2 uses have building areas greater than 5,000 ft² or occupancy greater than 100. Both criteria would be exceeded with this project; therefore, an NFPA 13 automatic sprinkler system, amongst other protection measures, would have to be installed throughout the 1st and 2nd floor and any proposed extension (with some exception provided for in the IBC).

It would be reasonable to categorize the existing building as type IIIA construction. Type IIIA requires a 2 hr. fire-resistance rating for the exterior walls, but the interior elements are permitted to be constructed with any material allowed by the building code. The exterior walls are composed of mostly structural clay-tile and brick, which should be able to be improved, if necessary to meet the necessary requirement for 2 hr. fire resistance rating.

Because varying combustible materials are permitted in the construction type, categorizing the building as Type IIIB can offer advantages to the designer. Considering this is a renovation, it is sometime difficult to maintain fire-barrier continuity on interior assemblies not originally designed to provide fire-resistance. Secondly, modules constructed with combustible materials such as wood sheathing or framing would be able to be considered for the expansion.

It is important to note a few points regarding construction type. When R-2 dwellings are constructed with Type IIIB construction type a maximum of four stories and 60 ft. maximum height (limited by section 504.2) is permitted according to Table 503, which should be adequate.
for a two-story expansion (the existing two floors plus a two-floor expansion equals four floors total). Increasing the level of protection to Type I construction would most likely increase cost may make the heavier non-combustible structural steel module types more reasonable to consider for expansion. With Type IIIB, any module type able to be fitted with non-combustible exterior walls could be used for the expansion. Categorizing the building as IIIB would open up the possibility of using cold-formed steel wall-bearing modules for a two story expansion.

If the building is categorized as a mixed occupancy the most limiting building area and heights of Table 503 apply to the non-separated portion of the building, which in this case will be the existing 1st and 2nd floor considering that the proposed extensions could be separated from the existing building by means of a horizontal fire barrier. The most limiting use would be the A-2 occupancy, which limits area per floor to 9,500 ft² and two stories with a maximum height of 55 ft. Considering the automatic sprinkler increases allowed for in sections 504.2 and 506.3, the allowable height and area can be increased to 19,000 ft² per floor and three stories with a maximum height of 75 ft. This is acceptable given the height and areas of the existing building listed in Table 7-2.

In order to consider the use of wood-framed modules for the expansion the building would be required to be classified as Type VA construction. According to section 903 automatic sprinklers are required in all Group R occupancies; therefore, if the automatic sprinkler increases available in section 504.2 are not taken, then the 1-hr. fire resistance ratings for building elements required in Table 601 can be eliminated. In this case, Table 503 allows for 12,000 ft² per floor and three stories with a maximum building height of 50 ft. This would indicate that it would be reasonable to anticipate a maximum of a one-story expansion using wood-framed modules.

Fire separation between the existing building and the proposed building is likely to be necessary. In order to consider a two-story addition, a separation is required between the Assembly use and the proposed vertical expansion. If the separation is not provided, then the maximum number of stories allowed to be constructed is limited by the Assembly use to three stories above grade (considering sprinkler increases allowed per section 504.2). Table 508.4 requires a 1-hr. separation between Assembly uses and Residential uses that have an NFPA 14 automatic sprinkler system installed. For this building the concrete roof slab, with some small improvements, would likely be adequate to serve this purpose.

Figure 7.6 shows the two construction type categorization options discussed in this section. In the Figure a space is provided between the expansion and the existing building to allow for a transfer mechanism. In Figure 7.6a and b the use of wall bearing modules are
considered. In Type VA and Type IIIB construction the limiting building heights are restrictive when considering the use of one-story wood-framed module or two stories of CFS modules. Reduced ceiling heights are to be expected in these instances. If modules constructed according to the requirements for Type I (non-combustible) construction, ceiling heights and general building heights limitations would not likely be restricted by building code requirements, as shown in Figure 7.6c.

Figure 7-6. Estimates of allowable module heights and ceiling heights based on IBC review (a) A single story of wood framed modules, Type VA construction (b) Two stories of wall-bearing CFS modules, Type IIIB construction (c) No reasonable restrictions on height, Type IA or B construction.

7.3.2 Special requirements based on occupancy

IBC Section 420 requires fire partitions separating R-2 dwelling units horizontally and floor/ceiling assemblies constructed as horizontal assemblies that separate the dwelling units vertically. Section 709 requires fire partitions to have a fire-resistance not less than a ½-hour in buildings equipped throughout with an automatic sprinkler system. The fire partitions are subject to the continuity requirements listed in section 709.4. Section 712 requires horizontal assemblies to have a fire-resistance rating not less than ½-hour in dwellings equipped throughout with an automatic sprinkler system. The fire partitions are subject to the continuity requirements listed in section 712.4.
7.3.3 Access and egress

Access and means of egress will need to be provided for the new residential addition. However, for this project it is a difficult task to say with confidence what the means of egress access requirements would be without a complete code assessment. There are two separate renovation types that are occurring simultaneously in which the requirements are different for each. The vertical expansion part is an addition per IBC definition and would be required to meet the egress requirements set forth in Chapter 10; however, since the addition is on top of the building, all egress must traverse through the existing building before reaching discharge. Any renovations occurring in the existing building would be considered under the alteration provisions which would be evaluated under different provisions.

The requirements of Chapter 10 Means of Egress, Chapter 11 Accessibility, Chapter 34 Existing Buildings and the International Existing Building Code (IEBC) should all be consulted to determine the project requirements and craft and access/egress plan. Designing a plan for renovations is sometimes a detailed process, which will not be discussed in great deal in this document, but is normally an iterative process agreed upon by the both the architect and the building code official (BCO). This project, according to the owner, is also listed on the national historic building registry. According to Chapter 34 of the IBC buildings listed on the registry can be exempt from some egress provisions with BCO approval.

7.3.3.1 Accessibility

Chapter 11 provides the required Accessibility features in new buildings; while, Chapter 34 and the IEBC contain modified provisions that pertain to existing buildings. In this project, it will be assumed that no common areas will exist in new addition. It will also be assumed that a one-story addition will add three new dwellings and a two-story addition will add a maximum of eight new dwellings.

It is likely that at least one floor of the residential addition will be required to be accessible. Sections 1107.6 and 1107.7 required that this new addition have at least some Type B units, which are defined in ICC/ANSI A117.1. The amount of requirements depend on whether an elevator is installed on the premise and the vertical extent of the service. In this case, an elevator is installed so any story having elevator access will be required to meet the Type B unit
requirements and be provided with an accessible route according to ICC/ANSI A117.1 and the IBC.

According to section 1104, the accessible route must be provided from the site arrival point to the accessible space, which in this case would be the Type B dwelling units and the site arrival point would most likely be the public streets or sidewalks, according to section 1104. An Accessible entrance is required and the elevator is used to maintain the continuity of the accessible route vertically.

According to section 1105, the building requires that at least 60% of the public entrances be made accessible, but Chapter 34 reduces the requirement to “at least one” and offers alternatives to making the main entrance accessible. This could be important for this building. Notice in Figure 7.7 that the main public entrance is elevated relative to the sidewalk. It may be cost prohibitive to make this entrance accessible without some exemption from full IBC requirements.

Figure 7-7. The main entrance is elevated relative to the sidewalk. To make this entrance accessible would require a ramp or other accessible means of addressing the change in elevation.

In addition to providing an accessible route to the Type B dwelling, accessible egress must be provided for the new addition to allow safe exit from the building in the event of an emergency situation. As mentioned previously, the existing building will be undergoing alteration. When considering alterations, both the IBC and the IEBC do not require accessible egress; however, both require accessible egress for the new addition. This is one point that a reasonable solution should be agreed upon between BCO and developer.
In this building, it is likely that both accessible egress and story access can be provided through the two extendable stair shafts in the rear and the newly installed elevator in the front of the building. Section 1021 requires a minimum of two exits from the 3rd and 4th floor. Section 1007 requires that both of those exits comply with the accessible egress requirements. Both the elevator and the stairways should be able to comply with the requirements. The stairway width and ability to provide an area or refuge should be reviewed for compliance with the requirements of Section 1107.
Chapter 8

Structural Analysis of the Existing Building

ETABS structural analysis software was selected to perform the structural analysis on both the existing building and the expanded building. ETABS was selected primarily because it is a software package designed specifically for the analysis of buildings as well as its similarities with SAP2000 (SAP), which was also needed for some portions of the research. SAP2000 was not used for the development of any portion of the existing building model but was used to develop and test the two different module configurations and base elements used in the vertical expansions. Specifically, the thin-shell elements that were used to construct the floor and wall assemblies of the modules were developed in SAP and the complete wall-bearing module was developed and tested in SAP before it was applied to the main ETABS existing-building model. SAP2000 was preferred to ETABS for this purpose because it is a general structural analysis software package, whereas ETABS is designed specifically for building structural analysis. ETABS contains predefined element and model settings that makes modeling buildings convenient but can make modeling general structures in the package cumbersome.

As mentioned briefly previously, three reasonable modifications were made to the building in order to simplify the analysis and the model. First, the original shape of the building, which included an angled rear wall, was squared off in the model. Secondly, only the addition constructed in 1929, which is outlined in Figure 8.1 was modeled. The original building was not modeled since no conceptual expansion is planned for that portion and for all intents and purposes the front and rear portion of the building are structurally independent. Finally to expedite the modeling, the majority of the wall openings were not included.

Removing the openings from the model can potentially increases the stiffness of the walls if the opening sizes are significant, which in turn reduces story drift and out-of-plane wall deflections. Additionally, lower stresses in the wall will be reported by ETAB than actually exists due to the extra wall material present where openings should be. To compensate for these potential inaccuracies the following steps are taken:

- Exclude out-of-plane wall deflections from results. The out-of-plane wall deflections should not be significantly impacted by any vertical extension of this building given
the structural steel primary gravity system is mostly independent of the exterior wall system.

- Report any drift comparisons between the existing structure and the expanded structure relatively in terms of percentages rather than actual values.
- Perform hand calculations in order to determine shear wall capacity.

![Outline of the portion of the building modeled for structural analysis.](image)

**Figure 8-1.** Outline of the portion of the building modeled for structural analysis.

### 8.1 External Loading Assessment

External loading was derived from ASCE/SEI 7-10 and the 2009 IBC. The detailed load calculations can be found in Appendix F. All of the potential gravity loads were considered and only the two wind load cases, which occur perpendicular to the exterior walls were considered for lateral loading. As per the Chapter 4 assumptions, the building was assumed to be in a low seismic region primarily dominated by wind loads. Seismic loads were not considered in this project. The following load cases listed in Table 8-1 were considered for the analysis:
Table 8-1. Load cases used for structural analysis of the HMAC expansions

<table>
<thead>
<tr>
<th>Load Case</th>
<th>Abbreviation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dead Load:</td>
<td>D</td>
</tr>
<tr>
<td>Floor Live Load:</td>
<td>L</td>
</tr>
<tr>
<td>Soil Lateral Pressure:</td>
<td>H</td>
</tr>
<tr>
<td>Roof Wind Pressure:</td>
<td>W_r</td>
</tr>
<tr>
<td>Winds From North:</td>
<td>W_n</td>
</tr>
<tr>
<td>Winds From South:</td>
<td>W_s</td>
</tr>
<tr>
<td>Winds From East:</td>
<td>W_e</td>
</tr>
<tr>
<td>Winds From West:</td>
<td>W_w</td>
</tr>
</tbody>
</table>

8.2 Load Management Plan

ASD, LRFD and service load combinations are investigated in the research, which are derived from ASCE/SEI 7-10 load combinations. The LRFD load combinations are used to determine the design loading for selected steel structural elements, whereas the ASD load combinations are used to calculate the design loads for the masonry shear walls and to check soil bearing capacity. The service loadings are used to check member deflections and story drift. The load combinations used for this research are listed in Appendix F. In accordance to the exceptions listed in ASCE/SEI 7-10 sections 2.3.2 and 2.4.2, a factor of 0.5 was applied on floor live load and a factor of 1.0 was applied to soil lateral loads.

Only full intensity live load is considered for this research. Pattern loading, unbalanced snow loading and drift will not be considered. Live load reduction is not applicable per 2009 IBC sections 1607.9.1.4 and 1607.9.1.5 for Assembly occupancies. Live load reduction could be applicable to the roof live load, but is neglected in this case because the snow load is controlling.

8.3 ETABS Model

Modeling the existing building proved to be challenging. Not only were there archaic materials to incorporate, but there were additional challenges posed by the steel framing integral with the exterior walls and the large roof trusses. Figure 8.2 shows an overview of the existing
building model. The model was constructed with frame, shell and link elements available in the software.

![Figure 8-2. Axonometric view of the existing building ETABS model.](image)

8.3.1 Boundary Conditions

The building was bound at the basement level by ground as well as the street level by street and sidewalk. Translations are restrained at column bases in all directions to simulate anchor bolt connection shown in Figure 8-3, as well as to ensure that adequate restraint is present in the model. Walls are assigned a translational base restraint in the Z direction (See figure 8-2 for reference direction) at mesh intersections to simulate uniform wall footing bearing and an orthogonal restraint at the same location to prevent out-of-plane base movement. Walls are free to deflect along their length and rotate freely. Additionally, walls are restrained against horizontal (X, Y direction depending on orientation of wall) translation at the 1st level to simulate the sidewalk or street at that level.
8.3.2 Exterior Walls

The exterior walls are modeled using the thin-shell elements available in the software. The shell elements used for the walls are 4-noded elements that are formulated to exhibit both membrane and plate bending behavior (Computers and Structures, Inc (CSI) 2013). The exterior walls function as the main wind force resisting system. The clay masonry windward walls transmit out-of-plane wind loading to shear walls through rigid diaphragms and the embedded steel columns. The building interior gravity loads are carried primarily by the steel frame. Exterior walls serve as envelope and provide shear resistance only. As far as gravity loads are concerned, the exterior clay-masonry walls carry their self-weight only and are support by concrete strip-footing. See Figure 8-4 for partial exterior wall section.

Two wall types are used in the model of the existing building. An 8” wall is used to model the clay-tile portion of the wall. The weight of the brick is incorporated into the unit weight of the wall but the thickness of the brick is neglected to avoid any increase in stiffness of the wall. Only the clay-tile is used in calculating the stiffness of the wall section. The second wall is the concrete foundation wall. The foundation wall is modeled as 17” thick as per plan. Both wall sections are defined using the materials discussed in Chapter 7.

Figure 8-3. Typical column footing.
Both walls are meshed with approximately 2'x2' squares, with finer mesh sizes found near columns and irregular geometry or connection points with other elements such as links or
frame elements. Walls are meshed in order to both simulate even load distribution at the base and to provide for connection to the embedded steel column, which is necessary in order to brace the out of plane movement of the columns. Additionally, by meshing the walls vertically, lateral loads can be varied along the height. Both soil and wind load can be applied as surface pressure and varied in increments equal to the mesh size. Wind and soil load application and magnitude can be seen in Appendices F and G.

8.3.3 Steel Framing

The steel framing serves as the primary gravity load resisting system. There are four main gravity frames in the building that transmit all loads from the roof and floor to independent pad footings. All steel framing is modeled as having simple connections and as is separated from both the floor and wall system such that non-composite action can be simulated. The floor separation will be discussed in the next section. The columns as mentioned previously and as can be seen in Figure 8.3 are embedded in the exterior wall system. The columns, in the model, must be able to transmit all the gravity loads to footings without transferring load to the adjacent walls. The walls must act to brace the columns out-of the wall plane, but at the same time should not act compositely with the columns. The columns do not have sufficient connection with or embedment to act compositely with the masonry wall system.

Figure 8-5. The girder shown in the picture is connected to an exterior column which is embedded in the exterior wall assembly.
To simulate this complicated behavior, two different types of link elements, available through the software are defined. The first is labeled as a column link. This link has all degrees of freedom fixed except for U2, which is the translational direction parallel to the longitudinal axis of the column. This direction is released to ensure no shear transfer occurs between the column and the wall. Figure 8.6 shows how the two different links were used at column–wall intersection.

![Diagram of column, wall, and links](image)

**Figure 8-6.** Three links are used at each wall mesh intersection / column intersection to simulate the appropriate column behavior. A column link connects the column to the wall segment on each side while the fixed link connects the wall segments together and maintains continuity of the wall.

This accomplishes two objectives. 1st it ensures that all loading from the floor and roof is transmitted to the column footing solely by the column without distribution to the surrounding walls and secondly it de-bonds the column from the masonry shear wall such that the column does not add stiffness to the wall when the wall is resisting in-plane loading. In order to maintain the integrity of the wall during in-plane loading, a second link is used to connect the wall segments adjacent the embedded column. The second link is simply a rigid link fixed in all directions.

The interior framing was defined as the custom sections listed in Table 7-1. A typical frame is shown in Figure 8.7. The large trusses supporting the roof are modeled as simple connections to ensure truss behavior. The trusses are loaded at the joints only. The steel open web joists are omitted from the model and added as dead load to the floor system. This greatly simplified the framing model.
8.3.4 Floors and Roof Slab

Floors are modeled in a few different ways to simulate different behavior. Particular attention should be given to modeling non-composite floors to ensure proper behavior. In this case, the floor and roof slabs act as rigid diaphragms, which transmit lateral load to exterior shear walls. In order to simulate this behavior and de-bond the diaphragm from interior locations, the diaphragm is attached to only exterior joints rather than assigning it to the entire floor systems. By assigning the diaphragm in this manner the joints on the interior are not subject to lateral loading.

A 2” membrane with special one-way action is used to model the first and second floor slab system. The slabs are drawn such that no interior joints align with slab edge joints, so the two do not act compositely. Auto cookie cutting mesh along with the one-slab action is used to properly distribute loading to the steel framing. The slab dead load is automatically calculated by the software, whereas other loads as calculated in Appendix F are assigned to the slab. A rigid diaphragm is assigned to the exterior slab joints as mentioned previously and can be seen in Figure 8.8. Each floor has its own diaphragm designated.
The roof slab is modeled similarly, but with one difference. The self-weight of the slab is set to zero, such that no load got distributed along the length on the truss top chord members. No additional loads are assigned to the slab. All loads are converted to point loads and applied at the joints of the trusses. A rigid diaphragm is assigned to the exterior slab joints in the same manner as the floors below.

8.4 Structural Analysis Results

Frame “3”, the exterior wall along line “A” and the intersecting column at A-3, which are all shown in Figure 8.8, were chosen as a basis for comparison of the load effects between the existing structure and the extended structure. Structural analysis was performed on the complete three–dimensional model but only the load effects for these structural systems are discussed. The detailed load calculations can be seen in Appendix H. Load effects in other structural systems were similar due to the regular shape on the building and uniformity of loading. Table 8-2 shows load effects from critical frame and wall systems, as well as the reactions at the base of column A-3 and controlling story shears in wall “A”. The member labels are shown in Figure 8.9.
Figure 8-9. Frame 3 truss (half-truss) member labels for cross reference in Table 8-2.

Table 8-2. Result of structural analysis of existing building for representative members of Frame 3

<table>
<thead>
<tr>
<th>Structural System</th>
<th>Magnitude</th>
<th>Limits/Capacity</th>
<th>Members</th>
<th>Load Case</th>
</tr>
</thead>
<tbody>
<tr>
<td>Frame 3</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Max. SL Deflection: S</td>
<td>0.15&quot;</td>
<td>L/360 ( \frac{1}{3} ) = 2.58&quot;</td>
<td>Truss</td>
<td>Roof Member SL Deflection</td>
</tr>
<tr>
<td>Max. TL Deflection: D+S</td>
<td>0.54&quot;</td>
<td>L/240 ( \frac{1}{3} ) = 3.88&quot;</td>
<td>Truss</td>
<td>Roof Member Total Deflection</td>
</tr>
<tr>
<td>Roof Truss Frame 3</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Max. Comp. Force</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Top Chord</td>
<td>56K</td>
<td></td>
<td>B426</td>
<td>Max Envelope</td>
</tr>
<tr>
<td>Bottom Chord</td>
<td>10K</td>
<td></td>
<td>B413</td>
<td>Max Envelope</td>
</tr>
<tr>
<td>Vertical Strut</td>
<td>138K</td>
<td></td>
<td>C134</td>
<td>Max Envelope</td>
</tr>
<tr>
<td>Max. Tens. Force</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Top Chord</td>
<td>94k</td>
<td></td>
<td>B413</td>
<td>Max Envelope</td>
</tr>
<tr>
<td>Bottom Chord</td>
<td>371K</td>
<td></td>
<td>B427</td>
<td>Max Envelope</td>
</tr>
<tr>
<td>Diagonal Tie</td>
<td>201K</td>
<td></td>
<td>D11</td>
<td>Max Envelope</td>
</tr>
<tr>
<td>Column A3</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( L_{\text{box b-1}} (\text{ft.}) )</td>
<td>12</td>
<td></td>
<td></td>
<td>LC2</td>
</tr>
<tr>
<td>( Mu (K*\text{ft.}) )</td>
<td>26.1</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( Pu (K) )</td>
<td>242.0</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( L_{\text{box 1-2}} (\text{ft.}) )</td>
<td>13</td>
<td></td>
<td></td>
<td>LC3A</td>
</tr>
<tr>
<td>( Mu (K*\text{ft.}) )</td>
<td>9.0</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( Pu (K) )</td>
<td>187.0</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( L_{\text{box 2-BOT}} (\text{ft.}) )</td>
<td>18</td>
<td></td>
<td></td>
<td>LC3A</td>
</tr>
<tr>
<td>( Mu (K*\text{ft.}) )</td>
<td>21.2</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( Pu (K) )</td>
<td>156.0</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Base Reaction (K)</td>
<td>182.1 (+Z)</td>
<td></td>
<td></td>
<td>ASD4</td>
</tr>
</tbody>
</table>
1. Plaster ceilings attached.

8.4.1 Truss Analysis, Frame 3

The controlling members, with design loading as described in Table 8-2, were further analyzed to determine if they would have adequate capacity to resist the existing loads under current building code as well as to be used as a reference for later comparison with the expanded structure. The results of the capacity checks are reported in Table 8-3. The checked members all have adequate capacity to resist the design loads.

Table 8-3. Capacities of controlling truss members

<table>
<thead>
<tr>
<th>Member Check</th>
<th>Capacity (K)</th>
<th>Loading</th>
</tr>
</thead>
<tbody>
<tr>
<td>B426 Compressive Check</td>
<td>1009</td>
<td>56</td>
</tr>
<tr>
<td>B427 Tensile Check</td>
<td>921</td>
<td>371</td>
</tr>
<tr>
<td>D11 Tensile Check</td>
<td>457</td>
<td>201</td>
</tr>
<tr>
<td>C134 Compressive Check</td>
<td>412</td>
<td>138</td>
</tr>
</tbody>
</table>

8.4.2 Column A-3 Check

Column A-3 was analyzed as a beam column with the clay masonry acting as brace for the weak axis (in the plane of the wall) but not for the strong axis (out of the plane of the wall). The floor diaphragm provided bracing at the floor levels only for the strong axis. The column was determined to be at 59% capacity under existing loading.

8.4.3 Pad Footing Supporting A-3

Only the bearing pressure was checked for the pad footing. Adequate detail was not available to check the strength of the concrete. The bearing pressure due to existing building loads was calculated at 2,746 pounds per square foot (PSF). It is likely that the bearing capacity is adequate for this level of loading, but as the pressure is increase with the extensions this may become a limiting criteria.
8.4.4 Shear Wall, Line A

The purpose of this preliminary shear wall analysis was to evaluate whether more detailed structural analysis and wall investigation would be necessary to judge if the existing exterior walls would support the increase lateral loading created by vertical expansion. The analysis results are used to comment on how the feasibility of expansion is affected by the shear wall capacity. The need for detailed structural analysis and wall investigation can increase engineering costs and if the capacity is questionable, costly wall strengthening measures could be warranted.

It was assumed for the research that the exterior walls provide stability and act as the lateral force resisting system. It is difficult to say with certainty that this was the original structural design intent, but the assumption seems reasonable considering the column embedment in the exterior walls and lack of any other obvious lateral force resisting system. The exterior wall along line “A” was assumed to be the critical shear wall line in the building because of the large amount of fenestration. The base of the walls is at the grade line where restraint is provided, as discussed previously, by the street or sidewalk.

Only a visual inspection was made during the initial HMAC site visit. The cell grouting percentage and amount of wall reinforcing (or lack thereof) could not be determined, therefore a true capacity cannot be assigned to the wall until this information is ascertained. For the analysis it was assumed that the walls did not contain any reinforcing or grout.

Three analyses, which were conducted according to the requirements of ASCE 7-10 and ACI 530-11, were performed to make commentary on capacity of the masonry. The detailed calculations can be seen in Appendix H. The 1st approach was the most conservative and considered only pier segments 1, 2 and 3, shown in Figure 8-10, to contribute to the overall capacity of the shear wall. In the second approach segments 1a, 1b, 1c and 4 were included in the capacity calculations as well as the perpendicular corner returns for segments 1 and 4. For the last approach the full wall length was considered neglecting all openings. The purpose of this approach was to determine an upper-bound capacity.
Figure 8-10. HMAC South elevation.

The result pointed toward the need for more detailed structural analysis and wall investigation. The shear capacity of the walls was shown to be adequate, but the moment capacity of some of the segments was not adequate as un-reinforced, un-grouted sections. The most conservative approach (method 1 of the calculations shown in Appendix H) indicated the need for some level of grouting to be present in segment 1 and reinforcing in segments 2 and 3 in order to contribute to shear capacity. The second approach (method 2) indicated the need for some level of grouting to be present in segments 1, 2 and 3, and reinforcing to be present in segment 4. The third upper-bound approach indicated that if no openings were considered an un-reinforced, un-grouted wall would have adequate shear and moment capacity.

A vertical expansion would require additional structural analysis to determine true shear capacity of the wall. Visually, the wall appears to be performing adequately under current loading, but the preliminary analysis shows that in order to pass current masonry and building code checks calculations would have to be enhanced to include contributions of the spandrel sections and potentially the brick veneer.

Some of the initial assumptions could be investigated to either reduce loading or increase capacity. The shear transfer mechanism between the embedded column and the wall could be revisited to determine if the shear load application assumed in the calculations is reasonable. The wind loads could be potentially reduced to give more consideration to the shelter provided by the
urban locations. Also, some of the mechanical properties of the masonry, such as the mortar strength can be reviewed. Lastly, it is clear that additional site investigation would have to be performed to determine if grout and reinforcement is present in the exterior walls.

Although the shear wall check did not result in an actual capacity, it does provide us a qualitative platform from which a comparison can be drawn after the expanded model is tested. The calculations show that it is questionable whether the wall along line A is adequate for the existing loads under current building code. This would indicate that an expansion of any magnitude has the potential to overstress the clay masonry walls. It would be important to consider this at the initial planning stages, because it may not be cost-effective to strengthen the existing walls for the expansion.
Chapter 9

Vertical Expansion of HMAC

In the previous chapter the characteristics and the analytical model of the existing building were defined. Additionally, structural analysis was performed on the building and the capacities of a representative selection of structural components were calculated so that those elements can be evaluated for adequacy when the structural effects of the expansions are discussed in this section. In this chapter, the remaining steps outlined in Figure 5.8 will be discussed as they pertain to this project. The remaining steps in the process are:

1. Define the expansion concept
2. Discuss module selection
3. Model the expansion
4. Evaluate feasibility

The expansion was modeled considering two different module types, for both a one and two-story expansion. For this particular building type it was not practical to model greater than two stories, due to the footing pressure of 2,746 PSF generated by existing loadings. Considering a likely allowable bearing pressure of between 3,000 PSF and 4,500 PSF, it was assumed that bearing pressure would likely be a limiting factor in determining the number of stories that could be placed on the existing building and an expansion over two stories could potentially trigger extensive foundation alterations.

The two types of modules selected for research were a cold-formed steel framed wall bearing module and a structural steel corner post bearing module. From literature review these two types of modules appeared to be the most widely used in the U.S multi-story modular construction industry.

Gypsum sheathed walls were chosen to transfer lateral loads for both module types. Gypsum sheathing is commonly used to resist lateral forces in industry and is a reasonable choice given the limited number of stories of the extension. The gypsum sheathing not only provides lateral resistance, but acts as part of the required fire-resistive assemblies that would be required between dwellings and on exterior walls in a residential addition constructed in the U.S.
A grillage of structural steel that can be seen later in Figure 9.6 and similar to that specified for the project shown in Figure 5.6, was used as a base for the modular expansion and to transfer the loads from the proposed addition to the existing structural system. The grillage was configured slightly different in each case to host the two distinctly different module types.

9.1 Expansion Concept

The expansion would be intended to provide one to two stories of leasable apartments. The idea is that the owner can install the expansion without much disruption of his day-to-day business operations. The modules can be installed quickly and the owner can see quicker returns on his investment through early occupancy lease income. Initially a reasonable floor plan, fitting to the project requirements and realistic in terms of modular construction had to be conceived. Figure 9-1 illustrates the floor plan used for the research.

![Floor Plan Images]

**Figure 9-1.** The conceived architectural floor plans for the proposed modular two story residential expansion.

The two-story expansion would reasonably be able to accommodate nine apartments ranging in size from 830 square feet to 1,430 square feet. As discussed in Chapter 7, an elevator
had been recently installed in the building, which would provide an accessible route and potentially accessible egress to the rooftop occupants. The modules were not extended to the rear of the building so that a common area could be provided for the occupants. The rear of the building is anticipated as space where occupants can enjoy the views of the city via a glass covered atrium. The potential also exist for this area to be used for other interests such as rooftop gardening.

There are three other reasons for cutting the modular expansion short of the back wall. As can be seen in Figure 7-5b, the last roof truss ends at the location under the rear of the proposed modular expansion. At this point, the roof framing changes from trusses to open-web joist framing. It was questionable whether open web joists would be able to support additional module weight without additional strengthening measures, therefore the decision was made to stop the expansion at the last truss rather than trying to maximize this portion of the roof with modular dwelling area. Additionally, the modules, as shown, are approximately 60’ in length and 15.75’ in width. This is about the maximum reasonable size for road transport (Smith 2010). It is feasible to transport a 15.75’ x 60’ single module, but any larger would most likely require multiple modules. Lastly in the existing building, two staircases enclosed in a masonry shaft are located on the opposing rear corners of the building with steps already constructed to the bottom of the roof (in preparation for a future expansion). These staircases will likely be needed as an egress route for the expansion and could be easily adaptable to the floor plan as shown in figure 9-1.

A two-story extension would require 14 Modules; nine for the 1st story of the expansion and an additional five for the second story. Assuming an installation rate of five to ten modules per day (SCI 2001) the bulk of the expansion could be installed in 2-3 days. Two of the 14 Modules are designed to house the mechanical equipment for the existing building.

One of the issues with adding on to an existing building is the loss of valuable roof space for mechanical equipment. In commercial buildings it is likely that the roof would contain such mechanical equipment as roof-top HVAC equipment (i.e., Heat Pumps, chiller equipment, and ventilation), communications equipment or fire-fighting equipment. When considering the expansion initially, a plan must be made to relocate these types of equipment to feasible locations. For this project it was assumed that there would be some mechanical equipment in need of relocation. The maintenance modules were conceived to allow space for this mechanical equipment. The notion is that the equipment can be installed or reconfigured at the ground level and then craned into place with final connections being made on the roof. The maintenance
modules are constructed with a structural steel frame and concrete floor, but have additional CFS in-fill framing to add cavity insulation to help acoustically isolate the mechanical equipment from living space. The implementation of the maintenance modules can be seen in Figure 9-2.

![Figure 9-2. (a) 1st story of the conceived expansion is shown with the maintenance module installed (b) Zoomed in view of an individual maintenance module shown the structural steel framing with CFS in-fill walls for acoustic buffering.](image)

The total expansion square footage would be approximately 8,580 square feet which would likely be below the minimum square footage required for a manufacturer of structural steel corner-post bearing modules to consider the project. Although corner-post bearing module are tried for this research project, considering scale of economy with current modular production methods, it is more likely that a manufacturer of wood or CFS light-framed modules would be better able to be deliver this type of project. Mark Line Industries, located in Ephrata, PA would likely be a good match for this type of project. They are primarily manufacturers of wood-framed modules, but have the ability to use CFS if required by the project specification. They produce many one-off type lower square footage structures, so their experience in this arena would be valuable.

### 9.2 Architectural Challenges

Architecturally speaking, many of the challenges have already been discussed previously. Application of the building code and its challenges have been discussed in Chapter 5 as well as in Chapter 7. Additionally, preliminary floor planning and the potential need to relocate mechanical equipment was discussed in the previous section. There are, however, a few straggling item to discuss.
An important topic yet to be discussed is mating the expansion with the existing building, such that it is aesthetically pleasing and competent as a building enclosure. This is a challenge best suited for the Architect of record for the project. The roof and wall system will have to be selected to blend into the existing architecture and neighborhood as well as meet the performance requirement set forth in the IBC, IECC, local amendments and project specifications. Additionally, it is possible that material selection would have to gain zoning approval since the building is recorded in the historic registry. It will be important to detail this joint such that it is attractive as well as watertight.

For the analysis, it was assumed that the expansion would be clad with insulated aluminum panels and have a built-up roof installed after module installation. Aluminum panels were chosen because of their relatively light weight (approximately 3 PSF), which reduces gravity loads on existing structural elements and improves feasibility of the expansion. The built-up roof is a reasonable selection based on the typical flat roofs associated with the modules. In addition to the cladding and the roofing, some attention will have to be given to the detailing of the joint between the new expansion and the existing building.

A decision will have to be made by the architectural team about how to best address the existing parapet. Three options were considered for the parapet. The options and consideration for each are listed below:

1. Keep the parapet in place and install the expansion on top of the existing masonry or use existing column stubs to attach a perimeter beam support system. Considerations:
   a. Wasted space under expansion and increased overall building height
   b. Extra structure required on the interior to make up the height difference created by the existing parapet.
   c. The details of actually attaching to the existing structure securely from top of the parapet may be difficult.

2. Demolish parapet and existing column stubs. Install expansion such that the edge of new cladding aligns with the edge of the existing façade. Considerations:
   a. The effort to remove existing column stubs and parapet may be labor intensive.

3. Leave the parapet in place and install expansion on the interior of the parapet wall. Considerations:
a. The offset between the existing walls and the new expansion walls may not be attractive.

b. Could potentially increase the difficulty of the water-proofing detail at this joint.

c. May interfere with 1st floor expansion window sill heights.

The wall-bearing modular expansion was modeled according to option 2 and the structural steel corner-post bearing modular expansion was modeled according to option 3. Both of these options appeared reasonable and by selecting different approaches for each expansion type a broader sense of potential issues could be developed.

In concluding the discussion about the façade it should be noted that in addition to selecting façade components considering the expansion only an overlay of roof or wall cladding could be considered for the existing building as well. In the event that this is a possibility, the wall cladding for both the existing structure and the proposed structure can be selected to match or at least improve consistency between the two. It is important to mention, as touched on in Chapter 5 and discussed much more thoroughly in the European Union’s ROBUST research project publication (Lawson et al. 2013) this modular expansion, if appropriate, can become part of a larger façade renovation or energy retrofit that could address other deficiencies with the existing building. It is possible that the feasibility and economy of this proposed modular expansion can improve considering a more comprehensive renovation approach.

9.3 Module Selection

In this section, the Modules that will be used in the testing are defined and discussed. If this were a design project then the project requirements and goals would be reviewed at this point to determine which type of module would be appropriate. In terms of economy of scale, as mentioned in the previous section, it would be likely that a light-framed module would make sense.

From conversations with representatives from CAPSYS, Deluxe Homes and NRB occurring on factory tours, the minimum threshold for the structural steel modules can be anywhere from 20,000 square feet to 50,000 square feet. The HMAC conceptual expansion, which falls under 10,000 square feet is not in this range. A Manufacturer of light-framed modules such as Mark Line Industries would likely prove to be a more appropriate modular
vendor for this project. Another advantage of the light-framed modules would be their weight. Table 5-1 shows that with similar interior finishes the light-framed modules are approximately 36% lighter than their structural steel corner-post bearing counterparts. According to the literature reviewed for this project, this would likely be advantageous for a vertical expansion project.

Although the light-framed modules have some advantages, Figure 7-6 shows that it may be difficult to meet building code regulations with a combustible construction. It may not be feasible to use a wood framed module for more than a single-story expansion due to building height restrictions. It is possible that a CFS module could be used in a two-story expansion but there would likely be some ceiling height restrictions unless the CFS modules were constructed as non-combustible. For these reasons, the potential still exists that a structural steel corner-post bearing type module may be required. This is the main reason both types were considered for the conceptual expansion. Other module types could be considered, such as those based on Vierendeel trusses, but the wall bearing and the corner-post bearing modules are the most common in the U.S.; therefore, this research will consider these types only for expansion.

9.3.1 Architectural and Structural Performance Requirements

During the conception stage of the expansion planning, it is important to completely define both the structural and architectural requirements of the project such that the right module can be selected and to determine whether modular construction is the appropriate delivery method for the project.

Architecturally speaking the floor plan/shape of building must be compatible with a modular layout. For this project any module type would be compatible with the proposed layout. The building plan is roughly square and the floor plan is arranged for cellular apartments. If the floor plan involved a more open layout then a structural steel module may be the better choice or it is possible that a hybrid construction would make more sense.

The intended building enclosure must be compatible with the module framing. All of the following items can have an effect on module selection and at some point decisions will have to be made regarding:

- Cladding System
  - Attachment details
b. Will the system contain modular elements or will it be installed for the most part by on-site contractors?

- Flooring
  a. Is the material selection compatible with a concrete floor?
  b. Does concrete provide the required comfort level for the occupants?

- Fire protection
  a. Can the proper level of fire protection be achieved with the module construction type?
  b. If so, is it cost affective or does it make more sense to choose a different module construction type?

- Hygrothermal concerns
  a. Insulation levels and type
  b. Water management detailing
  c. Fenestration
    i. Are large glazing panels required on exterior walls?

- Mechanical equipment
  a. Branch conduit runs
  b. Location of new HVAC equipment

The selection of module must be compatible with the existing structural system. It has to be possible and economical to transfer the loads from the proposed modular expansion to the existing structural system through a reasonable transfer mechanism. If the systems are too drastically different, then any economy gained through modular construction can be lost through oversized transfer structure requirements.

For this project either module type proposed should be compatible with the existing structural system. A simple grid of W-shape members will be able to be used for the transfer structure, which is likely to be economical. The existing roof is defined by roof trusses spaced at compatible dimensions to typical modules and the existing rigid concrete diaphragm should be compatible with the flexible module floor diaphragms. The capacity of two diaphragms at the transfer level will be difficult to predict, however. The many connections required at this joint and the merging of both a rigid and a flexible diaphragm into a questionably composite assembly will likely make it difficult to predict, with any certainty, the actual performance of this load resisting element. A conservative estimate of performance would most likely be required for this
element in an actual design situation. The performance of this combined assembly is beyond the scope of this research, but it is important to identify this area as a variable.

9.3.2 Module Loads

The module gravity loads can be seen in Appendix F and the lateral wind load calculations are located in Appendix G. In addition to the information in the Appendices, dead loads for the wall and floor system is discussed in the next section. The floor live-load is based on a residential occupancy with some exception for the maintenance modules.

9.3.3 Definition of Modules

Another difficult step in this modeling process was to define the appropriate modules, such that reliable test results could be gathered when the modules where assembled for the addition and analysis. No guidance or benchmarks were discovered in the literature review to provide suggestions on how to best accomplish this – therefore, the schema was developed based on existing material test data and typical module framing details that were discovered in the review. The detailed calculations for the module definition is located in Appendix I.

SAP2000 was used to run initial tests on the modules to determine the best way model the two different types. SAP2000 was used because it is a general purpose structural analysis software package that provided modeling flexibility, whereas ETABS has a focus on building structures. A benefit of using SAP2000 over another general purpose structural analysis software package was that both SAP2000 and ETABS were created by the same company and have similar elements and settings. Models created in SAP2000 were re-created in ETABS and the performances of those elements were generally the same.

The module models were developed to meet the main modeling goals of the research. The goal of the research is not to understand the internal loadings that are developed within the module, but to better understand the loads placed on the existing structure from the expansion. The following performance was expected from the module models:
1. Module should be able to be modeled relatively quickly and easily inserted into the main model. The connections should be easily implemented, such that the modeling of the expansion is not cumbersome.

2. These models are not intended to be used for a module design. A high level of accuracy was not required, because the goal is to get a general sense of how the loads are transferred to the existing structures such that a selection of structural element from the existing building can be reviewed to determine the effects of the expansion.

3. Accurate representation of the distribution of loads imposed on the modules to the connections/reactions of the grillage/existing structural system was necessary to discuss any group action that may occur and to ensure that the loads are distributed to the appropriate existing members.

4. In-plane stiffness is necessary for walls and floor system in order to gauge the expected amount of drift/deflection and to also more accurately predict how loads were to be distributed to the steel grillage.

5. It was not necessary to define the out-of-plane stiffness of the wall and floor systems as long as the loads distributed to the connecting elements correctly.

9.3.4 Define Module Structural Elements and Modeling Strategy

As noted in the previous section, the goal of the module model is to accurately transmit the external loads from the proposed structure to the existing structure; therefore, the modeling strategy was based on this premise. The module was primarily divided into sub-assemblies rather than individual structural elements. The main two sub-assemblies used for the construction of the modules were the floor and wall systems. These were the first two structural systems of assemblies to be defined in the research. The detailed calculations for the information contained in this section can be found in Appendix I.

Both weight and stiffness values were calculated for the wall and floor assemblies. Two sets of test data was used as a basis for comparison. The results of an ASTM E72 wall racking test conducted by Intertek (5/8" Dense-Glass Gold 2003) on a wall coupon sheathed with Georgia Pacific 5/8” Dense Glass Gold were used to develop the stiffness information necessary for analytical modeling of the wall assemblies and results from tests conducted by HUD (NAHB Research Center 1999) on cold-formed steel joist floor assemblies sheathed with OSB were used
to develop the stiffness information necessary for analytical modeling of the wall-bearing module floor system.

Initially, it was attempted to model the modules in their entirety with individual beam elements for the framing components and thin shells used for the gypsum wall sheathing and floor sheathing (Figure 9-3). Modules modeled in this manner were overly complicated for the intended use. The many elements necessary to construct one module slowed computation time down and the modules became too rigid without the use of multi-dimensional springs attaching the wall and floor sheathing to the structural framing.

The interaction of wall and floor sheathing with the structural framing (studs and joists) is complicated and difficult to model properly. Attachment of sheathing to stud or joist is accomplished by nails or screws and sometimes involves the use of an adhesive to strengthen the connection. Deformation of sheathed light-framed assemblies occurs due to rotation caused by nail slip, lateral translation due to bearing of fastener failure and direct shear deflection of the sheathing. The summation of these three actions produces the total lateral translation of the sheathed wall assembly (Vieira and Shafer 2012). In order to model the three translation actions accurately, Vieira and Shafer used springs to represent the stiffness in each of these directions.

![Figure 9-3. In the Initial prototype of a wall bearing module modeled in SAP2000 an attempt was made to model all framing members and diaphragm elements.](image)

As mentioned before, this was overly complicated for the goals of the project and it was decided to pursue an alternative approach, in which the walls and floor system would be represented by thin-shell area elements alone. A 5/8” thick shell wall and a 23/32” thick shell floor system was created and modeled in SAP2000. Empirical formulas and test data from
Intertek and HUD were used to calculate the weights and Modulus of Elasticity for the stud wall assembly and joist floor system. The Modulus of Elasticity (E) developed for the wall system and floor system are shown in Table 9-1.

Wall Type A was used for both a load bearing assembly in the wall-bearing modules and as infill for the corner-bearing modules. In both cases Wall A was used for the lateral resistance system. The test-coupon for the Intertek tests on Wall A was constructed from wood framing. It was assumed that the stiffness of a similar wall constructed from cold-formed steel elements would behave comparably to the wood-constructed coupon.

Table 9-1. Wall and Floor Assembly Properties

<table>
<thead>
<tr>
<th>Assembly</th>
<th>Description of tested assembly</th>
<th>E (KSI)</th>
<th>Wt. (PSF)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wall A</td>
<td>2x4 studs @ 16” O.C., 5/8” Dense-Glass Gold gypsum sheathing nailed @ 4” on the perimeter and 8” in the field (1 ¾” Galvanized Roofing Nails)</td>
<td>31,300</td>
<td>7</td>
</tr>
<tr>
<td>Floor A</td>
<td>2x8x43 mil CFS joists @ 24” O.C.; 23/32 OSB Sheathing attached with #8 Tek screws spaced @ 6” at the perimeter and @ joints and 12” in the field.; panels staggered.</td>
<td>60,000</td>
<td>10</td>
</tr>
</tbody>
</table>

9.3.5 Module Construction

Two separate module types were developed for the expansion testing. The first was based off of the wall-bearing modules reviewed. The structural systems and weights of wood-framed wall-bearing modules and cold-formed steel wall-bearing modules are similar so only one module was modeled to simulate the behavior of both types. The second module was a corner-post bearing module, which was also modeled based on those reviewed as part of the literature review.
9.3.5.1 Wall Bearing Modules

Figure 9.5 shows the wall bearing module that was conceived and modeled for this project. The wall bearing modules were the most difficult to model due to the complex behavior of light-framed assemblies. Because of this complex behavior and the need to use the floor and wall system elements created in this definition with the corner-post bearing model this module type was modeled first. Before attempting to model the expansion on the main model, a 12’x60’x10’ high two-story test module was constructed to determine if the thin-shells used to represent the wall and floor assembly would behave as anticipated. The details of the testing and loading can be seen in Appendix I. The model was constructed in ETABS to see if the results of the SAP trial would carry over properly to the ETABS platform.

Figure 9-4. ETABS wall bearing trial model shown in its deflected form.

Figure 9-4 shows the model studied, in its deformed shape due to wind load. A section of the W-shape transfer grid was included in the model to determine if the loads would transfer to the grid as anticipated. 5/8” thin-shells were used to model the walls. The walls were meshed manually at 16” increments to represent the stud spacing and the connection of the wall plates to the module below and ultimately to the transfer structure. The meshing lines can be seen in Figure 9-4. This is necessary to represent the load transfer of the individual studs to their bearing, assuming that the bottom track/plate is fastened to the transfer beam below at an approximate 16” spacing. The floors were represented by the 23/32” thick shell discussed in the previous section. Once again the shells were meshed at 16” increment to represent the ceiling/floor joist spacing as well as the stacking of ceiling joist on the wall studs. Unlike the walls the ceiling/floor joist were meshed in both directions. During module trials in SAP2000, it was observed that this enabled
the load to be better distributed from the floor system to the side walls and also qualitatively appeared to simulate the load sharing that could be expected due to the common diaphragm/joist attachments.

Figure 9-5. Wall-bearing module conceived for the research.

Semi-rigid diaphragms were designated for the roof and the floor below, but not for the base level. Using a diaphragm at this level caused errors and incorrect load distribution in the analysis. The purpose of defining a diaphragm was essentially to define the extents of the diaphragm, such that the lateral loads distributed properly. Designating the diaphragm as semi-rigid allowed the model to engage the stiffness of the shell definition in calculation.
The ceiling and the floor system was modeled as one assembly. Figure 9-6 shows a typical stacking arrangement for wall-bearing modules. As can be seen in the Figure, the floor joist transfer the gravity floor loads to the longitudinal walls system, which in turn distributes to the module wall below. The lateral loads are distributed through the floor sheathing to the perimeter rim joists and then to the shear wall panel below. The loads are collected and distributed to the shear panels below that are parallel to the loading as can be seen in Figure 9-4. The ceiling joists typically only supports gypsum board and mechanical items such as lighting installed in the ceiling cavity space.

Figure 9-6. Typical stacking arrangement of wall bearing modules, courtesy Steel Construction Institute, SCI (Lawson 2007).

The description of the module trial and the results are shown in Appendix I. In general, the stacked modules performed as expected with a few manageable exceptions listed below:

1. In the overturning analysis, the middle two reactions were opposite what would be expected. The middle reactions on the windward side were small in magnitude but compressive, whereas those on the leeward side were tensile.
2. Base shear instead of being evenly distributed to both the windward and leeward side was distributed more to the windward side. The link connections to the transfer frame is suspected of being less rigid than the direct node connection to the base restraints.
3. Slightly more load was distributed to those nodes that had direct connection to a restraint versus those that had an indirect connection via a link. This again might suggest that the link connections are less rigid than those directly connected to a restraint.
9.3.5.2 Corner-Post Bearing Module

Figure 9-7 shows a typical U.S. corner-post bearing module. The model for this type of module was developed largely from this design, as well as, other similar types that were discovered during literature review. A trial model was not necessary for this configuration because adequate information was gathered from the wall-bearing model research. The model for corner-post bearing modules was constructed from standard steel shapes and concrete floors. The engineering properties for these materials were specified from the ETABS library. The gypsum-sheathed wall assemblies developed for the wall-bearing modules are used as infill and were
assumed to be rigidly attached to the corner posts. They serve, along with the light-weight concrete floor diaphragm, as the lateral resistance system for the modules.

9.4 Modeling the Expansions

Once the modules were developed, the next step in the process was to proceed with the modeling of the expansion onto the existing building. The trussed roof of the existing building was well suited for the planned steel grillage. In this upcoming section, the transfer beam arrangement, the module connections and the details of the expansion model construction will be discussed for both the wall-bearing and the corner-post bearing expansions.

9.4.1 Wall-Bearing Module Expansion

The wall bearing modules were assembled from the wall and floor shells defined in the previous section. The modules were constructed and connected to the transfer structure in a similar manner to the test model except that the widths of the expansion modules were approximately 15.5’ versus the 12’ width used in the trial model. Rigid Links were used to connect adjacent modules to their common transfer grid beam and to each other where connections were designated. The purpose of these links are to simulate the gap between adjacent modules. Typically modules are only connected to each other at corners and at a few interior locations, if the module is long. As can be seen in Figure 9-5 adjacent modules typically have an air space between them to allow tolerance for misalignments and provide improved acoustics. A 6” air space was simulated between the modules in the model.

9.4.1.1 Transfer Grid

The transfer grid for the wall-bearing modules was modeled considering parapet option 2, as discussed in the Architectural Challenges section. Steel beams were specified in the North-South direction to align with the existing roof trusses and cross-members were detailed between these beams aligning with the diaphragm trusses below. Beams in both directions serve a
structural purpose because wall-bearing modules required continuous support beneath their exterior walls. The existing truss members are load bearing elements; however, the diaphragm trusses are not. Therefore, the beams above the diaphragm trusses were required to transfer loads to the main North-South grid beams.

As mentioned above, parapet Option 2 was considered for this grid arrangement. The arrangement is depicted in the screenshots of the model shown in Figure 9-8. This option involves demolishing the existing parapet and aligning the grid flush (or recessed slightly for cladding if required) with the exterior wall. By doing this, the cladding can align with or overlap the existing cladding; potentially simplifying the required waterproofing details and providing for a cleaner joint. The grid perimeter beams would attach to the column stubs located within the parapet. Those beams aligned with the main trusses will also require a connection of some type to this stub. The stubs will likely have to be trimmed flush with the top of the transfer grid to provide for a flat surface to set the modules on top of.

![Figure 9-8](image)

Figure 9-8. (a) Aerial view of the wall-bearing module transfer grid (b) Close up of the South-Western corner. It can be seen in the picture that the grid aligns with the existing wall.

W12x53 members were used for all grid members and the grid members were connected to each other and to the existing building with simple connections. The beam size was initially selected because of its relatively shallow span-to-depth ratio. Minimizing the transfer structures depth ultimately allows for maximizing the module ceiling heights. The W12x53 was a convenient selection for the grid because not only did the members have adequate stiffness and strength for the design, but they also had large flanges which was beneficial considering the bearing/connection requirements of the modules above (The beam must be able to support and transfer loads from two adjacent modules as well as facilitate the 6” air-space between) and the
composite connections to the existing roof truss below. Figure 9-9 illustrates the many connections that require consideration at this level. Notice the following about the figure:

1. HSS column: In order to make solid connections to the grid and other modules HSS column members are integral to the wall-bearing modules at corners and at intermediate locations for longer modules.

2. Steel angle clips: It is anticipated that the connection of the module to the grid will be made with these clips and bolts. The connections provide moment resistance from right-to-left, but do not resist moment in and out of the page.

3. Bottom plate: It is anticipated that the bottom plate will be fastened with either pneumatic fasteners or screws at approximately 16” O.C. intervals to the grid.

4. W12x53 grid beam: Wide flanges allow for connection of the module to the grid and the grid to the existing truss. Notice the simple shear-tab connection occurring at intersecting beams.

5. Connection to the existing truss: Bolted connection to the truss below provides for load transfer as well as strengthening of the existing truss through composite action.

6. Grid beams bears directly on existing concrete roof slab: The existing cinder-fill and roofing membrane has been removed to provide a level bearing surface for the grid. Removing the 60 PSF cinder fill also substantially reduced the roof weight.

7. Concealed space beneath the module: There is space available between the top of the existing concrete roof slab and the bottom of the wall-bearing module floor joists for use. This space can be used for insulation or mechanical equipment.
9.4.1.2 The Expansion

The wall-bearing modules will be placed directly on top of the grid and fastened, as discussed, at corners and intermediate locations. For this project, the intermediate locations will be at the grid intersections. Note that for the model the shear wall panels, developed in the module selection section, will be assumed to be rigidly attached to the HSS member embedded within the walls, therefore the overturning vertical loads, developed as a result of the module subjected to lateral loading will be transferred to the grid at these intersections. The test module developed in the module definition section was used to assemble the expansion components. Figure 9-10a shows the first story of the expansion. Module walls set on the grid beam. The grid beam was meshed to match the walls above; approximately 16” O.C. and the grid beam was attached to the main trusses via rigid links at truss nodes. The links were necessary to simulate the composite action provided by a bolted connection to the truss without engaging the roof slab.
9.4.1.3 Connections

There are several connections to consider in this modular expansion, especially at the grid level. In order to simulate the performance of the modules properly and to avoid an overly rigid building response, a series of links were used to simulate the assumed connections. The important connections in this model are the grid-to-existing building connection, the module-to-grid connection, the module-to-module connections and the connections occurring at the roof level.

The grid to existing building connection was the first to be planned for. This is the connection that provided for the load transfer from the proposed expansion to the existing building. As mentioned before, the grid beams were designed to act compositely with the existing roof trusses. Solid connection to the existing building is necessary to ensure proper load transfer. The bolted connection to the top-chord of the truss appeared to be the best way to accomplish this. The connections as shown in Figure 9-9 would be easy enough to implement in construction and the gains from the composite action would likely be worth the effort.

Composite action is a material efficient way to both strengthen and stiffen the existing roof structure. Because the truss is the main load-bearing element and not the grid beam, without engaging any composite action the beam would be under-utilized as a load bearing element and function mainly as a spacer and a connection point. By forcing the beam to act compositely the beams axial capacity is engaged. The composite action also increases the overall moment of inertia of the truss, which in turn, reduces deflections. This is important because plaster ceilings

Figure 9-10. (a) View of the 1st story expansion modeled in ETABS (b) View of the two-story expansion.
are attached to the bottom of the roof trusses. The existing total deflection calculated at mid-span, according to Table 8-2, was approximately 0.54”. Any more deflection, above and beyond this magnitude, could induce cracking in the plaster ceiling.

Connection of the grid to the existing roof slab was considered, but eliminated, due to the variability in the existing slab construction. As can be seen in Figure 9-11, the slab was formed by a draped mesh material installed over the steel joists. The slab varied between 2”-3” in thickness and sufficient connection to the joists or the trusses could not be identified that would indicate that composite action could occur between the roof framing members and the concrete slab.

![Figure 9-11. HMAC roof construction. Notice the draped mesh that the original contractors had used to form the roof slab.](image)

The roof slab did appear competent to act as a rigid diaphragm but not as connection point for the expansions. The main connection points would have to occur at the bolted connections to the top-chord of the trusses and the column stubs located in the parapet. In the model, the roof slab diaphragm extents did not include any connection to the interior roof framing nodes. The extents connected to the perimeter of the exterior walls only. This simulated the non-composite action that was anticipated to occur at the roof level.

Because composite action was intended to occur between the grid beam and the trusses, but not with slab, a mechanism was required to simulate this action. A link element was selected to accomplish this. Figure 9-12 shows the connection between the grid and the roof truss. The truss was loaded at the joint only. The links were not specified as completely rigid, but were assigned a high stiffness value for all degrees of freedom. This was necessary in order for the model to report the axial force developed in the links due to the expansion loads, which allowed
data to be gathered that would show the distribution of the vertical forces on the existing roof trusses at these points.

Figure 9-12. The connection between the grid beam and the top-chord of the existing truss was made with a stiff but not completely rigid link.

The base module-to-grid was the next connection considered. Figure 9-13 shows how adjacent modules were connected to the transfer grid. The one-way slabs were connected to the meshed steel grids at the nodes only by a rigid link. It was acceptable to specify this link as rigid because its purpose was to transfer load only. No internal force or stress information was needed from these elements. Connection of the modules to the grid in this fashion simulated that a connection existed only at the base and top levels of the modules. An advantage of this technique was that the effects of the gap between the two modules was able to be simulated.

Figure 9-13. Base module floor system showing connections via rigid links to the steel beam grid.

Module-to-module connections are to be discussed next. The module-to-module connection was required to transfer both horizontal and vertical loads between the modules. Figure 9-14b shows a typical module-to-module horizontal connection for this type of
construction. The connection is accomplished through the use of a steel plate bolted through another plate, which is welded to the corner angles of the module. This connection occurs at all corners and intermediate locations at the top and bottom (if other than base module) of the module. In the model, this connection was simulated by the same rigid link used for the base floor connections.

Figure 9-14a and 9-10b shows how the modules were stacked for both of the two-story expansions. By stacking the modules in the manner shown in the Figures, ETABS interprets this joint as a rigid connection. This is an appropriate simulation for the given construction. In referencing Figure 9-9, it can be seen that in actual construction the walls would be stacked directly on top of each other vertically and the floor and the ceiling assemblies essentially are one unit from a structural performance perspective. The main vertical connections occur at the steel angle corners and intermediate locations, where a steel plate similar to the horizontal plate will be used. The plate can be fastened to the module above with either bolts or a weld, whichever is convenient for the contractor. It was assumed that the stacked longitudinal walls would have some connection between them in the form of screws, straps or other similar light fasteners.

Figure 9-14. (a) Figure shows how the modules were vertically connected in the ETABS model (b) In practice modules can be connected, horizontally, using a steel plate and bolting to the corner angles installed within the walls.

The last connection to be discussed is located at the roof level. Figure 9-15 shows a typical roof connection in the model. The construction being simulated in the ETABS model is shown in Figure 9-5. It was assumed for this project that the roof would be built up using metal decking, rigid insulation and a waterproof membrane layer(s). For simplicity, the roof was
modeled using the same shell type defined for the floor system. It is likely that the engineering properties of the proposed built-up-roof system are different than those of the light-framed floor system, but for the purposes of this study, the differences were acceptable considering the objectives of the research.

It is likely, with this roof construction type, that connections would occur at interior locations in the form of screw fasteners or plug/puddle welds and that these connection provide for some lateral load transfer from the diaphragm to the structure below at these points. For this reason, the extents of the diaphragm, modeled in ETABS, include some interior points. Additionally as shown in Figure 9-15, unlike the floor shells, the roof shell spans the adjacent module gap and has nodal connections where the walls are meshed (at 16” O.C. intervals). Also noticeable in Figure 9-15 is the module-to-module link connection that occurs at the top of the module.

![Figure 9-15. Typical plan view of the module-to-module connection at the roof level.](image)

### 9.4.1.4 Load Path

The loads from the proposed expansion must be ultimately transferred to the existing structure. The manner in which this occurs is important such that the connections to the structure can be defined properly. For this example, the loads are transferred to the main structure through grid beam connections to the existing column stubs and main trusses. It is important to preface this discussion with a note on the lateral loads. This building is located in a region dominated by wind loads; therefore, only the wind loads were considered for the research. If seismic loads were considered for this project the connections could be subject to more demanding code requirements.
Figure 9-16. (a) Gravity load from modules distribution to grid and structure below (b) Lateral wind load distribution.
Both gravity loads and wind loads are required to be transferred to the existing structure. The transfer will occur predominantly at the roof level where gravity loads will be transferred via the grid to the existing steel frame and the lateral loads will be transferred through diaphragm action to the exterior clay masonry walls.

The gravity loads are transferred primarily through bearing (See Figure 9-16). The gravity loads will initially be transferred through the one-way floor and roof assemblies to the longitudinal walls of the modules. The loads will then be transferred as a line-load to the steel cross-members above the diaphragm trusses and then as point loads to the main trusses at the grid intersections. The trusses will deflect, engaging the anticipated composite grid beam connection (over the main trusses) and the loads will be transferred to foundation through the steel columns located in the exterior wall. As a note regarding the composite connection, the concrete roof slab was not modeled such that it will add to the stiffness of the composite truss-grid beam, but in reality once the truss deflects the slab will be engaged by bearing of the through bolts and will likely increase the stiffness of the composite assembly.

The wind loads are transferred through the module framing to the grid, into the original concrete roof diaphragm, to the existing building steel framing and finally to the masonry shear walls to foundation. Initially, the wind will load the exterior walls of the expansion, the walls will deflect out-of-plane and transfer the load to the flexible diaphragms in the form of a line load at the floor level above and below the wall. The diaphragms will then transfer the loads, based on the stiffness properties of the floor/roof assembly to the longitudinal walls of the modules as a line-load. The gypsum sheathed shear wall segments, assumed to span between the structural steel angles located within the walls, will transfer both an overturning force-couple and a base-shear to the story below at the intersection where a relatively rigid connection from grid to the structural steel angle embedded in module walls is present. The shear will be transferred to the rigid concrete diaphragm through bolt bearing of the through bolts making up the composite grid/truss. The rigid diaphragm transfers loading to the supporting steel frame into the masonry shear walls through the embedded columns. Additional gravity load result from lateral overturning forces in the shear panels. The summation of the force-couples will be applied at the grid level intersection points and will be transferred through bearing to the main roof trusses.
9.4.2 Corner-Post Bearing Module Expansion

The corner-post bearing modular expansion had similarities and a few differences when compared to the wall-bearing module expansion. The primary difference is that the corner-post bearing modules have a structural steel frame that transfers loads, primarily as point loads, at the corners of the modules. In order to simulate this the grid approach had to be slightly modified as well as the method of module-to-module connection in the vertical direction.

The ETABS model was based off of module construction discovered during literature review as well as that observed during factory visits. Figure 9-7 shows a typical example of a U.S. corner-post bearing module. The floor system for the module can be seen in Figure 9-17. Beam elements were used to model the frame, which consisted of W10 x 15 beams on the perimeter and simply connected W8 x 10 joists spaced at approximately 4’-0” on center spanning between. Figure 9-17 shows the thin-shell with special one-way action that was used to represent the 3” light-weight floor slab (100 pounds per cubic foot). The floor slab was modeled to act compositely with the joists because in practice shear studs or metal decking would typically be used to strengthen the thin light-weight floor system. The floor slab spans perpendicular to the joists and is meshed at intervals to simulate the composite connection.

![Figure 9-17](image)

**Figure 9-17.** The concrete floor slab was meshed to simulate composite connection with the joists. The one-way slab spanned between the joists.

The remainder of the structural system was modeled as follows. HSS 4 x 4 x 3/8 columns were specified as the corner posts. The thin shell walls that were defined for the wall-bearing modules were used as infill between the corner posts. The walls were assumed to have rigid connection to the corner-posts and an angle was attached to the top of the walls, which was
meshed to match the wall meshing (approximately 16” on center) below and spanned between the corner posts along the perimeter of the module to add stability to the top of the wall. Keep in mind that the corner-post modules transfer loads at the corners not along the top of the wall. Figure 9-7 shows a ¼” air gap between the top of the wall and the bottom of the upper module frame, which reinforces that load is not transferred via the longitudinal walls. The thin shells created for the floor system of the wall-bearing modules was used for the roof diaphragm, as in the wall-bearing module expansion.

9.4.2.1 Transfer Grid

The transfer grid arrangement for the corner-post bearing modular expansion was modified, from that used for the wall-bearing modules, to accommodate the different structure of this module type and to demonstrate an alternative approach to the parapet solution demonstrated in the example of the wall-bearing modules.

Two main changes were implemented in the model. First, as described previously, the corner-post bearing modules transfer loads through their integral structural steel frame to corner bearing locations. This creates the opportunity to remove those transfer beams perpendicular to the main roof trusses (above diaphragm trusses) that were necessary to provide continuous support for the longitudinal walls of the wall-bearing modules in the previous example. Instead, the beams within the floor system of the module are used to span between the trusses. This eliminates those beams above the diaphragm from the transfer grid, which reduces cost and weight of the grid. The loads are transferred to the existing main roof trusses as point loads at the joints.

The second difference is in the approach to dealing with the existing parapet. The corner-post bearing expansions are constructed according to parapet option 3, in which the parapet will remain in place and the exterior edges of the modules will be aligned with the interior edge of the parapet. Figure 9-18 shows how in the model the integral perimeter beams bear to the inside of the parapet. This creates an offset condition from the columns internal to the parapet and the main truss nodes below. In practice the grid beam that the module bears on should be checked at this location for a shear or local buckling failure condition.
9.4.2.2 The Expansion

The expansion using the corner-post bearing modules was conducted in a similar fashion to the wall-bearing module expansion. The grid was initially created and the single-story expansion was modeled and analyzed. Thereafter, the two-story expansion was modeled and analyzed. The differences were primarily in construction of the module, transfer grid modifications and connections.

9.4.2.3 Connections

The connections to consider for the corner-post bearing module expansion are those same connections discussed during the wall-bearing module expansion section. Module-to-Module connections were assumed to be made with small sections of plate steel either bolted or welded to vertically and horizontally adjacent modules.

As in the previous section, the important connections will be discussed here. The grid-to-building connection was the same as discussed previously with the exception of the elimination of the grid beam above the diaphragm truss. The module-to-grid connection was different and will be discussed in this section. The horizontal module-to-module connection was the same as before, but the vertical module to module connection was different and will also be discussed in
this section. Lastly, the roof diaphragm-to-module connection was the same as in the wall bearing module expansion.

It was important for the corner-post bearing modular expansion to separate the module framing from the grid framing; except in locations where connections occurred. This was accomplish by using rigid links at the connection points. By modeling the modules in this manner, the loads could be transferred at the corner and any intermediate location only. This approach also provides a way to model the frame within the module being stacked on top of the grid beam without making them act compositely. The length of the link was specified to match the depth of the integral structural steel floor framing. Figure 9-19 shows how the module framing was attached to the grid beams running parallel to the main trusses.

Figure 9-20 shows how the modules were vertically connected in the model. A small HSS section was used to separate the upper and lower modules. The gap was set to equal the half-width of the W10 x 15 perimeter beam. The beam elements shown in the figure are located at the centroid of the cross-section of the member. As discussed in the previous paragraph, modeling the modules in this manner ensures that loads are only transferred through the corner points. Also able to be seen in the Figure is the module framing, steel angle on top of the walls, the rigid link connecting the horizontally adjacent modules and the 6” air gap between the two modules.
9.4.2.4 Load Path

Much of the previous discussion regarding load path of wall-bearing modules applies to the corner-post bearing modular expansion. Ultimately, the loads get transferred to the existing structure at grid intersections as point loads. The only difference is instead of distributing the loads to a grid beam above the diaphragm trusses as a linear load and then to the grid beam above the main trusses as a point load, the gravity loads are directly routed through the corner posts to the grid beam as a point load. There are some slight differences in the way the loads get transferred within the modules.

One difference, as touched on above, is the load path for the gravity loads. Load on the floor slab are distributed to the steel joist first and then to the perimeter beams as point load. The perimeter beams carry the loads to the corner columns and then downward to the main truss bearing points. The roof assembly is assumed to be light-framed so instead of heavy steel joist carrying load to the perimeter beam light joist will be used.

The second difference is that the concrete floor will act like a rigid diaphragm, which is different than the flexible diaphragm action expected to occur with the wall bearing modules. For this project the difference may not be apparent due to the regular, symmetric shape of the floor plan. For a more irregular shaped floor plan, there may be noticeable differences. The main consideration would be the force distribution to the attached shear-walls below. A rigid diaphragm would distribute load based on stiffness of the lateral force resisting system to which it
is connected; whereas with a flexible diaphragm the loads would be expected to be distributed to members of the connecting lateral force resisting members below on a tributary width basis.

To summarize, both the gravity and lateral loads are expected to be transferred to the existing structure at the same points that were discussed for the wall-bearing modules. The gravity loads will be applied to the main truss nodes as point loads and the lateral loads will be applied as overturning loads at the main truss nodes and distributed to the clay masonry shear walls as line loads by the floor diaphragm at the module base level. The grid intersection locations will be the same for this expansion with the only difference being that instead of a grid beam defining the intersection the perimeter beam integral to the modules will mark that same intersection. The main differences are expected to be in the weight and overall stiffness of the expansion. The corner-post bearing modular expansion is expected to be heavier and stiffer than its wall-bearing counterpart.

9.5 Structural Analysis of Expanded Building Results

Structural analysis was conducted for both the one- and two-story wall and corner-post bearing expansions. Important findings are discussed in this section, but a complete summary of the results is presented in Appendix J and the individual member capacity calculations are located in Appendix H. Those same elements reviewed in Chapter 8 are discussed in this section. The capacity calculations conducted for the existing building analysis are reviewed in this section to determine if the members are likely to have adequate capacity to support the expansions.

9.5.1 Capacity Calculations

9.5.1.1 Truss Analysis, Frame 3

Table 9-2 summarizes the internal loading for critical members as a result of each expansions and the member capacities calculated in Appendix H for the roof truss located along frame line 3. See Figure 8-9 for member designations. Member internal loads and deflections were compiled from the ETABS analysis of the building. Critical tension and compression members of the truss were reviewed for adequate axial capacity, respective to their loading.
As can be seen from the table, the truss appears to have adequate capacity to support all expansions. Noticeable in the results is the effects of the composite grid beam and the removal of the cinder fill on the roof. The internal loading for both the single-story wall bearing and single-story corner-post bearing are comparable to those of the existing building. It would appear that attaching the grid beam compositely to the existing truss would improve feasibility of expansion, especially a single story expansion.

**Table 9-2.** Summary of the capacity and internal loadings for all the expansions; Exist. = Existing Building, 1WB = Single-Story Wall Bearing Expansion, 2WB = Two-Story Wall-Bearing Expansion, 1CB = Single Story Corner-Post Bearing Expansion, 2CB = Two-Story Corner-Post Bearing Expansion, C = Member in Compression, T = Member in Tension.

<table>
<thead>
<tr>
<th>Member</th>
<th>Check</th>
<th>Capacity (Kips)</th>
<th>Exist. (Kips)</th>
<th>1WB (Kips)</th>
<th>2WB (Kips)</th>
<th>1CB (Kips)</th>
<th>2CB (Kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B426</td>
<td>C</td>
<td>1009</td>
<td>56</td>
<td>38</td>
<td>56</td>
<td>28</td>
<td>41</td>
</tr>
<tr>
<td>B427</td>
<td>T</td>
<td>921</td>
<td>371</td>
<td>360</td>
<td>519</td>
<td>377</td>
<td>568</td>
</tr>
<tr>
<td>D11</td>
<td>T</td>
<td>457</td>
<td>201</td>
<td>185</td>
<td>265</td>
<td>195</td>
<td>293</td>
</tr>
<tr>
<td>C134</td>
<td>C</td>
<td>412</td>
<td>138</td>
<td>128</td>
<td>183</td>
<td>134</td>
<td>202</td>
</tr>
</tbody>
</table>

The mid-span deflection for the roof truss are shown in Table 9-3. Once again the effects of the composite grid action and removal of the cinder fill can be seen. The magnitude of the deflections, due to dead load, for the single-story expansions are comparable to those without any expansion; however, there is additional floor live load that must be considered when evaluating deflection. Considering the floor live load an extra 0.25” exists for the single-story expansions.

Both of the two-story expansions create mid-span deflection of approximately 1”, which is almost double that of the existing building deflections. Whether this is acceptable would be largely up to the owner of the building. Considering an L/240 deflection limit, which is typical for roof structural members, the deflection would be acceptable; however, plaster ceilings are attached to the bottom chord of the trusses which could be damaged by the additional truss deflection. It may be more appropriate to set a total deflection limit. It would be prudent to determine the consistency and condition of the plaster ceiling and investigate an acceptable deflection limit based on experience of the design team.
Table 9-3. Mid-span deflection of the roof truss along frame line 3; S = Deflection due to snow loading, D = Deflection due to dead loading, L = Deflection due to floor live loading, N/A = Not Applicable.

<table>
<thead>
<tr>
<th>Type</th>
<th>$\Delta$ Limit (in.)</th>
<th>Exist. (in.)</th>
<th>1WB (in.)</th>
<th>2WB (in.)</th>
<th>1CB (in.)</th>
<th>2CB (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\Delta_D$</td>
<td>N/A</td>
<td>0.39</td>
<td>0.40</td>
<td>0.52</td>
<td>0.42</td>
<td>0.62</td>
</tr>
<tr>
<td>$\Delta_S$</td>
<td>L/360=2.58”</td>
<td>0.15</td>
<td>0.12</td>
<td>0.12</td>
<td>0.12</td>
<td>0.11</td>
</tr>
<tr>
<td>$\Delta_L$</td>
<td>L/360=2.58”</td>
<td>N/A</td>
<td>0.23</td>
<td>0.36</td>
<td>0.22</td>
<td>0.36</td>
</tr>
<tr>
<td>$\Delta_{D+S}$</td>
<td>L/240=3.88”</td>
<td>0.54</td>
<td>0.52</td>
<td>0.64</td>
<td>0.54</td>
<td>0.73</td>
</tr>
<tr>
<td>$\Delta_{D+S+L}$</td>
<td>L/240=3.88”</td>
<td>N/A</td>
<td>0.75</td>
<td>1.00</td>
<td>0.76</td>
<td>1.09</td>
</tr>
</tbody>
</table>

9.5.1.2 Column A-3 Check

The analysis results and capacities for the critical column length located between the basement floor and 1st floor are shown in Table 9-4. The column has adequate capacity to support all expansions; however, the two-story corner-post bearing expansion is approaching the maximum column capacity.

Table 9-4. Summary of the structural analysis results for the column at grid intersection A-3.

<table>
<thead>
<tr>
<th></th>
<th>Exist.</th>
<th>1WB</th>
<th>2WB</th>
<th>1CB</th>
<th>2CB</th>
</tr>
</thead>
<tbody>
<tr>
<td>$P_u$ (Kips)</td>
<td>242</td>
<td>283</td>
<td>355</td>
<td>291</td>
<td>395</td>
</tr>
<tr>
<td>$M_u$ (Kip*ft.)</td>
<td>26.1</td>
<td>26.2</td>
<td>28</td>
<td>27</td>
<td>28</td>
</tr>
<tr>
<td>Capacity Ratio</td>
<td>0.592</td>
<td>0.674</td>
<td>0.824</td>
<td>0.693</td>
<td>0.903</td>
</tr>
</tbody>
</table>

9.5.1.3 Pad Footing Supporting Column at A-3

The soil pressures developed as a result of external loading at the bottom of the footing for both the existing condition and the expanded conditions are shown in Table 9-5. As expected, the corner-post bearing expansion created the largest effect on soil pressure. The allowable bearing capacity is not known for the soils in the vicinity of the footings but would have to be determined in practice in order to decide whether soil pressures are acceptable.
In order to estimate adequacy of existing conditions without geotechnical information, it is helpful to look at the presumptive values shown in IBC 2009 Table 1806.2. The table estimates that sedimentary or foliated rock would have a bearing capacity of 4,000 PSF, sandy gravel and/or gravel would have a bearing capacity of approximately 3,000 PSF and that soils consisting of sand, silty sand, clayey sand, silt and sandy silt would have a bearing capacity limit of 2,000 PSF. Considering the IBC bearing capacity estimates; the footings for the two-story corner-post bearing expansion would have to be bearing on a weak bedrock, at a minimum, in order to not exceed the bearing capacity.

Table 9-5. Base reactions (P) and base soil pressure (q footing) at the footing for column A-3.

<table>
<thead>
<tr>
<th></th>
<th>Exist.</th>
<th>1WB</th>
<th>2WB</th>
<th>1CB</th>
<th>2CB</th>
</tr>
</thead>
<tbody>
<tr>
<td>P (Kips)</td>
<td>182.1</td>
<td>196.7</td>
<td>248.2</td>
<td>204.0</td>
<td>278.9</td>
</tr>
<tr>
<td>q footing (PSF)</td>
<td>2745.7</td>
<td>2940.0</td>
<td>3624.8</td>
<td>3037.1</td>
<td>4034.8</td>
</tr>
</tbody>
</table>

9.5.1.4 Shear Wall, Line A

The clay masonry exterior walls are part of the lateral load force resisting system and as noted in the structural analysis of the existing building discussion. The analysis of the walls under existing building loads pointed to the need for additional structural analysis and site investigation to determine actual capacity, which are beyond the scope of this research. The preliminary shear-wall analysis, located in Appendix H, indicates that it is likely that a percentage of wall line “A” (critical wall line) would have to be grouted and possible contain some level of reinforcement in parts to pass current masonry code checks. It is important to note; however, that the approaches were conservative in nature and the upper-bound check shows adequate capacity with an unreinforced, un-grouted section. As noted in the analysis of the existing building section, the contributions of additional wall assembly components would have to be reviewed along with the mechanical properties and loading. A site investigation would be required to determine the level of grouting and reinforcement levels of the wall assemblies.

No further analysis was conducted on the masonry walls. The necessary information was from the preliminary checks adequate to determine feasibility of expansion and make commentary on anticipated repairs or investigation. In regards to the shear-walls, it should be recognized that added cost would have to be budgeted to conduct further investigation of the
exterior walls to determine whether steel reinforcement is present. Additional structural analysis would also be required to confidentially assess whether the wall indeed has the required capacity to support any level of expansion.

9.5.2 Load Distribution Check

Figure 9-12 and 9-19 shows the link element grid connection to the existing roof trusses. This connection was designed not only to transfer the loads properly, but to provide information regarding the axial loads at these points. The links were assigned some stiffness in order to develop axial loading in the analysis providing for the load distribution from the proposed expansions onto the existing building to be mapped (Grid coordinates shown in Figure 9-21). The detailed results of this mapping are located in Appendix K; whereas, summaries of the results are shown in Figures 9-22 and 9-23.

Conditions were mapped for three load cases and all four expansions. The load cases reviewed consisted of the highest gravity load case and lateral wind loading from both the North and West in conjunction with a high gravity load (not the highest). The distributions were reviewed to assess the loading pattern that develops on the existing building when the proposed expansions are subjected to external loading and to determine if increasing the amount of stories in the expansion would change the pattern. The following load cases were reviewed (See Appendix F for a more complete description of each load case:

1. LC2 – Highest gravity load
2. LC4a – High gravity load in combination with winds from the North
3. LC4d – High gravity load in combination with winds from the West
Figure 9-21. Coordinate map for the transfer grid.
Figure 9-22. Graphs showing the axial load (measured at the link) at grid intersections along labeled grid lines for both the single and two-story wall-bearing expansions. See Figure 9-20 for grid line location; L1_1 = Grid Line 1 Single-Story, L1_2 = Grid Line 1 Two-Story, L2_1 = Grid Line 2 Single-Story, L2_2 = Grid Line 2 Two-Story, A_1 = Grid Line A Single-Story, A_2 = Grid Line A Two-Story, C_1 = Grid Line C Single-Story, C_2 = Grid Line C Two-Story.)
Figures 9-22 and 9-23 show the LC2 load distributions that were mapped for all four expansions along an interior grid line and an exterior wall line for both directions. In general, the one and two-story modular expansion did not behave in an unusual manner due to the construction type and cellular nature. The load distributions, for these limited height expansions, were largely similar to what you would expect from a more conventional construction method. In comparison, the distributions for the wall-bearing expansion and corner and the corner-post bearing expansion had slight differences but were similar.

The wall-bearing axial load distributions on the existing trusses for both the one-story and the two-story expansion were symmetrical to match the symmetrical floor plan and the axial loads, as expected, were lower in magnitude than the comparable corner-post bearing expansions. Following are few more observations, regarding the wall-bearing expansion.

- L1 is along the line of the existing roof truss adjacent the East exterior wall. Nodal loading is relatively uniform along the line with the main nodes (A, B, C, D, E and F) experiencing more load than the intermediate nodes (A.5, B.5, C.5, D.5 and E.5). The nodal loads on the intermediate nodes rise slightly peaking at the central C.5 node. The rise is visible in the two-story expansion as well.
- In regards to L1 again, the loading level rises at intermediate nodes when comparing the one and two-story expansion. This is consistent with what was expected. Since these are wall bearing modules the exterior module wall along the line would carry some load from roof and upper floors. This load would be distributed uniformly on the grid beam below.
- Looking at L2, which is along an interior grid line, it can be observed that the increase in nodal load exhibited at the exterior wall between the two expansions does not exist at the interior location. This can be expected because there are no traverse walls modeled along this line, therefore the load is transferred by the longitudinal walls only.
- Along L2, two peaks can be observed in the graph at the intersection of grid lines B and E. These peaks are caused by the increased floor live loading, weight and framing differences of the maintenance modules.
- Looking at the graphs for lines A and C, it can be observed that nodal loads are greatest and relatively equal at the interior lines 2 and 3.
- In regards to exterior grid line A, the increases in nodal loads were more uniform between the two expansions. Looking at grid line C, it can be observed that the
difference between the exterior nodal loads (Intersection of grid lines 1 and 4) and the interior nodal loads (Intersection of grid lines 2 and 3) increases slightly when comparing the one and two-story expansion.

As noted earlier the corner-post bearing module distributions were largely similar to those of the wall-bearing modules with some small differences. The expansions were heavier than those created using the wall-modules. Other than the weight, the following can be observed from the graphs in Figure 9-22 when compared to those of Figure 9-23:

- The increase in magnitude of the nodal loads at intermediate location observed in the L1 graphs in Figure 9-22 between the one to the two-story wall bearing expansions is not present in the L1 graphs that compare the one and two-story corner-post bearing expansions.

- Along L2, the intermediate nodal loads appear to increase at the interior intersection of grid lines B, C and D; whereas, they appear to increase when reviewing the wall-bearing graphs in Figure 9-22.
Figure 9-23. Graphs showing the axial load (measured at the link) at grid intersections along labeled grid lines for both the single and two-story corner-post bearing expansions. See Figure 9-20 for grid line location; L1_1 = Grid Line 1 Single-Story, L1_2 = Grid Line 1 Two-Story, L2_1 = Grid Line 2 Single-Story, L2_2 = Grid Line 2 Two-Story, A_1 = Grid Line A Single-Story, A_2 = Grid Line A Two-Story, C_1 = Grid Line C Single-Story, C_2 = Grid Line C Two-Story).
The distribution graphs for LC4a and LC4d are located in Appendix K. Although the magnitude of nodal loads for these load cases are less than those in LC2 due to the load factors, the load distribution patterns are largely the same. No significant variation was noted between the distribution of these loads and LC2 results.

It was originally anticipated that overturning forces created by lateral wind loading would increase the axial gravity loads at interior node locations; however, this was not the case. Overturning forces were noticeable as axial loads at exterior grid lines, but not at interior grid intersections along the longitudinal wall lines (A, B, C, D, E, and F).

Figure 9-24 illustrates why this occurred. At those grid locations the adjacent modules are connected to the grid beam running along the main truss line through their HSS columns, which are separated by a 6” air space. Because of the air space the adjacent shear wall segments are relatively independent of each other and each create their own overturning force couple, due to shear (V) loading. Referencing Figure 9-24, at the intersections it can be seen that the compressive reaction (C) at the left hand column is offset by the tensile reaction (T) generated at the right hand column. Although the net axial force, at the intersection is zero, there will be an applied moment to the grid beam below, noticeable at the intersection, from the overturning effects.

![Overturning couple created at the intersection due to wind loading on the expansion.](image)

Figure 9-24. Overturning couple created at the intersection due to wind loading on the expansion.
9.5.3 Summary of Structural Analysis Results

As a general note regarding the structural analysis conducted for this project, the analysis conducted for this research was based on certain assumptions contained within this document and also a simplified floorplan. In addition, the analysis was only conducted on a representative sample of structural assemblies in order to make some commentary regarding the feasibility of a one or two-story modular expansion and the resulting effects of the expansions on the existing structure. This analysis should, by no means, be misconstrued as a complete structural analysis for the expansions adequate for design. In order to construct any of these expansions a much more complete analysis and specification would have to be conducted. That being said, the following can be said about the expansions as determined by the limited analysis conducted.

The existing roof trusses appear to have adequate strength to support all of the expansions, with a few caveats. If the cinder fill is removed and if the main roof trusses are able to assume some composite action with the aligned grid members placed above them, the effects of a single story expansion (either wall or corner) would be minor. The results of the expansion analysis show comparable levels of stress and mid-span deflections for the roof truss along frame line 3, which is likely to be a controlling member in a design situation. The analyzed truss also shows adequate strength for both the two-story wall-bearing expansion and the two-story corner-post bearing expansions; however, the deflections for both of the two-story expansion were close to double that of the original configuration. It is possible that the attached plaster ceilings below may experience some level of cracking if a two-story expansion is added to the existing building.

The column along frame line 3, embedded in the wall, shows adequate strength for all proposed expansions, but it is questionable whether the soil has adequate strength to support the expansions. Both single-story expansions would require a bearing capacity of approximately 3,000 PSF, whereas the two-story expansions would require bearing capacity of approximately 4,000 PSF. The corner-post bearing expansions are heavier and do cause greater soil bearing pressure increases when compared to the wall-bearing expansions so if bearing capacity is critical the lighter wall-bearing modules may become preferable to the heavier corner-post bearing modules.

The clay-masonry shear walls will require more detailed analysis and site investigation to determine whether they would be adequate for any level of expansion. The adequacy of the shear capacity under existing conditions, as evaluated by current masonry and building code is unable to be determined with confidence using the preliminary methods used in this thesis. A more
detailed check should be conducted, which would take credit for both the spandrel sections over openings contribution to overall shear wall capacity and the urban setting to potentially reduce wind loading. An investigation (destructive or non-destructive) should be made to determine, more accurately, the cross-section of the exterior wall and the level of reinforcement in the exterior wall.

In regards to the load distribution, it appears from the graphs, that when considering a one or two-story expansion the performance and distribution of the loads from the proposed expansion is similar to what would be expected from traditional construction methods. There were some slight differences in the data but they were too small to identify any solid trends in comparison to traditional framing methods. It is possible if more stories are added the distribution could change (or noted observations amplified) and difference could be identified between the load distributions of the cellular based modular construction method and site-built framing methods, but at this limited level of expansion no significant effects can be noted.

9.5.4 Discussion on Feasibility of Modular Vertical Expansion

The last step listed in Figure 5.8 is to evaluate the feasibility of the expansion. As noted throughout the research, many items such as building and township codes, existing building condition and geometrical constraints, construction concerns, architectural considerations and project schedule can affect the feasibility of expanding an existing building using modular construction.

In this case, a one- or two-story expansion appears to be feasible, based on the structural analysis and the preliminary code assessment conducted in this research (assuming that zoning or township restrictions do not exist). Whether it is practical to expand this building using modular construction is dependent largely on whether the beneficial aspects of modular construction such as reductions in project schedule, reduced community and business disruption and reduction in stockpiling requirements have value to the building owner.

An initial factor found that influences the use of modular construction methods for a vertical expansion is whether there is a modular manufacturer in the vicinity that will accept the project. Considering a vertical expansion with a relatively low square-footage of required modules, such as the one researched in this thesis, it may be challenging to identify a modular manufacturer that could deliver such a small project. It was noted from the factory visits that
each manufacturer develops a sense of an economy threshold, in terms of deliverable project square-footage. The threshold appears to depend primarily on the module type produced and the efficiency of the manufacturer’s production process. If manufacturers accept projects below this threshold they risk losses.

It is important to note that much work is necessary to mobilize a modular plant for any particular project. The plans must first be evaluated for module construction, efficient work-flow established, labor must be organized and building material ordered and staged. There is a significant startup cost associated with a modular project. This startup cost must be recouped before any significant cost-savings can be realized, which sets a break-even point that must be crossed before a modular project is likely to be economical. Each type of module has different thresholds and may be appropriate for different scale projects. For this project it was identified that a wood-framed wall-bearing module would likely have the lowest break-even point in the U.S.

However, a major limitation that could prevent wood-framed modules from being used on a project is the building construction-type classification. From the review of the 2009 IBC requirements pertinent to this project, it was identified that modules constructed with combustible materials may not be acceptable, due to building height restrictions, for use in greater than a one-story expansion or a two-story expansion with reduced ceiling heights. This is of concern because it would limit the use of wood-framed wall-bearing modules, as well as, CFS wall-bearing modules with combustible items such as subflooring in some situations.

Considering their use in a vertical expansion of an existing building, the wall-bearing module type has two main advantages over the corner-post bearing module type. Initially, the wall-bearing modules are lighter than the structural steel corner-post bearing modules, which has been identified as beneficial to vertical expansion projects. Less weight means less stress increase to the existing structural system. Results from the structural analysis show that the soil bearing pressure created by the two-story corner-post bearing modular expansion is the greatest and close to the threshold of what a designer might expect to be acceptable. Secondly, as discussed above, the wall-bearing modules are likely to have a lower scale of economy.

It is not likely from discussion with corner-post bearing module manufacturers, that a project of this size would be economical for them to accept. This may eliminate modular construction from consideration if the building owner requires a minimum of a two-story expansion to make the project economical and is unwilling to accept reduced ceiling heights.
It is important to note that building height restrictions could make other construction methods such as panelized construction, traditional light-framing methods or moment frame constructions more attractive alternatives. The redundant floor/ceiling assembly may make modular construction less attractive to designers when building height restrictions exist and it is still desirable or economical to use combustible construction. It may be possible to use thinner floor/ceiling assemblies with site-intensive construction methods. With thinner floor/ceiling assemblies it may be possible to increase ceiling heights, to acceptable levels, without increasing overall building height; thereby, meeting building code restrictions for combustible construction.

It appears from the review of studies conducted in the European union (EU) on renovation and extension of existing buildings (Lawson et al. 2013), (The Main SuRE-FIT Results 2009) that competition does exist for modular construction methods used in vertical expansions. From review of the EU studies, steel moment frame construction, panelized construction, and light-framed construction do appear to be strong competitors. Modular construction does; however, appear to be the most expedient construction method.

Two other observations regarding the EU studies were that nearly all extensions discussed in the studies where single-story and the extension component of the project, in many of the case studies, was only part of a more comprehensive energy or cladding retrofit. As noted in Chapter 7, when additions are proposed to a structure the 2009 IBC Sections 3403.3 and 3403.4 permits existing structures to remain unaltered when the increase of gravity loads from the addition is less than 5% and the demand/capacity ratio for members of the lateral force resisting system is maintained at less than 10% when considering the addition.

It is possible that a light-weight single story expansions may be able to meet this requirement in some instances and reduce the amount of structural analysis involved in a project as well as potential strengthening measures required for the structural system of the existing building. When considering single-story expansions, especially in this research project, wood-framed wall bearing modules would be a viable construction method alternative to consider. Reducing the amount of structural engineering required and strength retrofits required both have the potential of improving the overall economy of a project.

Additional items also have the potential to improve project economy. Considering any necessary energy/cladding improvements, as mentioned earlier, could bring additional economy to a project. Feasibility may improve when considering the factors discussed in this section, in addition to the lease income generated by the expansion. Incentives may also be available from
state or federal agencies for energy improvements to existing buildings or restoration of historic buildings, which may further improve project economy.

Following is a list of a few other items that may have an effect on the feasibility of a modular expansion. Although these items are specific to this project the discoveries could relate in general terms to other projects.

1. Transfer structure – The need for exceedingly complicated transfer structures could increase costs and decrease the feasibility of vertical expansion. Construction with modules can require more bearing points than construction with some site-intensive construction methods such as moment frame construction, were opportunity exist to design long spans and reduce interior bearing locations. For this project the transfer structure was relatively simple and should not be overly costly to construct. It is likely that the grid transfer structure used for this project would not decrease feasibility of using modular construction in this case.

2. Dimensional Constraints – In this project, other than the questionable ceiling heights, it appears that modules could be constructed reasonably to meet project requirements. Projects with irregular floor plans, changes in ceiling heights in a story, or non-repeatable floor plan elements may not be practical to modularize and if so may require complicated transfer arrangements.

3. Access and Egress – Both of these necessities could limit feasibility of any vertical expansion project. In this case, both primary access and egress was already provided by a recently constructed elevator core. Secondly, because it appears as if the building was originally designed for an additional floor stair shafts in the rear of the building were already extended to roof level, which would provide for convenient rear access and egress. In buildings not intended for vertical additions in their original designs the ability to provide access and egress to the new addition, especially accessible access and egress could decrease the feasibility.

4. Fire Sprinklers – 2009 IBC requires an automatic sprinkler fire protection system throughout all building containing a group “R” fire area. Therefore, the addition of residential units to an existing building invokes a building code mandated requirement for the provision of an automatic sprinkler system throughout, not only the new additions but also throughout the existing building. If an existing building does not have an automatic sprinkler system previously installed then the cost of adding one throughout can affect the feasibility of an expansion. It is likely, that
provisions for automatic sprinklers could be made in the factory when constructing the modules, but keep in mind that each module is shipped and installed individually. Mechanical splices must be made in the field after module installation at adjacent modules for utilities services. This could potentially make a site-intensive construction method more attractive in some instances.

5. Mechanical Connections – As mentioned in item number 4 utility service through adjacent modules must be spliced. For complicated mechanical systems the planning of the splices as well as the addition of a discontinuity in run at each splice location may deter a design team from selecting modular construction methods for a project.

6. Existing Mechanical Systems – For this project it was assumed that mechanical system existed on the roof and would require relocation. It was also assumed that interior renovations would be happening simultaneous to the vertical expansion and that it was likely that new roof-top equipment would be installed for existing space conditioning. So, in this example of a vertical expansion there would be no loss on the project besides that of floor space on the 1st modular floor. Once again it seems as if the feasibility of a vertical expansion can improve when paired with another required renovation. In many instances the amount of roof-top equipment could be substantial and no replacements anticipated. In these instances it could be cost-prohibitive to relocate all the equipment or plan around it.

7. Truss Deflection – Limiting the expansion to a single story has many positive points one of which concerns the existing truss deflection. In this project, the existing roof truss was detailed to act compositely with the transfer grid and the heavy existing cinder fill was removed before the grid installation. Because of these two items, both the expansion with a single-story of wall-bearing modules and corner-post bearing modules, created deflections in the roof truss comparable to that of the building subjected to current loading. With the addition of a second story, truss deflections could nearly double. This may be problematic considering the plaster ceiling attached to the bottom chord of the truss.

8. Shear Walls – The extent of structural investigation required for the project may influence feasibility. The initial structural investigation of the shear walls in the HMAC building showed pointed to the need for further structural investigation and analysis. This increases initial costs of a project and may be problematic or
prohibitive if the wall is indeed deficient and requires strengthening measures. This could decrease the feasibility of the project.

9. Bearing Capacity – In this project bearing capacity can be a limiting factor, especially considering a two-story corner-post bearing expansion. Structural investigation will likely be required to determine bearing capacity along with composition and reinforcing of pad footings.
Chapter 10

Summary and Conclusions

In this chapter, a short summary of the work that has been conducted will be given and general conclusions discussed. The chapter begins with a section summarizing the research that was conducted and the purpose of the project. Following is the conclusions of the research, in which the overall project conclusions are discussed along with results of the vertical expansion study. In the vertical expansion discussion, opportunities for module improvements, as well as, potential improvements in the structural analysis are identified. Lastly, the contributions of the research are stated and some opportunities for future research are described.

10.1 Project Summary

Prefabication methods have been recognized by NIST as well as other organizations having stake in the U.S. construction market as a potential way to increase productivity within the U.S. construction industry. In order to successfully implement prefabricated components into a project, pre-planning efforts must be made to ensure that seamless integration of the prefabricated component occurs and economy can be gained by selecting a prefabrication method over a site-intensive construction method. Tools like BIM, VDC and CAD/CAM can be used to assist the design team in the integration process. Prefabrication can be used in a project to speed-up project schedule, cut costs or incorporated complicated details/materials into components. Misuse or misapplication of prefabrication on project can lead to losses on a project, which can discourage the use of prefabrication on future projects. Ongoing research is being conducted in the field of prefabrication to educate the construction industry on how best to harness the productivity benefits available through the use of prefabrication in construction projects.

This research is part of the ongoing research effort within the academic community and is intended to add information to the growing body of knowledge on the use of prefabrication methods in building construction projects. In this research one form of prefabrication, multi-story
modular building construction was investigated to determine the available benefits of the construction methods, how to best utilize those methods and define appropriate current uses as well as potential future uses of the technology.

A state-of-the-art review was initially conducted to assess the use of multi-story modular construction, identify appropriate applications, identify typical modules, and identify design guidance as well as codes engineers could use for the purpose of feasibility analysis and design of modular structures. The state-of-the-art review helped to accomplish the first three objectives of the research and provided the background information necessary to proceed with the final objective of testing an underutilized concept that could be used more frequently in the U.S. construction market.

Modeling and structural analysis were conducted for the fourth objective. The results of the analyses were used primarily to discuss the potential productivity benefits of the construction method and bring to light factors that could affect the feasibility of multi-story modular construction. Prior to the concept selection, a conceptualization exercise was conducted to identify a few contending concepts that could potentially be used as the tool needed to accomplish the fourth objective. Three were identified/conceived and one was selected.

The idea of vertically expanding existing commercial buildings using modular construction was selected as the conceptual tool. An analytical model of an urban two-story existing building was created and structural analysis was performed on the existing structural system to identify critical structural elements that could be effected by a vertical expansion. Next a one and two-story expansion was added to the model and analyzed to determine the likely effects of the expansion on the existing structural system. Expansions were modeled using both a wall-bearing module type and a corner-post bearing type module to identify any advantages/disadvantages of each system.

10.2 Conclusions

The initial question of why multi-story modular methods appeared to be used and researched more broadly in the U.K. in comparison to the U.S. construction market was addressed in the report by Lawson et al. (Lawson et al. 1999). Lawson et al. discusses that interest in modular construction largely arose from the successful implementation of bathroom pods in commercial building construction in the U.K in the 1970’s. Modular construction began to
expand to the construction of whole cellular buildings, such as hotels and residential buildings, to meet the time period demands for increased speed of construction and early return on investment. In the U.S. since World War II, primary manufacturing focus has been on mobile homes and single family dwellings. The U.S. industry largely developed to provide temporary workforce housing for those involved in war efforts and meet housing shortages due to the homecoming.

Currently the presence of multi-story modular construction in the U.S. building construction market is increasing but still limited to niche markets. Permanent modular construction is being utilized to construct repetitive cellular building such as student housing projects, apartment buildings and military housing. Modular methods are chosen over others primarily for reasons of speed of construction, early return on investment and reductions in community disturbance.

In general, productivity increases appear possible through the selection of multi-story modular construction methods, but are sensitive to the application and quality of implementation. Modular construction methods are not appropriate for all projects and the success of using these construction methods for a project appears to be dependent on application, design effort and project management throughout. It is no surprise that modular projects have met with varying degrees of success in the past. Implementation of prefabrication methods in a project requires a significant amount of pre-planning to ensure success. Additionally, it is very difficult to make changes once production of the modules begin and especially construction of the modular structure. Small mistakes can easily cascade into costly remedial effort. The greater the amount of prefabrication involved in a project the larger the preplanning effort likely involved. Smaller prefabricated items such as utility racks and medical headwalls have been successfully used on larger projects. Smaller prefabricated assemblies are likely to have less risk involved with successful implementation.

Design and construction of a building is a difficult process filled with many choices that are, at times, difficult to pre-plan. Situations often arise during construction where it is beneficial to the owner to make changes to a building or redesign portions of the building after it is initially constructed. When using site-intensive construction methods it is likely that it is easier to respond to changes.

BIM has been identified by many sources as a management technique that could further improve productivity of modular construction. BIM can potentially help organize the pre-planning effort required for larger-scale project such as multi-story modular buildings. Advanced construction management tools such as VDC and cloud services along with dedicated BIM
managers can likely assist prefabrication integration efforts in a project. It is important to note that BIM is in its early stages of development.

As mentioned previously, modular methods have been found to increase the productivity of a multi-story residential project if the application is appropriate for the methods, management is dedicated, adequate pre-planning efforts are made and a modular manufacturer is brought in to a project at the conception stage rather than after the project is awarded. In particular, smaller apartments, military housing, social housing, work force housing, housing in remote areas not able to secure building materials readily, projects located in areas where significant community disturbance is unacceptable, such as inner city locations and student housing where only short windows of time exists to construct the building, are applications that multi-story modular construction methods have been successfully used for in the past.

10.2.1 Vertical Expansion

The goals of the modular vertical expansion research were accomplished through the research provided. The first goal of the study was to define the design process. This was accomplished in Section 4.8. The second goals was to identify the factors that affect feasibility of modular vertical expansion and separating out those that apply specifically to modular construction. This was accomplished by shading the factors that are directly related to modular construction in the process map located in Section 4.8 and also by summarizing them and adding description in Table 10-1. The objectives of the study were accomplished as follows:

1. Accomplished in Chapters 5 and 6.
2. Accomplished in Chapter 8.
3. Accomplished in Chapter 9
4. Accomplished in Table 10-1

European examples of vertical expansion using modular construction were identified in literature review. These examples were used largely in retrofits and often combined with other activities such as cladding or energy improvements. It is possible that vertical expansion could be another U.S market for modular manufacturers. Although it is likely that modular construction has been used for these types of projects in the U.S., no examples were discovered in literature review.
Vertical expansion using modular construction was found to be feasible and practical in some instances, but once again dependent on the application, existing building type and the items discussed in the previous discussion regarding feasibility. It was found that this construction method might be advantageous if the geometry of the floor plan was appropriate, regular and repeatable as in newly constructed modular buildings and the transfer structure can be economically implemented. Other situation where it might be advantageous to use modular construction for the expansion include situations where time is of the essence, when the project is located in cities were wages are high, when there is limited stockpiling space available or the expansion needs to be constructed quickly without disturbing the existing business or community. It is likely that modular construction will face significant competition from other site-intensive forms of construction, such as light-framing, structural steel framing and panelized construction when attempted to be applied to projects that vary from these types of advantageous situations.

To summarize a few important points regarding the use of modular construction for vertical expansion. Wall-bearing modules were found to be lighter than structural steel framing, but perhaps not lighter than a light-framed or panelized framing construction method. The lighter weight modules have advantages for vertical expansions but could face limitations such as building height restrictions mandating non-combustible construction types. Corner-post bearing modules were comparable in weight to structural steel framing and are non-combustible allowing their use in greater height expansions. Redundant floorceilings were found to be a concern. The redundancy adds overall height to the expansion and may restrict number of stories or ceiling heights. This can present difficulties when considering the economy of a project. Lastly wood-framed wall-bearing modules showed promise for use in single story expansions and appeared, from discussion with modular manufacturers and successful history of the single-family modular industry, to be the most realistic choice, in terms of economy of scale, for lower square footage additions.

This research was conducted to help identify factors that impact feasibility of implementing the pre-fabricated method of multi-story modular construction. The design process for a case study building was studied in order to bring-to-light parts of the design process that could impact feasibility of project success. In this instance, vertical expansion was chosen as an application potentially appropriate for modular construction methods. The process map identified in Section 4.8 was used as a guide for the design process. The process is quite lengthy and has many processes that need to be completed, having many inputs and decisions that need to be made along the way. A large percentage of the factors that impact the feasibility of a vertical
expansion project apply to both site-intensive methods and modular methods of construction. Although some steps along the path do not directly apply to modular construction methods, the proper accomplishment of those tasks still affect overall project success. Poor performance of step not related to modular construction methods could in fact produce the outward appearance that modular construction is to fault for project failure.

Table 10-1 was produced to help summarize the factors that were found to directly impact feasibility of a modular vertical expansion. The general categories that apply to all potential modular vertical expansion are shown in the table along with the project specific findings for the category.

**Table 10-1.** Summary design process steps, specific to modular construction, that have been found to have a potentially impact feasibility of a vertical expansion project.

<table>
<thead>
<tr>
<th>Generalized Process Steps:</th>
<th>Commentary</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Pre-Planning Phase:</strong></td>
<td></td>
</tr>
<tr>
<td>1. Properly evaluating the candidacy of a building for vertical expansion is important.</td>
<td></td>
</tr>
<tr>
<td><strong>1.1 Conceptual Evaluation:</strong></td>
<td></td>
</tr>
<tr>
<td>(b) Constructability</td>
<td>1. Can strong points be identified in the existing structure for module attachment to ensure stability</td>
</tr>
<tr>
<td>(c) Interfaces</td>
<td>1. It may be difficult to apply a fully modular construction method to some projects.</td>
</tr>
<tr>
<td>(d) Safety/Access</td>
<td>1. Ability of the modular method to provide the required fire-resistance.</td>
</tr>
<tr>
<td><strong>1.2 Modular Application Evaluation:</strong></td>
<td></td>
</tr>
<tr>
<td>(a) Transportation regulations</td>
<td>1. Limiting dimensions.</td>
</tr>
<tr>
<td>2. Regulations affecting design.</td>
<td></td>
</tr>
<tr>
<td>(b) Proposed geometry of expansion</td>
<td>1. Repetitive, easily reproducible units.</td>
</tr>
<tr>
<td>2. Is there a lot of open-spaces required?</td>
<td></td>
</tr>
<tr>
<td>(c) Amount of available planning time</td>
<td>1. Is there adequate time for complete design?</td>
</tr>
<tr>
<td>2. Change orders are difficult to implement after factory production has begun.</td>
<td></td>
</tr>
<tr>
<td>(d) Manufacturer input</td>
<td>1. Manufacturer can provide planning input to maximize benefits of the modular construction method.</td>
</tr>
<tr>
<td>(e) Factory location</td>
<td></td>
</tr>
<tr>
<td>(f) Site access and stockpiling space</td>
<td></td>
</tr>
<tr>
<td>(g) Factory break-even point</td>
<td>1. It may be difficult to find modular manufacturers to deliver low square-footage projects.</td>
</tr>
<tr>
<td>2. Wood-framed modules show promise for lower square footage expansions of acceptable height in relation to the building code.</td>
<td></td>
</tr>
<tr>
<td>(h) Applicability of Modular Benefits</td>
<td>1. Do the main benefits of time savings and reduced community disturbance apply to the project?</td>
</tr>
<tr>
<td><strong>Structural Evaluation of Existing</strong></td>
<td>1. There is little in this step that is directly related</td>
</tr>
</tbody>
</table>
**Building Phase:**

Building Phase: to modular construction; however, neglecting portions of this important step can have large impacts on project feasibility. There are many steps in this section that require careful attention to ensure project success.

### 14 Incentive Programs

(a) Financial incentive programs

1. Enrolling in an incentive program may bring additional economy to a modular project
2. Combining a modular vertical expansion with a cladding or energy retrofit may bring additional economy to a project.

(b) Special Programs

1. Programs that give recognition for material efficiency or project safety may give credit for using modular construction methods.

### Expansion Conception Phase:

(16) Expansion Building Code Review

1. This has been an area that has been found to potentially have a large impact on feasibility of modular construction.

(a) Allowable building heights and construction type

1. Redundant floor/ceiling assemblies can affect the selection of modular construction methods if ceiling height is a premium and designers are attempting to maximize stories.
2. Mandates whether combustible or non-combustible is required, which influences module selection.
3. Redundancy in modules can reduce finished floor area.

(b) Fire-protection requirements

1. The level of difficulty of implementing these requirements can influence the decision to use modular construction methods.

(15) Generate Module Options

1. This is part of an expansion planning iterative loop.

(a) Floor plan geometry

1. Open floor-plans can require structural steel modules or a hybrid construction.

(b) Height and area estimates

1. Necessary to determine the appropriate structural system.

(c) MEP connections

1. Complicated systems may be difficult to implement and involve too many splices/connections.

(18) Transfer Mechanism Planning

(a) Bearing requirements

1. Influences type of transfer mechanism required.

(d) Connection points

1. The ease and amount of connections required can impact feasibility.
2. Transfer mechanisms must satisfy both module bearing requirements and existing structural system requirements.

Existing and new MEP equipment

1. Will fully constructed modules be able to be implemented with existing MEP equipment or will
It is possible that there are short-comings and opportunity for new module types in the multi-story modular industry. Current module type appear suitable for a niche market of projects and more suitable for use in re-locatable buildings. U.S. modular manufacturers appear to have large overhead, on average low levels of technology implemented for production of modules and large amounts of employees. In literature review some modular factories tend to struggle or go
out of business when market demand slows. The redundancy created by the structurally independent six-sided modules appears to be useful for a re-locatable building, but not in a permanent setting where little provision is made for future disassembly.

The modular construction industry could benefit from research involving new module structural systems or fundamental modular design philosophies. Current modular design appears to be largely based on adapting typical site-intensive construction methods to a modular design goal. There could be benefit on revisiting basic multi-story modular design assumptions and investigating new module designs based on modern materials, structural systems and analysis techniques based on current markets for multi-story modular construction.

Non-structural modules appeared to have some promise for expanded use in the construction of multi-story buildings. Many designers were found to be presenting multi-story open-building concepts that incorporate modifiable floor plans (some examples can be seen in Appendix C). Non-structural modules were currently found in the literature review to be used mostly for bathroom pods and similar. Designers of structures like the Nakagin capsule tower and Disney’s contemporary resort attempted to use non-structural modules for replaceable apartment units. Unfortunately, both of these structures had issues that prevented the realization of the intention.

Arup’s City of the Future, referenced in Appendix C, incorporates non-structural replaceable modules into its vision of multi-family housing to the future. The vision involves the use of a structural system that can house these modifiable modular components and buildings that are mixed use in nature, can generate power, purify air and have multiple owners. Much opportunity exists for research on buildings of this nature. The opportunity specific to the theme of this research would be to identify appropriate structural systems, transport systems and module types that could support this vision.

The novel idea of MODS conceived and described in Appendix A is a possible research avenue for the use of non-structural modules. This idea takes the concept of a mobile home and adapts it to the needs of this generation. MODS can provide an affordable housing solution to a struggling demographic group.

In regards to vertical expansion using modular construction, an opportunity for research may exist with the transfer mechanism. Integrating the transfer structure into the module structural system may save height and eliminate a site-intensive construction effort. It should be possible to incorporate a truss structures in the longitudinal walls of modules. The walls of modules typically have adequate height and thickness to make large spans possible.
An opportunity may exist for a new type of modular manufacturer the U.S. It was identified in the research that it could be difficult to find a modular manufacturer than could profitably deliver a smaller project like the expansion proposed for this research. Japanese modular manufacturers have been documented to produce custom modular products using high levels of technology, lean manufacturing philosophies and reduced personnel. An opportunity may exist for a manufacturer such as this to have a modular market share. The role of the manufacturer would be to produce lower square footage one-off custom modular projects. The manufacturer would have to keep overhead low and have sufficient flexibility in operations to adapt to custom project needs.

10.2.3 Potential Improvements with the Structural Analysis

In general, the structural analysis of the vertical expansion produced meaningful results; however, the many link connections required to unify the contiguous modules made grid modeling more tedious than desired and certain analysis results such as diaphragm stresses and grid member internal load effects difficult to assess. The level of detail and meshing of the area elements and beam elements was not appropriate to pick up these types of results. These results were out of the scope of this research, but in order to capture these effects in the analysis, a finer mesh would likely be required in both the beam elements and diaphragms at connection points.

Another improvement that could be made with the analysis is refining the wall and floor panel shell elements definitions. In order to simplify the modeling and computation required to define the model, these elements were designed to perform isotopically. This was adequate for the results that were sought in this research, but not truly representative of how the panels would perform in service. It would be an improvement to define these panels as orthotropic and specify the out-of-plane behavior.

Lastly, it would interesting to see how the shell models performed in higher expansions or even in models of new construction buildings. With the higher expansions it may be beneficial to model the shell modules at the base level with restraints versus at the roof level of an existing building model. It is possible to model the transfer grid at the base level as well if that is of interest, as in the test module model located in Appendix I. Loads could be transferred to an existing building model after analysis if required. Modeling an expansion in this manner may produce clearer results and the modeling process would be greatly simplified expedited.
10.3 Contribution of Research

This research adds to the growing body of research on the use of prefabrication techniques in building construction. The use of prefabrication in construction has been identified as a potential way to increase productivity in the construction industry, but the methods can be easily misapplied and requires significant planning to ensure successful implementation. Research in this field helps to identify potential design aspect important to consider when using the prefabrication technique of multi-story modular construction. Additionally, the research identifies appropriate general uses for multi-story modular technology and opportunities for future markets and research in the field. Continuing research in the field of prefabrication is necessary to increase confidence and improve implementation practices within the AEC community. It has been identified that some opportunity exists in revisiting modular design concepts, as well as with increased use of non-structural modules.

The vertical expansion component of the research provides useful information for those considering modular construction for an existing building expansion. Chiefly, it identifies modular construction as a strong contending construction method for expansions in some situations. The shell panel modeling technique developed for the analysis, with some refinement, could be a useful tool for designer to quickly develop models for foundation design or determine effects of a proposed modular expansion on an existing building. Additionally, the design codes, methodology and feasibility study results provide useful insight to those considering vertical expansion of an existing building.

Outlined below are the important contribution of the research:

1. Summary of observations and commentary on the state-of-the-art review of the multi-story modular construction industry,
   a. There was a significant amount of effort that went in to the state-of-the-art review. A combination of literature review, interviews with professionals, factory visits, conference attendance, and review of the multi-story building construction outlook was conducted for the research. This was necessary to develop a complete understanding of the market in order to identify appropriate applications and develop the design process for vertical expansion. The summary and commentary provided in Sections 2.10, 2.11 and 2.12 provide valuable insight developed from the combination of review components.
2. Identification of the new or underutilized applications discussed in Section 2.13, based on literature review and conceptualization,
3. Identification of the potential market for a modular manufacturer who can deliver low square-footage, one-off type projects,
4. Identification of potential increased uses of non-structural modules,
5. Design process map shown in Section 4.8. This map summarizes the major structural and architectural influential factors that apply to a modular vertical expansion,
6. Case study results discussed in section 9.5.4,
7. Generalized factors that affect the feasibility of a modular expansion identified in Table 10-1,
8. The use of shells to conduct preliminarily structural analysis for modular expansions. This method could have broader applications.

10.4 Concluding Statements and Lessons Learned

The goal of the research was accomplished by providing the new knowledge listed in Section 10.3. The knowledge produced from this research can be used by design professionals, modular manufacturers and other industry representatives to improve confidence in using modular construction methods for the appropriate projects. The overall research objectives were achieved as follows:

1. Accomplished in Sections 1 and 2.
2. Defined in Section 2.9.1
3. Defined in Section 2.13
4. Defined in Section 2.13 and Section 4.8
5. Detailed in Section 4.8 and Table 10-1

Prefabrication in building construction projects has been recognized as a way to improve productivity. Unfortunately, prefabrication is sometimes applied inappropriately and blamed for project failure. This can discourage the use of prefabrication in the construction industry. Fully understanding the prefabricated construction method and how it impacts the design process is critical to the success of project and achieving productivity gains. The largest factor that has been found to impact feasibility of a multi-story modular construction project is application selection. Improper application of Multi-story modular construction methods can have negative impact on
project success. Multi-story modular construction methods have been found to serve a niche group of uses in the building construction market. Vertical expansion is thought to be one of these uses.

The second largest factor affecting feasibility was the design process. The design process for a modular vertical expansion was studied in this research. It was found that there were many steps in the design process that were affected by using modular construction. Not understanding how the methods impact the design process can lead to inefficiencies and loss of productivity. It was also found that many steps in the design process of vertical expansion were not impacted by modular construction, but if not given proper attention, could impact overall project success. Failure to attend to the steps not impacted by modular construction, can provide bad input to those steps that are, thus giving the outward appearance that the use of modular construction methods is to blame for failure.

This research was a good opportunity to work with a prefabricated method and outline the design process. The conceptual case study exposed issues that would not otherwise be addressed until the method was put into practice on a project. Examples would be:

- Working with geometric constraints of modular construction was challenging.
- The building code review was quite challenging when addressing a modular expansion. It was difficult to say what module type would be appropriate for the project considering the break-even points, IBC construction type requirements, economy and structural capacity of the existing structure.
- Using structural sheathing as a lateral force resisting system made it difficult to model the modules.
- Wasted modeling time trying to use a detailed module model.
- Developing the analytical model of HMAC was time consuming due to the archaic materials and assumptions that went into identifying the lateral force resistance system.
- After working with the analytical model created in this research it was realized that much of this research could have been conducted without attaching the expansion to the existing building model. A ground-mounted module approach could have produced the same results and would have allowed time for greater height expansions to be reviewed.
10.4 Future Research

Opportunity exist within the field of modular construction for future research. Initially, multi-story modular structural design using structural modules is not well documented in the U.S. and proprietary guidance appears to be mostly in the hands of modular manufacturers and their third party engineers. Opportunity exists for module lab testing on the current configurations of structural modules. The testing would help develop performance related data and design guidance as relates to U.S. Design codes and standards.

It was found in this research that application and the design process can impact feasibility of modular vertical expansion. One component missing from this literature review is a detailed review of failed projects that include prefabricated components. It would be beneficial for anyone studying the use of prefabrication in construction projects to look at these types of case studies and analyze the design process to determine why these projects ultimately fail and compile the results of the review.

As mentioned previously, there is also some opportunity for research on alternative modular philosophies and designs, both structural and non-structural. Non-structural modules show potential for expanded use in multi-story buildings of the future as replaceable components fitting into independent structural framing system. Research could be conducted on framing systems that could support this type of arrangement. Lessons from the past could be reviewed to determine why previous attempts at these types of structures have not been well received in the past.

Considering the vertical expansion component of this research, as identified previously, there is some opportunity for further research of modular expansions of greater height. Although the design process was design for a one- and two-story expansion, the process could potentially be reviewed for applicability to expansions of greater height. The shell method of preliminary analysis should be applicable to expansions of greater height. Wall and floor panel shell elements could be redefined to increase accuracy and constructed on the base level of a model versus on top of an existing structure. Both the panels and beam elements could also be meshed further to increase analysis accuracy at connections and improve confidence in stress distributions at those locations. This would allow for review and commentary on those critical junctions. Lastly, opportunity exists to evaluate the influence of seismic loading on these types of modular structures.
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Appendix A

MOD’s and Core Design
Concept #1: MOD’s Defined, Mobile Homes Redefined

What’s a MOD and why do we need them?

A MOD is a next generation mobile home unit. The units are able to be relocated by means of personal or commercial trailering and can be inserted in a structural framing system similar to the one shown in Figure 1 or used in a stand-alone setting as depicted in Figure 2. MOD’s are owned by the occupants and provide an affordable housing alternative that lends itself to the continuously changing habits and lifestyle of a young adult. The MOD travels with the occupant providing semi-permanent housing during the transient stages of early adulthood.

Figure 1. MOD inserted into leasable bay

A few trends were identified in the state-of-the-art review of multi-story modular high rise buildings that might indicate that the concept of “MOD’s” may have merit and could be a marketable product in the near future. Census data indicates that the “One Person household” demographic group is growing as well as the urban population. The Pew research report, “The Lost Decade” discusses the degeneration of the middle class in the United States and the drastic need for financial relief in this group. Additionally, recent news articles describe trends toward “Micro Apartments” in larger cities like San Francisco and New York City. Finally, there is the current demand for energy efficient buildings and the sluggish U.S. economy. These trends
combined, might suggest that a well-planned modular solution for affordable housing in urban areas might be popular and effective with the younger singles / couples demographic group.

It can be surmised that the youth of America have become increasingly more transient in nature largely because of the push for higher education in our society and the popular career centered lifestyle. It is typical for young adults to pursue college directly after high school. Credible universities are scattered throughout the world. Many times youths relocate to particular universities that offer programs that interest them, often far from their original homes and family. Upon completion of university studies the student must find suitable employment in his or her field. Once again the student must relocate to follow the work.

Many businesses and organizations are centered on the major cities of the United States so it follows that many of the current jobs are centered on those same cities. Additionally, it’s not common and many times not encouraged by mentors in professional fields to remain in your first job for more than a few years. Many people find themselves moving soon after they get settled in to a place, often to completely different city. People of modern day society can change residence for much of their early adulthood before actually settling down in a permanent dwelling. This is why having a MOD, something that can travel with you and become familiar to you over the years would be comforting and convenient throughout ones early adulthood.

**Thoughts on the Current Mobile Home Design**

Factory built housing is susceptible to an inherent flaw found in the industrialization process. Many see the benefits of assembly line construction such as those the auto industry enjoys. The benefits of factory built products are indisputable. The problem with moving the production of homes to a factory is that the end user needs a higher level of customization in the product to achieve satisfaction. Cars can be produced without an end user in mind because the end users do not typically live in their cars. The concept of a “car” is that of a tool, an object used to complete a task. A home on the other hand is not a tool, it is an “environment” and there are not many species on this planet that require the specialization of environment that humans need to survive and maintain wellness.

The current design of Mobile Homes is an example of this flaw manifesting itself. Mobile homes are typically not custom designed for a specific individual. They are designed for a typical resident and factory mass-produced for that same typical resident. This does not allow for much customization by the end user. Additionally, a modern mobile home tends to be constructed like a slightly leaner site-built home on wheels. This is also an issue because a mobile home has a completely different set of design requirements that needs to be met to ensure
customer satisfaction. Lastly, the current mobile home model is not incredibly attractive. A big contributor to the unsightliness is the integral chassis / transportation system and the crude methods used to place the temporary homes. These few things said, there is a lot of room for improvement in the mobile home of the future.

The MOD’s Concept, Mobile Homes in a Truer Sense

The MOD is a true open building solution with the “infill” component of the philosophy being the MOD. The design and purchase of the MOD is completely separated from the construction of the “structural frame” portion of the building. Those that construct the structural frame should not necessarily be those that produce the MOD unit. The MOD unit could be produced by any manufacturer that follows some set standards. MOD’s are a simple concept, but can completely redefine the way transient housing is currently approached. The following is a list of goal for MOD’s.

- Provide a semi-permanent, occupant owned affordable housing option that can be used in a structural framing system or in a stand-alone setting.
- Provide energy-efficient, high-performance, recyclable, Low-maintenance, self-healing housing units.
- Increase the wellness of occupant through customizable interiors and floor plans.
- Provide developers with a profitable and popular modern housing option to market.

MOD is the term used to describe the mobile unit component of the design. The Mod, as illustrated in Figure 2, is a weatherproof, stand-alone mobile home unit which can be inserted into a structural framing system or used as a stand-alone unit placed on a dependable surface. The MOD can be removed from the structural framing system and relocated to a different structural framing system (or standalone setup) any time the occupant changes locations.

Figure 2. The simple conceptual Mod

The general concept is that the potential occupant will purchase a MOD from a participating dealership or potentially directly from the manufacturing facility. The end-user
directly interfaces with the dealer/manufacture to customize the unit to their needs. The customer can either pick a stock floor plan or completely customize the interior environment. There will be a simple graphical drafting interface or possibly an internet based tool where the customer can arrange floor plan elements and pick finishes. The customer can spend as much or as little time as he or she chooses to define their future environment.

The level of customization of the MOD’s, because of their small size, should not affect the fabrication cost excessively; choices of interior finishes and mechanical upgrades will however affect the cost. With the availability of many different personal computing devices and the improving BIM modeling capabilities within the AEC community a customer should be able to effectively communicate their input to the manufacturer. The idea of custom modular is essential to the success of the MOD’s idea. The end user should have almost complete control over the floor plan of the mobile home. The MOD’s will have interior floor areas of between 300 ft\(^2\) and 400 ft\(^2\) (Figure 3).

![Figure 3. Potential MOD floor plan dimensional options](image)

**Foundational Concepts**

MOD’s are based on the idea that a young occupant will purchase a modestly priced unit when leaving home for the first time, which gives the occupant an immediate asset. It is assumed that the youth will be transient for some time before settling down in their permanent residence.
The individual takes the MOD to college then to their first job, then to their next job, then back to college and so on and so forth. The MOD eliminates the constant packing and unpacking associated with the moving process. Your home goes with you. Something familiar to take on your journey to a new location. There will be no hassle of looking for apartments, paying expensive closing costs on condominiums or town homes, or arranging the connection and disconnection of utilities. When the user needs to change locations they arrange for the unit to be removed. The owners can trailer the unit themselves and take it with them if they have the ability, or call up a mover, tell them were to take it; get on a plane and walk back into their own home in a totally different city. The MOD will travel with the individual for the span of their transient period and then will be used as a part of their permanent residence, sold, passed on to children or recycled. Figure 4 shows pictorially the life cycle of a MOD.

![MOD Life cycle](image)

**MOD’s Structural Framing Systems**

The idea is that interested developers will independently construct structural frames in urban areas in anticipation of transient users. The building shown in Figures 5 and 6 show one conceptual way to house the MOD units. These frames do not necessarily have to be completely dedicated to the MOD’s concept. It should be possible to construct hybrid structures where part of the building is dedicated to commercial space, public space, storage, or permanent housing units. The structural system could include leasable amenities such as rooftop gardens (Figure 6a) to provide attractive incentives to the occupants. The owner of the MOD’s will lease a bay from the developer.
Figure 5. Example of a MOD independent concrete structural frame. The MOD units are inserted into a vacant bay. Bays not being used will have overhead doors to close openings. Valuable 1st Floor area can be used as leasable commercial tenant space.
Figure 6. Close up view of the interior of a typical bay. Notice the steel guide rails mounted on the beams defining the bay. Overhead doors (Not Shown) should be installed in the bays as well as weather seal around the perimeter of the exterior and interior openings (Not Shown).

Figure 6a. The possibility exists to incorporate leasable garden space or greenhouses on the roofs of these structures. Alternatively the excellent solar access on the roof provides an opportunity for power generating elements such as solar panels or algae farms.
Utilities

The structural frames will have connections for utilities with individual metering capabilities and the ability to insert the unit and remove the unit when needed. Figure 7 shows a possible floor plan arrangement. The units are arranged around a structural core.

![Figure 7. Example layout of a typical floor plan. MOD's are arranged around a structural core. Utilities in this layout are routed between the units with ladder access between stories. The connection for the utilities will be on the sides of the units.](image)

MOD Delivery System

Eventually, delivery should be able to be accomplished by a piloted drone flight vehicle or some other type of floating air craft, but until technology advances a bit a mechanical system is required. Figures 8 and 9 illustrate one type of possible mechanical delivery system. This system takes advantage of a cantilevered lift system and would be appropriate, safe, quick, and easy to use for removal and extraction of the units from the structural framing bay. The concrete columns in this example have steel sleeves cast-in-place with the columns (Figure 10 and 11). The sleeves have a gear teeth and a guide, in which four motorized gears are used to lift the MOD to the appropriate bay. Retractable pins are used to hold the lift system in place while in service. When the lift system arrives at its destination the unit will be inserted into the bay by a two position pulley system (Not shown, but will use a rotating arm that can be placed in either the insert of remove position).
Figure 8. The lift system raises the MOD unit to the appropriate bay.

Figure 9. The unit is inserted onto a rail system. The final attachments, utility connections and weather sealing measures are then completed inside the building.
Figure 10. Front elevation showing the concrete columns with embedded gear guide assembly.

Figure 11. DO1, close up detail of the conceptual drive and guide for MOD lift system. Inserts will require additional column anchorage (Not Shown).

Stand-Alone Application

The MOD’s will be self-contained units with all services integral to the unit. The units will not have a chassis system like current mobile homes. The frame will be lightweight (plastic or light gauge steel, resource efficient and recyclable). The owner of the MOD can choose to use the unit on a private lot or even a park setting with utility connections similar to current mobile home parks. The foundation system for the MOD will be integral to the unit. The unit will have the ability to be directly set on gravel, sand bed or similar relatively level and reliable pad.
Ideally, the concept of a mobile home park should be revamped as well. Mobile home parks currently are not really set up for “mobile” homes. The homes that are typically located on the park lots usually do not move once their set. If they do it is not more than once.

The term mobile home currently has stigma attached to the term. People usually associate low quality homes and unattractive settings with mobile homes and mobile home parks. The concept surely needs to be redefined to incorporate a more transient nature to the park and a cleaner modern look to the park as a whole to attract a younger crowd. Some elements such as personal garden space or storage space might be incorporated to provide the transient tenant a feeling of a more permanent home setting. A revamped rural mobile home park would be an excellent companion to the urban steel frame environment, giving the occupant their choice of lifestyle; country mouse or city mouse!

**Trailering System and Other Items**

The trailering system will be independent of the unit and can be used with personal vehicles or commercial transportation vehicles. Standardization of the dimensions of the units will be required to ensure that the units will fit both on the trailering system and in the various structural framing systems. No further thoughts on this at this point. This will however need to be developed along with some other items. The following items are a list of items that require design development:

- Graphical interface for effective user interior design.
- Unit lightweight plastic or light-gauge steel structural design with foundation, a new foundation system should be developed such that the need for cumbersome piers or extensive ground preparation should be eliminated. The unit should be able to be mounted on a gravel or sand bed when used in the stand alone application.
- High performance building envelope design:
  - Self-Healing possibly extruded or cast walls containing light-weight thin possibly Aerogel insulation.
  - Advanced Glazing:
    - Solar energy harvesting glazing
    - Glazing that contains computer or television screens to save floor space
- Trailer for units. The trailer should not be hard to develop. It simply needs a method of fastening the unit to the bed of the trailer. It could be hooks and ratchets or a screw type fastener. The base of the unit will have to be relatively plane do to the need for the
ability to be inserted in square to rectangle bay, coupled with the need for direct gravel or sand bed mounting.

- Rail delivery system for units.
- Steel Frame bay receiving system
  - Mechanical
  - Structural
- Utility delivery system for the superstructure.
- High-efficiency mechanical system design for unit:
  - Possibly microbial Fuel Cells combined with solar harvesting glazing.
Concept #2: Residential Utility Cores

Description of the Core

A residential core will be a pre-manufactured product that will serve as utility core to house the majority of the residences utility services. The main benefits of the core is energy, cost and labor savings due to the compact arrangement of the utilities and the factory built aspect of the design. The cores can have single to three-story arrangements depending on the needs of the project. Most likely, the cores will require independent foundation that would be installed much like that of an elevator core in commercial projects. The residence will be constructed around the installed core. The core can provide stability to the structure as well an interior load bearing point. The following is list of key points and benefits associated with the concept.

1. Solves many problems in home design
   a. Multi-level circulation
      i. Stairs are one of the most difficult items to place efficiently in a floor plan. They are also one of the first items to be considered.
      ii. Using wide stairs with shallower risers allows for easy replacement of equipment as well as making it easier to go up and down stairs.
   b. Utility Routing and placement
   c. Venting
      i. Radon
      ii. Chimney’s
      iii. Kitchen Exhaust
      iv. Bath Fan
      v. Ventilation and Combustion Air

2. Advantageous for passive design
   a. Thermal mass
      i. Avoid adding thermal mass in the floor
         1. Temperatures make walking on uncomfortable without in floor heating system.
         2. Concrete is hard and generally uncomfortable to walk on.
         3. Concrete can cause greater injury to occupants if they fall.
      ii. Can circulate ducts through mass
         1. Helpful for passive heating and cooling
            a. Can conduct saved energy to integral ductwork
iii. Thermal mass can be added adjacent the core in the form of vegetation
   1. Non-pollinating
      a. i.e. ferns or grasses
   2. Good for indoor air quality as well
      a. Air filtration and oxygen production

3. Bath rooms can be located in the core or adjacent the core
   a. Short plumbing runs
   b. Stacked bathrooms

4. MEP Service can be branched from the core
   a. Almost like a tree distributes nutrients to its individual components.
   b. Eliminate need for additional shafts in homes
      i. Improved safety
      ii. Reduced air sealing details
   c. Waste Heat stored in thermal mass of the core
   d. Reduce the need for many specialized contractors on site or at a minimum limits
      their time spent on the project. Much of the complicated routing and installation
      can be handled at the factory

5. Media/Server/Home Automation can be centralized in the core

6. Kitchen and Laundry Equipment can be integral to the core or adjacent the core
   a. This allows for easy venting and recycled heat from appliances

Sample Cores

Cores can be arranged in any manner, as long as the design is efficient and feasible in
terms of operation and equipment servicing. The exterior walls can be made from a variety of
materials such as concrete, cold-formed steel or wood. The following images show a few
examples of potential floor plans.
Figure 1. 1st floor of a conceptual two-story core. This particular core was designed for a home that would have a crawlspace or slab-on-ground foundation.

Figure 2. 2nd floor of a conceptual two-story core.
Figure 3. An example of the 1st floor of larger core which incorporates the laundry facilities.
Figure 4. An example of how the smaller core shown in Figure 1 could be incorporated into the first floor plan of a home.
Appendix B

Interviews with Professionals
Interviews

Q = Question
A = Answer
C = Commentary by interviewee

Interview #1: Architect (Principal), State College, Pa

Q: What type of construction would you recommend to a client asking you to design a mid to high-rise affordable apartment building or student housing complex?
A: Considering 4-7 story buildings.
   • Masonry with pre-stressed or hollow-core floor system (Block and Plank)
     - Cost effective, easy to finish interior and route utilities
   • 2D panelized wall systems
     - Cost effective and fast, cladding already installed
     - More expensive than a metal building, but a lot faster

Q: In your initial discussion with the same client why didn’t you recommend a modular solution?
A: Don’t see a product around.
   • Modular construction in this area is primarily single family and townhomes, sometimes maybe two-story buildings.

Q: If the same client suggested initially that he was considering a modular solution, what would your response be and would you be positive about it?
A: I would be open to it and I think it’s a great idea for some buildings (time and work savings), but I would have to compare costs to other systems.
   • The construction industry in the State College area is not very progressive. There is some resistance to alternative construction methods.
   • If you can combine a cladding system, that can be quickly installed, with a modular solution it would be an added benefit.
   • The time savings would be a big benefit in this area and would help with problems such as site traffic.
We would have to consider the aesthetics and mechanical routing.

Q: Have you been involved in any multi-story modular construction projects in the past.
A: No, just single family and townhomes.

Q: Are you currently using BIM and do you plan on using it the future?
A: No I’m not currently using it. It’s not cost-effective for the projects that I’m involved with. It’s quicker to use my current 2-D drafting system.

- The only people I see using it are the larger architecture firms.
- Some larger projects require it.
- I think it’s a great idea and definitely the wave of the future but I see it taking a long time to become integrated.
- Interdisciplinary Communication is going to be a big hurdle; some local designers are just now adopting CAD drafting systems. We currently have problems sharing 2-D Autocad files between disciplines now, I imagine the sharing of 3-D model components might bring additional complications.

Q: Do you currently have a relationship with any modular manufacturers?
A: No, the single family/townhome projects were ordered through my local lumber yard.

Interview #2: Structural Engineer (Principal), State College, PA

C: Unions (i.e. IBEW, plumbers union, etc.) are a big opponent of modular technology. Modular factory jobs take jobs away from field contractors.

- Example: A local manufacturer was producing non-structural infill bathroom modules for high-rise buildings. They were doing fine for about 4-6 years. They ran into some union problems and you never heard about them again.

C: Overall I think the concept of modularization is good; good quality control and superior comfort in factory setting. The problem with incorporating modular is that it’s very hard to change the system. Contractors and designers are comfortable with what they are doing now.

C: About 20-30 years ago we were using modular construction. We ran into some problems with dimensional stability and fitting the units together properly. They were all made of dimensional
lumber. The instability of wood was probably the issue. I imagine the light gauge steel units are superior and more stable.

**C:** Another problem I see with modular construction is that the modular units are typically constructed with 6-sides. That means that there are repetitive structural subsystems on some of the sides (i.e. Floor assembly on top of bottom unit ceiling assembly, wall next to neighboring unit wall.). It seems some economy is lost here.

**C:** I have seen problems maintaining the integrity of protective building elements (i.e. fire-wall, fire-barrier, etc.). There might be difficulty constructing or maintaining the continuity of fire-resistive elements once the units are set. I have seen some units being shipped with drywall attached to the outside of the units for this purpose.

**Q:** What types of structures do you see modular construction methods being used for in this area?

**A:** Mostly single-family dwellings and townhomes. Most of the modular construction in the locale is accomplished by the same companies that produce mobile homes. They have the facilities so they are the only one producing modular units.

**Q:** Consider the following occupancies. Can you tell me what structural system you would recommend and why?

1.) Mid-rise, affordable housing complex?
   - **A:** For a four-story building I would recommend wood or light gauge steel platform framing. It is the most economic and the contractors are familiar with the construction methods.

2.) High-rise, affordable housing complex?
   - **A:** Masonry bearing walls and precast hollow core floor system. Most economical, very durable finish, meets code separation requirements. Primarily code compliance drives structural system selection.

3.) Mid-rise student housing?
   - **A:** Same answer as first question number.

4.) High-rise student housing?
   - **A:** Same answer as second question number.

**Q:** Are you using BIM for any of your projects? AutoDesk Revit or something similar?

**A:** No, still 2-D drafting. I’m just now using AutoCAD on a regular basis. I haven’t had the need to draft in the past. I normally would do the design work and the drafters would take care of the drafting.
Interview #3: Structural Engineer (project manager), New York City, NY

C: We’ve conducted some studies regarding the use of multi-story modular methods in the past, but we’ve never actually used it on a project yet. We conducted a cost comparison for a student dormitory project between precast concrete and modular construction. The modular construction option actually was less economical.

C: Contractors in NY City are resistant to change. They prefer to use methods that are familiar to them.

C: Precast, cast-in-place concrete or masonry construction is used for most multi-family buildings in the city. Owners typically do not come into a residential project with the mindset that they are going to modularize a project. I can see volumetric being an option for dormitory or affordable housing uses.

C: One of the challenges with modularizing the residential buildings (both mid and high rise buildings) in NY is that most of the structures, for aesthetic reasons, have moving or walking columns (not directly stacked, staggered between floors). This could be a problem for attempting a modular stacking strategy.

Q: Does your company use a BIM based drafting system?

A: We use both. It tends to be more economical to use 2-D AutoCAD for small renovations. Most or our large renovations and new construction projects are drafted in Revit. Some projects it works out for, others it’s not worth it. Designing in 3-D is time consuming and very detail orientated. You have to evaluate whether the project will benefit from the increase workload. We have some in-house issues with 3D drafting. We have a separate drafting department and design department. Some of drafters are excellent in 2-D AutoCAD but have some problems switching to 3-D modeling. BIM is still in a transitional phase.

Q: Do you have difficulty with sharing your REVIT drawings with other professionals (i.e. Architects, MEP designers, Contractors)?

A: Most Architects in our area are on board with BIM. Mechanical contractor seem to have some difficulty with BIM modeling.

Interview #4: Prior Architectural engineer, university instructor, and current director of regional code office, State College, PA

C: Some IBC Notes concerning the construction of apartment complexes and student housing.

- All residential buildings must have automatic sprinklers.
• 4 stories or above with require a minimum of Type 5A construction. 5A construction can be difficult to constructed due to the need for protection of wood framing elements.
• There is a specific definition in the IBC for High-Rise. Around 75’ above grade story. Something like that, verify in IBC.
• Structures greater than 5 stories will most likely be of type I construction.

Q: What type of structural system would you recommend to a client that wants to construct an 4-story affordable apartment building or student housing complex?
A: My first question would be are you going to maintain ownership of the building or are you going to flip it?
• I would consider wood or light gauge steel platform framing. 5A or typeII construction type might be economical if the person was going to flip it. It probably would be the cheapest type of construction.
• If the person was going to keep the building I would recommend considering a block and plank system or a pre-cast wall and hollow-core slab system. It provides a more durable building and would be similar in cost to the platform framing option. Block and plank would provide a type I building.

Q: What about a 10 story building?
A: Block and Plank.

Q: Do you feel that a volumetric modular solution would be appropriate and cost competitive?
A: Yes, in some instance more than others.

• The primary benefits of modular construction are speed of construction, predictability of scheduling, deliveries and costs and minimal stockpile space requirement.
• If you have an infinite time to construct a building then a site built option will most likely be more cost-effective. If you have a tight schedule a modular solution may be competitive.

C: An example of were off-site construction can save money is prevailing wage jobs.
Prevailing wage act, Pennsylvania law, the comparable federal law is the Davis Act, which was initiated by unions to fix labor prices on non-union jobs that are publicly funded (i.e. government jobs, schools, etc.). On publicly funded jobs over $25,000 L&I sets the labor rates according to the prevailing county labor rates. Mostly pertains to non-union contractors. Union wages would track closely with the prevailing rates anyway. A prevailing wage job typically increases overall construction costs by around 33%.
• The important thing to note is that prevailing wage jobs only regulate the labor rates on site. All offsite labor rates are not regulated. This could offer the client considerable additional savings.

Q: If it was a union job would this benefit still be applicable?
A: Yes there’s nothing prohibiting it, but typically unions hire unions. The modular industry is not really unionized.

Q: What types of modular projects do you see in the area?
A: Primarily single-family and townhomes. There’s not a lot of factories in the area and the one that are here pretty limited to the production of single family or town-homes.

Q: One of the comments I receive in an earlier interview was that it might be difficult to construct fire barriers when using modular construction methods. Do you feel that this is true?
Q: I don’t think it’s a big problem. You just have to use different methods. I’ve seen instances in town-home construction where the contractor used stand-alone separation walls between the units and stack the units on either side. Sometimes they add the protection to the outside of the units and just seal the joints when assembled.

Q: Have you used BIM in the past and do you feel that it will be important in the future?
A: No I have used mostly 2-D drafting systems. I don’t think anyone really knows the benefits of BIM at this point. It might be overkill in a lot of instances. Another problem is if everyone works from a common model than everyone might have access to everyone else’s drawing files. What are the restrictions? If you’re an MEP engineer do you really want the fabricator to have direct access to your original plans? Another issue is liability; for instance if you’re the structural engineer and the fabricator fabricates some beam connections based on one set of drawings. If the drawing are then updated and the fabricator doesn’t know about it, who’s liable? Who’s liable for the entire model?

Main Points Taken From Each Interview

Interview #1

• He considers block and plank construction as the most economical construction method for the construction of affordable multi-story housing.

• 2-D panelized wall systems have a strong presence in the local building construction market.

• He’s positive about the use of modular methods, but just doesn’t see a product.
• The local factories are only producing single-family homes and townhomes.
• No relationship with modular manufacturer.
• It’s currently not cost-effective to use BIM on his projects.
• Contractor resistance to new types of construction is heavy in the locale.

**Interview #2**

• Union resistance to off-site construction methods.
• The 6-sided nature of modular units may result in redundant wall and floor / ceiling elements and ultimately lead to material inefficiencies.
• Potential continuity problems with fire-resistive structural elements.
• Platform framed buildings are economical for low to mid-rise affordable multi-story housing and Block and plank is economical for mid to high-rise buildings.
• Contractor resistance to new types of construction is heavy in the locale.
• Positive about the use of modular methods.
• The local factories are only producing single-family homes and townhomes.
• No relationship with modular manufacturer.
• It’s currently not cost-effective to use BIM on his projects.

**Interview #3**

• His company conducted a cost comparison for a dormitory project. The modular solution was not the most cost-effective.
• Contractors in New York City are resistant to new types of construction.
• Modular construction methods have potential for use in dormitory or affordable housing projects.
• Most multi-story, multi-family projects are constructed with block and plank.
• Owners don’t think in terms of modularization, when initially conceptualizing a multi-family building.

• Walking columns can be an potential obstacle for modularizing some buildings.

• His company uses AutoDesk Revit for larger project. Most architects in the area are competent with BIM.

**Interview #4**

• All residential commercial buildings need a fire-sprinkler system. This should be considered when a modular solution.

• Wood framed volumetric modules probably only make sense for structures under 4-stories. After that typically structures are required to use a minimum of type 5A construction, which would mean fire-protection for wood structural elements. It’s likely that a non-combustible type construction would have advantages at this point.

• Would consider platform framing for low to mid-rise affordable housing and block and plank for mid-to high rise affordable housing.

• Considers volumetric modular construction a cost-effective option in some scenarios. Off-site construction can lead to cost savings in some instances.

• If a project has time constraints then modular construction becomes more cost-effective. If the project has an unlimited time frame then it’s likely that site-built methods will be more economical.

• Doesn’t see big issues with fire-resistive element continuity. The construction of these elements is just different then their site-built counterparts.

• See’s liability issues with BIM.
Appendix C

Commentary on Multi-Family Buildings, Looking Forward
Commentary on Multi-Story Buildings Looking Forward

The role of multi-story structures, primarily in urban centers, is slowly expanding and the buildings of the future will likely serve many additional duties. As a society, we have some interesting challenges to face in the not so far future that must be addressed by community planners when considering any new or rehabilitated structures. Complex environmental and social issues exist that are only being exacerbated by the increasing urban densities. The most notable of these issues are housing affordability, community development, efficient use of natural resources, air and water pollution, waste management, and the reduction of our natural areas. Buildings are a major physical man-made item in any city and can be used more effectively as tools to reduce the damage to the environment and promote increased quality of life. When considering the provision of urban housing in the future, one must examine these issues.

The most obvious issue is the limited available urban land. In older developed population centers such New York City the prohibitive cost of urban parcels makes buildable land a precious commodity and sought after by many. Population growth and a demographic shift to urban living have presented the possibility of density problems in some U.S. cities, which could eventually negatively impact the quality of life for city occupants. According to the 2010 U.S. Census 83.7% of the people living in the U.S. live in one of the nation’s 366 metropolitan areas with New York City, NY and Los Angeles, CA being the most populous. In the last decade metropolitan area population growth was nearly double that of rural areas (10.8% compared to 5.9%). Houston, TX, Atlanta, GA and Dallas-Fort Worth TX experienced growth rates of 26.1%, 24.0%, 23.4% respectively (U.S. Department of Commerce 2011). With urban growth rates increasing and little available developable land remaining in metropolitan population centers the density level of some U.S. cities may become problematic in the future without innovative building designs and strategic planning.

Innovative solutions are being proposed to solve affordable housing and urban density issues that are arising globally in super dense cities. The reality that our natural energy and land resources are finite is beginning to become apparent and planners are beginning to putting forth creative conservation based solutions. Newly developed cities such as Singapore have acknowledged the possibility of urban overpopulation and have promoted smart growth concepts to address density issues, in advance. The urban land institute (ULI) suggests the following principles for smart urban growth from lessons learned

1. Plan for long-term growth and renewal
2. Embrace diversity, foster inclusiveness
3. Draw nature closer to people
4. Develop affordable, mixed-use neighborhoods
5. Make public spaces work harder
6. Prioritize green transport and building options
7. Relieve density with variety and add green boundaries
8. Activate spaces for greater safety
9. Promote innovative and non-conventional solutions
10. Forge “3P” (people, public, private) partnership

Another example of a country trying to think ahead and plan for future urban sustainability is China. China is currently developing at a very fast pace. China has unique opportunities due to the land apportionment relative to the population and the relatively infancy of its undeveloped economy. There are large dense population centers like Beijing but there are also large tracts of undeveloped farmland in the rural areas. Forward thinking Chinese planners are able to think in terms of the development of new cities as well as addressing issues in existing populous cities. An example is the “carless city” near Chengdu conceptual plan, recently unveiled by Chicago-based architects Adrian Smith and Gordon Gill (“China Plans New Car-Free City” 2013).

Because China is a developing nation the opportunity exists to choose the location of development. When afforded the opportunity to develop brand new cities, many development problems existing today can be avoided up front. Unlike China, The U.S. is a developed nation and must address challenges that differ from those of developing nations. In highly developed countries, such as the U.S. planning new cities is not impossible, but is unlikely to be a realistic solution in the short term to sustainability issues. U.S. Cities such as New York have aging infrastructure and buildings that sprawl over large areas. Wide spread sustainable concepts are much more difficult to implement due to the established architecture of the cities. New ideas must be pieced in as the opportunities arise. When land or defunct buildings become available it might be possible to construct new buildings or renovate or recycle existing ones. In general, it is likely that the development of sustainable cities in the U.S. will take considerably more time than those being envisioned in developing nations. The U.S. should however have opportunity in the future to enhance the urban buildings.
When large cities already exist, such as Beijing, space is a commodity, and different approaches to solving urbanization based issues must be considered. One solution, shown in Figure 1, proposed by the architecture firm ANDO|Andalucia is to construct cities in an upward direction. If you think about this it makes perfect sense. The amount of vertical space available for a given fixed price parcel is only limited by our engineering knowledge, zoning and flight restrictions. This project is dubbed “The Mutant Vertical City” because throughout the height of the building varying components of a city can be placed. The design is based on an “Open Building” concept. The modular components can be rearranged to meet the needs of the occupants at any given time. Modules can be arranged to form communities of housing units as well as provide for commercial or public areas. A social advantage of this design is the ability to form communities. Most modern high rise accommodations are relatively isolated from one another. The designs do not facilitate the need for neighbor relations or communal activities, which are essential to the well-being of occupants. The ability to arrange modules in different configurations can encourage occupants to form communities and participate in public activities without traveling past the confines of their own building.

Figure 1. 150 Story “Mutant Vertical City” Proposed as a solution to urban density issues in Beijing China (Image by DesignBuild Source, 2012).

Sustainable design and quality of life are quickly becoming important design criteria when planning the construction of new buildings. The U.S. urban public is beginning to demand open space for recreation and relaxation, pollution free air, and reduced energy bills and housing costs, when searching for appropriate housing. These are heavy demands, considering some of the previously discussed issues, but many creative architects and planners are putting forth unique designs aimed at addressing these demands.
One particular concept that is beginning to take hold is the idea of incorporating vegetative element in buildings. Notice the extensive plant life included in the façade of the Park Royal hotel shown in Figure 2a. Building like these are rare, but may offer a glimpse into what we may see in the future. The vegetative features undoubtedly add to the aesthetic appearance of the building and helps blend the structure into the surrounding environment. The addition of greenery not only serves to enhance the aesthetic appeal of the building but also can perform functions such as air filtration, oxygen production, energy production, food production, shading, micro-climate regulation, and building insulation. The benefits are quite extensive, but the challenges are also quite extensive. The idea of incorporating living, dynamic elements in a supposed static building can create many design considerations. The building in Figure 2b shows a good starting point for a transition toward vegetative buildings. Urban architects are currently making better use of valuable roof space. Many roofs in the city are ideally located in regards to solar access. The greenhouse located on the roof is an efficient use of the mostly vacant roof area. Greenhouses can provide a pleasant communal space for occupants as well as an opportunity for income generation and food production. The idea of vegetative structures is just one of many innovative concepts that are being explored currently by the design community.

![Figure 2](Image by Inhabit Blog; creative commons, https://creativecommons.org/licenses/by-nc-nd/2.0/legalcode.) (b) Arbor House apartment building in Bronx, NY shows off impressive rooftop greenhouse (Image New York City Housing Authority; creative commons)

Many architects and engineers are reforming their concepts about the residential housing of the future. New concepts are being developed especially in urban planning, resulting from the wave of technological change and discoveries that has hit the building construction industry in recent decades. The engineering and consulting firm ARUP has an excellent compilation of the technologies that might be incorporated into the buildings of the future. The firm has been conducting research for the past 11 years monitoring trends that might affect the building industry. The research team has expertise in
sociology, biology, psychology, economics and more (“ASCE’s Civil Engineering Magazine” 2013). In a recent publication (“It’s Alive,” 2013.) the research team assembled all the technology they thought might be available by the year 2050 and developed a proto-type, shown in Figure 3 of what they thought the skyscraper of the future might look like.

Figure 3. Engineering and Consulting firm ARUP’s vision of the skyscraper of the future (image by ARUP)
At first this structure is a bit overwhelming to look at and may seem entirely unbelievable, but upon further inspection and investigation, it can be recognized that many of the technologies, depicted in this building, are currently being researched and developed. A hypothetical building of this caliber can begin to address many of the issues previously discussed and is truly aimed at meeting the needs of the occupants and preserving the environment.

Principle 10 of the ULI document suggests that a partnership between the people, public and private be established in order to promote smart growth. It’s easy to see how a relationship of this sort can develop and become important when considering the construction of these futuristic buildings. Some of the features integral to this building would be difficult for a private owner to afford on their own. It would be more rational to consider a consortium between private industry and local government agencies. It is reasonable to assume that the public sector would be interested and likely to support projects like this. The entire burden of the building construction should not be placed squarely on the shoulders of private industry. A building of this sort has direct public elements such as public transportation systems and pedestrian walkways. The building also provides environmental benefits such as air filtration, oxygen production and reduced, if not eliminated energy requirements. The point being is that it’s hard to envision structures of this type without public involvement and community involvement. The likely scenario is that a building of this sort will actually have three owners. Certain components will be owned by a private investor, where as other element will be owned by a government agency and yet other components might be actually owned by the occupants. This is completely different than our current building ownership schema.

In summary the structures shown in this section are examples of how some designers and planners are becoming pro-active and attempting to consider solutions to changing conditions on our planet. The issues, we as designers and planners face in the future are very real and difficult to solve. The inevitable U.S. population increases, depletion of fossil fuel based energy sources and potential climate change repercussions are difficult to accept as reality, but if ignored can put citizens in danger and the U.S. in a difficult position in the upcoming decades. Building design philosophies of the past may not be applicable for the future. Solutions exist, but need research to bring some of the recent promising technological advances in the world to the construction market. The U.S. has been a world leader over the last century in building construction. We now, as a nation face falling behind some of the more forward thinking countries in this market if we don’t soon establish a firm vision of the future of our built environment and take bold steps toward making it a reality.
Relevance to Modular Construction

How does this all relate to modular construction? The answer is that modular construction is a very important part of this vision. If you look at Figure 3 you’ll see that it has many modular components. The concept of living buildings is difficult to imagine without modular components. The buildings will need to be fluid and accept modification as needs of the occupants and tenants change. Modular construction is a very efficient form of construction and has all the benefits discussed in the introduction and in the literature review portion of this document. Modular construction methods are extremely efficient when considering repetitive component, while traditional site built construction methods are advantageous when constructing customized, irregular shaped structures or elements.

It’s not rational to think that the buildings of the future will be completely modular; however it is naive to think that traditional site built construction methods will produce more economical and higher quality building components that are of a repetitive nature or require the ability to be relocated. It is equivalent to thinking, that when somebody wants to purchase a car, it would it be less efficient to produce that car in a factory and more efficient for the workers to go to that persons house and custom build the vehicle.

Modular construction methods, though not ideal for all construction projects, do have their place in both the current building construction market and more importantly the, not so distant, future building construction market. The current available methods can be useful for improving the productivity of modern construction, while at the same time reducing community disturbance, improving affordability and enhancing the quality of the components. Modular components will most likely be necessary in the buildings of the future. The concepts are not new, however much of the construction industry is most likely unaware of the possibilities that exist with modular construction methods, and there is some warranted social resistance to its use presently. The use of these methods for appropriate applications should improve productivity in the U.S. construction industry, but the process of identifying the proper applications, integrating the technology in the manufacturing sector and convincing the construction industry of its merits will be time consuming. Strategic planning is necessary to ensure the smart integration of this promising technology and reduce the risks of a failed integration attempt.
References


“It’s Alive.” n.d.

Appendix D

Module Weight Estimates
Calculation of Module Estimated Weights

\[ L := 26.45 \text{ ft} \quad \quad W := 13.67 \text{ ft} \quad \quad A := L \cdot W = 362 \text{ ft}^2 \]

Cornerpost Bearing Modules

- Calculated weights consider only the structural members and interior gypsum.

Floor System:

- Perimeter beam (W10x15): \( w_1 := 15 \frac{\text{lbf}}{\text{ft}} \)
  - \( W_{pb} := w_1 \cdot 2 \left( L + W \right) = 1204 \text{ lbf} \)
- Floor Joists (W8x10): \( w_2 := 10 \frac{\text{lbf}}{\text{ft}} \)
  - \( W_j := w_2 \cdot 5 \cdot W = 684 \text{ lbf} \)
- Lightweight Concrete Slab: \( \gamma_{lwc} := 100 \frac{\text{lbf}}{\text{ft}^3}, t_{slab} := 3 \text{ in} \)
  - \( p_{slab} := \gamma_{lwc} \cdot t_{slab} + 1 \text{ psf} = 26 \text{ psf} \) (1 psf for shear studs)
  - \( p_{floor} := \frac{W_{pb}}{A} + \frac{W_j}{A} + p_{slab} = 31.2 \text{ psf} \)

Wall System:

- Corner Posts and Intermediate Posts (HSS 4 x 4 x 3/8): \( w_3 := 17.27 \frac{\text{lbf}}{\text{ft}}, h_i := 10.83 \text{ ft} \)
  - \( W_{cp} := w_3 \cdot 6 \cdot h_i = 1122 \text{ lbf} \)
- Wall Studs (400S200-33 @ 24” O.C): \( w_4 := 1.05 \frac{\text{lbf}}{\text{ft}}, h_2 := 10 \text{ ft} \)
  - \( W_s := w_4 \cdot 66 \cdot h_2 = 693 \text{ lbf} \)
- Top and Bottom Track:
  - \( W_t := w_4 \cdot (4 \cdot W + 4 \cdot L) = 169 \text{ lbf} \)
- Interior and Exterior Gypsum Wallboard/sheathing: \( t_{gyp} := \frac{5}{8} \text{ in}, \rho_{gyp} := 5 \cdot 0.55 \text{ psf} = 2.75 \text{ psf} \)
Calculation of Module Estimated Weights

- \( W_{gyp} := p_{gyp} \cdot 4 \left( h_2 \cdot L + h_2 \cdot W \right) = 4413 \text{ lbf} \)

- \( p_{wall} := \frac{W_{cp} + W_s + W_t + W_{gyp}}{(L \cdot W)} = 17.7 \text{ psf} \)

Ceiling System:

- Ceiling Joists (600S162-33 @ 24" O.C.): \( w_5 := 1.17 \frac{\text{lbf}}{\text{ft}} \)
  \( W_{cj} := w_5 \cdot 19 \cdot W = 304 \text{ lbf} \)

- Steel Angle (L 2.75x6x16 gage): \( w_6 := 1.05 \frac{\text{lbf}}{\text{ft}} \)
  \( W_{sa} := w_6 \cdot 2 \cdot (W + L) = 84 \text{ lbf} \)

- Interior and Exterior Gypsum Wallboard/sheathing: \( t_{gyp1} := \frac{5}{8} \text{ in} \), \( t_{gyp2} := \frac{1}{2} \text{ in} \)
  \( p_{gyp1} := 5 \cdot 0.55 \text{ psf} = 2.75 \text{ psf} \), \( p_{gyp2} := 4 \cdot 0.55 \text{ psf} = 2.2 \text{ psf} \)
  \( W_{gyp1} := (2 \cdot p_{gyp1} \cdot L \cdot W) + (p_{gyp2} \cdot L \cdot W) = 2784 \text{ lbf} \)
  \( W_{gyp2} := \left( \frac{W_{cj} + W_{sa} + W_{gyp2}}{L \cdot W} \right) = 8.8 \text{ psf} \)

Total Weight of Corner Bearing Module:

- \( p_{total} := p_{floor} + p_{wall} + p_{ceiling} = 57.7 \text{ psf} \)
Calculation of Module Estimated Weights

Cold-Formed Steel Wall Bearing Modules

Floor Cassette:

- 3/4" Gypcrete: \( p_1 := 7.2 \text{ psf} \)
- 3/4" OSB Flooring: \( p_2 := 2.5 \text{ psf} \)
- Floor Joists @ 16" O.C. (800S162-54): \( w_1 := 2.28 \frac{\text{lbf}}{\text{ft}} \)
  
  \[ W_{fj} := w_1 \cdot 21 \cdot W = 655 \text{ lbf} \]

- Top and Bottom Track (800T200-54): \( w_2 := 2.31 \frac{\text{lbf}}{\text{ft}} \)
  
  \[ W_t := w_2 \cdot 2 \cdot L = 122 \text{ lbf} \]

\[ p_{\text{floor}} := \frac{(W_{fj} + W_t)}{L \cdot W} + p_1 + p_2 = 11.8 \text{ psf} \]

Wall Panels:

- Wall Studs @ 16" O.C. (400S200-54): \( w_3 := 1.70 \frac{\text{lbf}}{\text{ft}} \), \( h_1 := 10 \text{ ft} \)
  
  \[ W_{\text{stud}} := w_3 \cdot h_1 \cdot 2 \cdot 35 = 1190 \text{ lbf} \]

- Top and Bottom Track:
  
  \[ W_t := w_3 \cdot 2 \cdot (L + W) = 273 \text{ lbf} \]

\[ p_{\text{wall}} := \frac{(W_{\text{stud}} + W_t + W_{\text{gyp}})}{L \cdot W} = 16.3 \text{ psf} \]

Total Weight of Cold-Formed Steel Wall Bearing Module:

\[ p_{\text{total}} := p_{\text{floor}} + p_{\text{wall}} + p_{\text{ceiling}} = 36.9 \text{ psf} \]
Calculation of Module Estimated Weights

Wood-Framed Wall Bearing Modules

- Assume SPF #1, #2
- $\gamma_{\text{wood}} \approx 27 \text{ pcf}$
- $w_{2\times4} := (1.5 \text{ in} \cdot 3.5 \text{ in}) \cdot \gamma_{\text{wood}} = 0.98 \frac{\text{lbf}}{\text{ft}}$
- $w_{2\times6} := (1.5 \text{ in} \cdot 5.5 \text{ in}) \cdot \gamma_{\text{wood}} = 1.55 \frac{\text{lbf}}{\text{ft}}$
- $w_{2\times8} := (1.5 \text{ in} \cdot 7.25 \text{ in}) \cdot \gamma_{\text{wood}} = 2.04 \frac{\text{lbf}}{\text{ft}}$
- $w_{2\times10} := (1.5 \text{ in} \cdot 10.25 \text{ in}) \cdot \gamma_{\text{wood}} = 2.88 \frac{\text{lbf}}{\text{ft}}$

Floor Assembly:

- 3/4” Gypcrete: $p_1 := 7.2 \text{ psf}$
- 3/4” OSB Flooring: $p_2 := 2.5 \text{ psf}$
- Floor Joists @ 16” O.C. (2x10):
  - $W_{fj} := w_{2\times10} \cdot 21 \cdot W = 828 \text{ lbf}$

- Double Rim Board (2-2x10):
  - $W_{\text{rim}} := w_{2\times10} \cdot (4 \cdot L + 2 \cdot W) = 384 \text{ lbf}$
  - $p_{\text{floor}} := \frac{(W_{fj} + W_{\text{rim}})}{L \cdot W} + p_1 + p_2 = 13.1 \text{ psf}$

Wall Panels:

- Wall Studs @ 16” O.C. (2x4 interior and 2x6 exterior): $h_1 := 8 \text{ ft}$
  - $W_{\text{stud}} := (w_{2\times4} \cdot h_1 \cdot 2 \cdot 25) + (w_{2\times6} \cdot h_1 \cdot 2 \cdot 18) = 839 \text{ lbf}$

- Top and Bottom Plate:
  - $W_{\text{pl}} := (w_{2\times4} \cdot 4 \cdot L) + (w_{2\times6} \cdot 4 \cdot W) = 189 \text{ lbf}$
  - $p_{\text{wall}} := \frac{(W_{\text{stud}} + W_{\text{pl}} + W_{\text{gyp}})}{L \cdot W} = 15 \text{ psf}$
Calculation of Module Estimated Weights

Ceiling System:

- Ceiling Joists (2x8 @ 16" O.C.):
  - \( W_{cj} := w_{2x8} \cdot 19 \cdot W = 530 \text{ lbf} \)
- Rim Board (2x8):
  - \( W_{rim} := w_{2x8} \cdot 2 \cdot (W + L) = 164 \text{ lbf} \)
- \( p_{ceiling} := \frac{(W_{cj} + W_{rim} + W_{gyp})}{L \cdot W} = 9.6 \text{ psf} \)

Total Weight of Wood-Framed Wall Bearing Unit:

- \( p_{total} := p_{floor} + p_{wall} + p_{ceiling} = 37.7 \text{ psf} \)

Structural Steel Construction (Estimate for Reference)

- 4" Concrete Floor: \( p_{floor} := 30 \text{ psf} \)
- Metal Deck : \( p_{deck} := 2.49 \text{ psf} \)
- Structural Steel: \( p_{beam} := 8.00 \text{ psf} \)
- Ceiling: \( p_{ceiling} := 3.00 \text{ psf} \)
- Exterior Framing: \( p_{wall} := 17.7 \text{ psf} \)

Total Weight of Comparable Steel Framed Construction:

- \( p_{total} := p_{floor} + p_{deck} + p_{beam} + p_{ceiling} + p_{wall} = 61.2 \text{ psf} \)
Appendix E

HMAC Drawings and Module Figures
Appendix F

Load Combinations and Gravity Loads
### Load Combinations

<table>
<thead>
<tr>
<th>ASCE 7-10 Base LC No.</th>
<th>Load Combination</th>
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<tr>
<td><strong>LRFD:</strong></td>
<td></td>
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<tr>
<td>2</td>
<td>1.2D+1.6L+0.5S+1.0H</td>
</tr>
<tr>
<td>3a</td>
<td>1.2D+1.6S+0.5L+1.0H</td>
</tr>
<tr>
<td>3b</td>
<td>1.2D+1.6S+0.5Wn+0.5Wr+1.0H</td>
</tr>
<tr>
<td>3c</td>
<td>1.2D+1.6S+0.5Ws+0.5Wr+1.0H</td>
</tr>
<tr>
<td>3d</td>
<td>1.2D+1.6S+0.5W+0.5Wr+1.0H</td>
</tr>
<tr>
<td>3e</td>
<td>1.2D+1.6S+0.5Ww+0.5Wr+1.0H</td>
</tr>
<tr>
<td>3f</td>
<td>1.2D+1.6S+0.5(0.75Wn)+0.5(0.75W)+0.5Wr+1.0H</td>
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<td>1.2D+1.6S+0.5(0.75Wn)+0.5(0.75W)+0.5Wr+1.0H</td>
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<td>3h</td>
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<td>4a</td>
<td>1.2D+1.0Wn+0.5L+0.5S+1.0H+1.0Wr</td>
</tr>
<tr>
<td>4b</td>
<td>1.2D+1.0Ws+0.5L+0.5S+1.0H+1.0Wr</td>
</tr>
<tr>
<td>4c</td>
<td>1.2D+1.0Wc+0.5L+0.5S+1.0H+1.0Wr</td>
</tr>
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<td>4d</td>
<td>1.2D+1.0Ww+0.5L+0.5S+1.0H+1.0Wr</td>
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<tr>
<td>4e</td>
<td>1.2D+1.0(0.75Wn)+1.0(0.75W)+1.0Wr+0.5L+0.5S+1.0H</td>
</tr>
<tr>
<td>4f</td>
<td>1.2D+1.0(0.75Wn)+1.0(0.75W)+1.0Wr+0.5L+0.5S+1.0H</td>
</tr>
<tr>
<td>4g</td>
<td>1.2D+1.0(0.75Ws)+1.0(0.75W)+1.0Wr+0.5L+0.5S+1.0H</td>
</tr>
<tr>
<td>4h</td>
<td>1.2D+1.0(0.75Ws)+1.0(0.75W)+1.0Wr+0.5L+0.5S+1.0H</td>
</tr>
<tr>
<td><strong>ASD:</strong></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>D+L</td>
</tr>
</tbody>
</table>
3  
D+S

4  
D+0.75L+0.75S

5a  
D+0.6Wn+0.6Wr

5b  
D+0.6Ws+0.6Wr

5c  
D+0.6We+0.6Wr

5d  
D+0.6Ww+0.6Wr

6a  
D+0.75L+0.75(0.6Wn)+0.75(0.6Wn)+0.75S

6b  
D+0.75L+0.75(0.6Ws)+0.75(0.6Ws)+0.75S

6c  
D+0.75L+0.75(0.6We)+0.75(0.6We)+0.75S

6d  
D+0.75L+0.75(0.6Ww)+0.75(0.6Ww)+0.75S

7a  
0.6D+0.6Wn+.6Wr

7b  
0.6D+0.6Ws+.6Wr

7c  
0.6D+0.6We+.6WR

7d  
0.6D+0.6Ww+.6WR

Service:

Drift1  
Wn

Drift2  
Ws

Drift3  
We

Drift4  
Ww

Deflection1  
D+L

Deflection2  
D+S

Deflection3  
L

Deflection4  
S

Existing Building Loads

Dead Loads

<table>
<thead>
<tr>
<th>Item</th>
<th>Weight (lb/ft²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1st Floor:</td>
<td></td>
</tr>
<tr>
<td>Linoleum</td>
<td>1.0</td>
</tr>
</tbody>
</table>
Plaster Ceiling (lath + gypsum plaster) 10.0
Concrete Floor Lath 1.0
Steel Joists 4.0
Total: 16.0

2nd Floor:
Concrete Floor Lath 1.0
Plaster Ceiling 10.0
2x6 Wood Sleepers 1.4
Hardwood 1” Nominal 4.0
Steel Joists 4.0
Total: 20.4

Roof:
Suspended ceiling system 15.0
Cinder fill (12”) 60.0
3-ply ready roofing 1.0
Steel Joists 4.0
Total: 80.0

Stair:
Total: 60.0

Exterior Walls:
4” Brick Veneer 39.0
8” Structural Clay Tile Masonry 38.0
Total: 77.0

Interior Walls:
1” Plaster (2-sides) 10.0
8” Clay Tile 38.0
Total: 48.0

Live Loading

Floor Live Load = 100 psf for both floor areas and stair areas (2009 IBC section T1607.1).
Roof Live Load = 20 psf
**Snow Load**

\[ P_g = 30 \text{ psf (IBC F1608.2)} \]

Flat Roof Snow Load (ASCE 7-10 7.3)

\[ C_e = 1.0 \text{ Terrain B Partially Exposed, } C_t = 1.0 \text{ Conditioned Structure, } I_s = 1.1 \text{ Risk Category III} \]

\[ P_f = 0.7 \ C_e \ C_t \ I_s \ P_g = 0.7 (1.0)(1.0)(1.1)(30 \text{psf}) = 33 \text{ psf} \]

**Lateral Soil Pressure**

Assume at-rest earth pressure \( \gamma = 125 \text{ pcf} \), \( k_0 = 0.8 \) (conservative),

\[ P_v = 125 \text{ pcf (12') } = 1500 \text{ psf} \]

\[ P_o = k_0 P_v = 0.8 (1500 \text{ psf}) = 1200 \text{ psf} \]

**Expansion Loads**

**Assigned Module Loads**

<table>
<thead>
<tr>
<th>Item</th>
<th>Weight (lb/ft2)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wall Bearing Module:</td>
<td></td>
</tr>
<tr>
<td>Roof Dead Load</td>
<td>15.0</td>
</tr>
<tr>
<td>Roof Snow Load</td>
<td>33.0</td>
</tr>
<tr>
<td>Floor Dead Load</td>
<td>30.0</td>
</tr>
<tr>
<td>Aluminum Cladding Load</td>
<td>3.0</td>
</tr>
<tr>
<td>Dwelling Floor Live</td>
<td>40.0</td>
</tr>
<tr>
<td>Maintenance Module Floor Live</td>
<td>100.0</td>
</tr>
<tr>
<td>Wind</td>
<td>Ref. Appendix C</td>
</tr>
<tr>
<td>Wall Dead</td>
<td>Ref. Appendix E</td>
</tr>
</tbody>
</table>

Corner Bearing Module:
<table>
<thead>
<tr>
<th>Load Type</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof Dead Load</td>
<td>15.0</td>
</tr>
<tr>
<td>Roof Snow Load</td>
<td>33.0</td>
</tr>
<tr>
<td>Floor Dead Load</td>
<td>ETABS *</td>
</tr>
<tr>
<td>Aluminum Cladding Load</td>
<td>3.0</td>
</tr>
<tr>
<td>Dwelling Floor Live</td>
<td>40.0</td>
</tr>
<tr>
<td>Maintenance Module Floor Live</td>
<td>100.0</td>
</tr>
<tr>
<td>Wind</td>
<td>Ref. Appendix C</td>
</tr>
<tr>
<td>Wall Dead</td>
<td>Ref. Appendix D</td>
</tr>
</tbody>
</table>

* Calculated by software
Appendix G

Wind Load Calculations
Loading: Wind Load

Wind Cases:

- Note building plan was simplified and squared off.
- ASCE 7-10 Chapter 27 Part II wind provisions were used to calculate wind loads.
  
  - Enclosed simple diaphragm
  - Regular shape

- Only Case 1 was investigated.
- Building specifications
  
  - Risk Category II
  - Iw = 1.0
  - V = 115 mph
  - kd = 0.85
  - Terrain Category B, partially exposed
  - kzt = 1.0
  - Exposure B

![Figure 1. ASCE Load Case I wind in two orthogonal directions in combination with uplift](image)

**Geometry (Class I Building)**

Requirements:

1. 0.2L ≤ B ≤ 5L: B = L therefore OK
2. h = 59.5' (expanded building) < 160': OK for chapter 27 calculations
3. Building is torsionally regular per Appendix D 1.5.1, Class A, Only the evaluation of load cases 1 and 3 (Table 27.4-8) is required. For the purposes of this research only Load Case 1 will be evaluated for both orthogonal directions.
4. Because lateral resistance system consists of rigid shear walls and diaphragms apply net wind pressure on one windward side only.
5. L/B = 80'/80' = 1.0 therefore apply the same magnitude wind pressures in both the x and y direction.
Evaluations of the Wind Load For the Existing Building

From Table 27.6-1 the horizontal wall loading is as follows:

The Building: \( h_{\text{mean}} := 38.5 \text{ ft} \quad P_o := 20.4 \text{ psf} \quad P_h := 23.2 \text{ psf} \)

The Parapet: \( h_{\text{meanp}} := 42 \text{ ft} \quad P_{hp} := 23.9 \text{ psf} \quad P_p := 2.25 \cdot P_{hp} = 53.8 \text{ psf} \)

From Table 27.6-2 the vertical roof loading is as follows:

Exposure Adjustment Factor (Estimate with \( h = 40' \)):

\( \lambda := 0.729 \)

Zone₁ := \(-31.8 \text{ psf} \cdot \lambda = -23.2 \text{ psf} \) \quad Zone₂ := \(-28.4 \text{ psf} \cdot \lambda = -20.7 \text{ psf} \)

Zone₃ := \(-23.3 \text{ psf} \cdot \lambda = -17 \text{ psf} \)

\[ \begin{align*}
\text{Figure 2. Wind loading on the original building. The wind pressure increases uniformly from the ground to the roof (with the exception of the parapet), but the pressures were squared off at each floor for simplification and conservative estimate.}
\end{align*} \]

\[ \frac{P_h - P_o}{h_{\text{mean}}} = 0.073 \text{ psf} \quad \text{ft} \quad P_1 := P_o + m \cdot 13 \text{ ft} = 21.3 \text{ psf} \]

\[ B := 80 \text{ ft} \quad L := B \quad P_2 := P_o + m \cdot (13 \text{ ft} + 18 \text{ ft}) = 22.7 \text{ psf} \]
Loading: Wind Load

\[
P_{\text{uplift}} := \frac{(\text{Zone}_3 \cdot B \cdot 21 \text{ ft}) + (\text{Zone}_4 \cdot B \cdot 21 \text{ ft}) + (\text{Zone}_5 \cdot B \cdot 38 \text{ ft})}{B \cdot L} = -19.6 \text{ psf}
\]

Evaluations of the Wind Load For the Expanded Building

From Table 27.6-1 the horizontal wall loading is as follows:

The Building: \( h_{\text{mean}} := 60 \text{ ft} \) \( P_o := 21.9 \text{ psf} \) \( P_h := 27.1 \text{ psf} \)

The Parapet: (Parapet not installed at roof of expansion)

From Table 27.6-2 the vertical roof loading is as follows:

Exposure Adjustment Factor (Estimate with \( h = 40^{\circ} \)): \( \lambda := 0.751 \)

\( \text{Zone}_3 := -34.7 \text{ psf} \cdot \lambda = -26.1 \text{ psf} \) \( \text{Zone}_4 := -30.9 \text{ psf} \cdot \lambda = -23.2 \text{ psf} \)

\( \text{Zone}_5 := -25.3 \text{ psf} \cdot \lambda = -19 \text{ psf} \)

Figure 3. Wind load applied to the expanded building; two story expansion shown.
Loading: Wind Load

\[ m := \frac{P_h - P_o}{h_{\text{mean}}} = 0.087 \text{ psf/ft} \quad B := 80 \text{ ft} \quad L := B \]

\[ P_{\text{uplift}} := \frac{(\text{Zone}_3 \cdot B \cdot 21 \text{ ft}) + (\text{Zone}_4 \cdot B \cdot 21 \text{ ft}) + (\text{Zone}_5 \cdot B \cdot 38 \text{ ft})}{B \cdot L} = -22 \text{ psf} \]

\[ P_1 := P_o + m \cdot 13 \text{ ft} = 23 \text{ psf} \]

\[ P_2 := P_o + m \cdot (13 \text{ ft} + 18 \text{ ft}) = 24.6 \text{ psf} \]

\[ P_3 := P_o + m \cdot (13 \text{ ft} + 18 \text{ ft} + 7.5 \text{ ft} + 1 \text{ ft}) = 25.3 \text{ psf} \]

\[ P_4 := P_o + m \cdot (13 \text{ ft} + 18 \text{ ft} + 7.5 \text{ ft} + 1 \text{ ft} + 10 \text{ ft}) = 26.2 \text{ psf} \]
Appendix H

Member Capacity Calculations
Determine Capacity of Column

- Member has internal moment and axial effects present, therefore must be treated as a beam column.
- The concentrated loads created by the truss top and bottom chord should be checked in design, but will not be checked in this calculation.
- 2nd order effects should be checked in design, but will not be checked in this calculation.
- H10x72,H12x126.5
- Columns are continuously braced by the masonry along the weak axis, but are only braced @ the floor levels in the direction of bending of the strong axis.

Column Properties (AISC Design Guide 15)

\[ E_s = 29000 \text{ ksi} \]
\[ F_y = 30 \text{ ksi} \]
\[ F_u = 55 \text{ ksi} \]

Calculate Capacity of H12x126.5

Compressive Strength

\[ A_g = 37.21 \text{ in}^2 \quad d = 12.88 \text{ in.} \quad t_w = 0.740 \text{ in.} \]
\[ b_f = 12.27 \text{ in.} \quad t_f = 1.188 \text{ in.} \quad h = 9.257 \text{ in.} \]

Check compactness

\[ \frac{b_f}{2 \cdot t_f} \leq 0.56 \cdot \sqrt{\frac{E_s}{F_y}} = 17.4 \quad \text{Non-compact Flange} \]
\[ \frac{h}{t_w} = 12.5 \leq 1.49 \cdot \sqrt{\frac{E_s}{F_y}} = 46.3 \quad \text{Non-compact Web} \]
\[ k_x = 1.0 \quad L_x = 13 \cdot 12 = 156 \text{ in.} \quad r_x = 5.38 \text{ in.} \]

Slender \[ \leq \frac{k_x \cdot L_x}{r_x} = 29 \leq 4.71 \cdot \sqrt{\frac{E_s}{F_y}} = 146.4 \leq 200 \]

Therefore:

\[ F_e = \frac{\pi^2 \cdot E_s}{\text{Slender}^2} = 340.4 \]
\[ F_{cr} = \left(0.658 \frac{F_y}{F_s}\right) \cdot F_y = 28.9 \]
\[ \phi_c = 0.90 \]

\[ \phi_c P_n := \phi_c \cdot F_{cr} \cdot A_g = 968.3 \text{ Kips} \]
Member Capacities: Frame 3 Column

Moment Capacity (strong axis, x-x)

Check compactness

\[
\frac{b_f}{2 \cdot t_f} = 5.16 \leq 0.38 \cdot \sqrt{\frac{E_s}{F_y}} = 11.8 \quad \text{Compact Flange}
\]

\[
\frac{h}{t_w} = 12.5 \leq 3.76 \cdot \sqrt{\frac{E_s}{F_y}} = 116.9 \quad \text{Compact Web}
\]

\(L_b := 0\) and the member is compact; therefore yielding controls design.

Estimate Plastic Section Modulus

\[
Z_x := 2 \cdot \left( b_f \cdot t_f \cdot \left( \frac{d - t_f}{2} \right) + \left( \frac{d - 2 \cdot t_f}{2} \right) \cdot \left(\frac{d - 2 \cdot t_f}{4}\right) \right) = 190.8 \quad \text{in}^3
\]

\(\phi_b := 0.90\) \quad \(\phi_b M_n := \phi_b \cdot F_y \cdot Z_x \cdot \frac{1}{12} = 429.4 \quad \text{K*ft.}\)

Take Column to be a member in a braced frame

- Assume insignificant 2nd order effects.

\(B_1 := 1.0\)

The corner bearing module two-story extension produces the maximum load effects in this basement column.

- Compare to capacity of column

\(P_u := 395\) Kips \quad \(M_u := 28\) K*ft.

\(P_r := P_u = 395\) K*ft. \quad \(M_{rx} := B_1 \cdot M_u = 28\) K*ft.

\(P_c := \phi_c P_n = 968\) Kips \quad \(M_{cx} := \phi_b M_n = 429\) \quad \(\phi_b M_n\)

\[
\frac{P_r}{P_c} = 0.4 \geq 0.2 \quad \text{Therefore,} \quad \frac{P_r}{P_c} + \frac{8}{9} \left( \frac{M_{rx}}{M_{cx}} \right) = 0.466 \leq 1.0
\]

Therefore column has adequate capacity to support a 2-story corner-bearing module extension.
Calculate Capacity of H10x72

Compressive Strength

\[ A_g := 20.91 \text{ in}^2 \quad d := 10.38 \text{ in.} \quad t_w := 0.510 \text{ in.} \]
\[ b_f := 10.12 \text{ in.} \quad t_f := 0.813 \text{ in.} \quad h := 7.692 \text{ in.} \]

Check compactness

\[ \frac{b_f}{2 \cdot t_f} = 6.22 \leq 0.56 \sqrt{\frac{E_s}{F_y}} = 17.4 \quad \text{Non-compact Flange} \]
\[ \frac{h}{t_w} = 15.1 \leq 1.49 \sqrt{\frac{E_s}{F_y}} = 46.3 \quad \text{Non-compact Web} \]
\[ k_x := 1.0 \quad L_x := 18 \cdot 12 = 216 \text{ in.} \quad r_x := 4.40 \text{ in.} \]

Slender := \frac{k_x \cdot L_x}{r_x} = 49.1 \leq 4.71 \sqrt{\frac{E_s}{F_y}} = 146.4 \leq 200

Therefore:

\[ F_e := \frac{\pi^2 E_s}{\text{Slender}^2} = 118.8 \quad F_{cr} := \left(0.658 \frac{F_y}{E_s}\right) \cdot F_y = 27 \quad \phi_c := 0.90 \]

\[ \phi_c P_n := \phi_c \cdot F_{cr} \cdot A_g = 507.9 \text{ Kips} \]

Moment Capacity (strong axis, x-x)

Check compactness

\[ \frac{b_f}{2 \cdot t_f} = 6.22 \leq 0.38 \sqrt{\frac{E_s}{F_y}} = 11.8 \quad \text{Compact Flange} \]
\[ \frac{h}{t_w} = 15.1 \leq 3.76 \sqrt{\frac{E_s}{F_y}} = 116.9 \quad \text{Compact Web} \]

\[ L_b := 0 \quad \text{and the member is compact; therefore yeilding controls design.} \]
Member Capacities: Frame 3 Column

\[ Z_x := 2 \cdot \left( (b_f \cdot t_f) \cdot \frac{d}{2} - \frac{t_f}{2} \right) + \left( \frac{(d-2 \cdot t_f) \cdot t_w}{2} \right) \cdot \left( \frac{d-2 \cdot t_f}{4} \right) = 88.5 \text{ in.}^3 \]

\[ \phi_b := 0.90 \quad \phi_{b,Mn} := \phi_b \cdot F_y \cdot Z_x \cdot \frac{1}{12} = 199.1 \text{ K*ft.} \]

Take Column to be a member in a braced frame

- Assume insignificant 2nd order effects.

\[ B_1 := 1.0 \]

The corner bearing module two-story extension produces the maximum load effects in this basement column

- Compare to capacity of column

\[ P_u := 280 \text{ Kips} \quad M_u := 26 \text{ K*ft.} \]

\[ P_r := P_u = 280 \text{ K*ft.} \quad M_{rx} := B_1 \cdot M_u = 26 \text{ K*ft.} \]

\[ P_c := \phi_c P_n = 508 \text{ Kips} \quad M_{cx} := \phi_{b,Mn} = 199 \quad \phi_{b,Mn} \]

\[ \frac{P_r}{P_c} = 0.6 \geq 0.2 \text{ Therefore,} \quad \frac{P_r + 8 \left( \frac{M_{rx}}{M_{cx}} \right)}{9 \left( \frac{M_{cx}}{M_{cx}} \right)} = 0.667 \leq 1.0 \]

Therefore column has adequate capacity to support a 2-story corner-bearing module extension.
Member Capacities: Shear Walls Line A

Determine the Shear and Flexural Capacity of the Shear Wall Segments Along Wall Line A (South Elevation of the plan set - Figure 2)

Member Specifications:

- Exterior walls constructed of 8x8x12 Structural Clay Tile (See Figure 1).

Reference Publications:

1. ACI 530-11
2. ASCE 7-10
3. Appendix A wind load calculations

General Assumptions (Method 1):

- That segments contain no reinforcement or grout.
- That only segments 1, 2 and 3 contribute to shear resistance.
- Type A mortar (obsolete) was used for the joints; compressive strength = 2,500 psi.
- Assume that bed joints were divided joints at a width of 2.5".
- Assume that the floor systems are diaphragms that transmit the shear loading to the side walls.
- Assume load combinations 7C and 7D control, 0.6D + 0.6W.
- Assume walls are side constructed in running bond.

Figure 1. Structural Clay Tile Block

Figure 2. South Elevation
Calculated Relative Rigidity of Wall Segments to Distribute Shear Load

- Evaluate wall between the 1st and 2nd floor
- Assume fixed base and top for the purposes of calculating rigidity.

\[ d_1 := 23 \text{ ft} \quad d_{23} := 6 \text{ ft} \quad h := 13 \text{ ft} \]

\[ \Delta_1 := 0.1 \cdot \left( \frac{h}{d_1} \right)^3 + 0.3 \cdot \left( \frac{h}{d_1} \right) = 0.188 \]

\[ \Delta_{23} := 0.1 \cdot \left( \frac{h}{d_{23}} \right)^3 + 0.3 \cdot \left( \frac{h}{d_{23}} \right) = 1.667 \]

\[ R_1 := \frac{1}{\Delta_1} = 5.33 \]

\[ R_{23} := \frac{1}{\Delta_{23}} = 0.6 \]

\[ R_{\text{total}} := R_1 + 2 \cdot R_{23} = 6.5 \]

\[ F_{R1} := \frac{R_1}{R_{\text{total}}} = 0.816 \]
\[ F_{R23} := \frac{R_{23}}{R_{\text{total}}} = 0.092 \]
Member Capacities: Shear Walls Line A

Calculate Design Shear and Moment in Wall Segments

- Use an adjustment factor: \((\text{Rigidity}) \times (\text{LF}=0.6) \times (1/2 \text{ of Total Shear} = 0.5)\)

\[
\text{adjust}_1 := F_{R1} \cdot 0.6 \cdot 0.5 = 0.245 \quad \text{adjust}_{23} := F_{R23} \cdot 0.6 \cdot 0.5 = 0.028
\]

\(w := 80 \text{ ft} \quad \text{(Building Width Perpendicular to Wind Load)}\)

**Segment 1**

\[
V_{11} := \text{adjust}_1 \cdot \left( (58.5 \text{ psf} \cdot 3.5 \text{ ft} \cdot w + 23.2 \text{ psf} \cdot 7.5 \text{ ft} \cdot w + 22.7 \text{ psf} \cdot 5.25 \text{ ft} \cdot w) = 9.75 \text{ kip} \right)
\]

\[
V_{21} := \text{adjust}_1 \cdot \left( (22.7 \text{ psf} \cdot 12.75 \text{ ft} \cdot w + 21.4 \text{ psf} \cdot \left( \frac{13 \text{ ft}}{2} \right) \cdot w) = 8.4 \text{ kip} \right)
\]

\[
V_{1\text{total}} := V_{11} + V_{21} = 18.15 \text{ kip} \quad M_1 := V_{11} \cdot (38.5 \text{ ft}) + V_{21} \cdot (13 \text{ ft}) = 5816245 \text{ lbf} \cdot \text{in}
\]

**Segments 2 and 3**

\[
V_{123} := \frac{V_{11}}{\text{adjust}_{23}} \cdot \frac{\text{adjust}_{23}}{\text{adjust}_1} = 1.1 \text{ kip} \quad V_{223} := \frac{V_{21}}{\text{adjust}_{23}} \cdot \frac{\text{adjust}_{23}}{\text{adjust}_1} = 0.94 \text{ kip}
\]

\[
V_{23\text{total}} := V_{123} + V_{223} = 2.043 \text{ kip} \quad M_{23} := V_{123} \cdot (38.5 \text{ ft}) + V_{223} \cdot (13 \text{ ft}) = 654572 \text{ lbf} \cdot \text{in}
\]

Calculate Shear Capacity in Segment 1

- Calculated shear capacity at the top of the 1st floor wall. This is where shear is maximum and the weight of the wall above is minimum.

\[
h_1 := 18 \text{ ft} + 7.5 \text{ ft} + 3.5 \text{ ft} = 29 \text{ ft} \quad \text{(Height of wall above top of 1st floor wall)}
\]

\[
h_2 := h_1 + 13 \text{ ft} = 42 \text{ ft} \quad \text{(Height of wall above bottom of 1st floor wall)}
\]

\[
P_{\text{block}} := 34 \text{ psf} \quad \text{(Weight of 8" clay tile)} \quad f^\prime m := 363 \quad \text{(psi)}
\]

\[
A_{\text{shear}} := \frac{(2 \cdot 2.5 \text{ in} \cdot 12 \text{ in})}{\text{ft}} = 60 \frac{\text{in}^2}{\text{ft}} \quad \text{(Shear Area, grout contact area on bed joints)}
\]
Member Capacities: Shear Walls Line A

\[ N_{v1} := p_{block} \cdot h_1 \cdot d_1 \cdot 0.6 = 13.61 \text{ kip} \]

\[ f_v := 1.5 \cdot \left( \frac{V_{\text{total}}}{A_{\text{shear}} \cdot d_1} \right) = 19.7 \text{ psi} \]

\[ F_{v1} := 1.5 \cdot \sqrt{f'm} = 28.6 \text{ (psi)} \]
\[ F_{v1} := F_{v1} \cdot \text{psi} = 28.6 \text{ psi} \]
\[ F_{v2} := 120 \text{ psi} \]

\[ F_{v3} := 37 \text{ psi} + 0.45 \cdot \left( \frac{N_{v1}}{A_{\text{shear}} \cdot d_1} \right) = 41.4 \text{ psi} \]

\[ F_v := \min(F_{v1}, F_{v2}, F_{v3}) = 28.6 \text{ psi} > f_v = 19.7 \text{ psi} \text{ therefore OK for shear} \]

Calculate Moment Capacity in Segment 1

- Calculated moment capacity at the bottom of the 1st floor wall. This is where moment is maximum.

\[ I_1 := \frac{(2.5 \text{ in} \cdot 2) \cdot (d_1)^3}{12} = 8760240 \text{ in}^4 \]
\[ N_{v1} := p_{block} \cdot h_2 \cdot d_1 \cdot 0.6 = 19.71 \text{ kip} \]

\[ f_{\text{ten}} := \frac{M_1 \cdot \left( \frac{d_1}{2} \right)}{I_1} - \frac{N_{v1}}{A_{\text{shear}} \cdot d_1} = 77.3 \text{ psi} \]

Type A mortar is obsolete, therefore compare to the allowable flexural tension for Type M mortar (PCL). Both Type A and Type M have the same compressive of strength = 2,500 psi.

\[ F_{\text{Tungrouted}} := 33 \text{ psi} \]
\[ F_{\text{Tgrouted}} := 86 \text{ psi} \]

This shear wall segment does not pass code as an unreinforced ungrouted section but might as unreinforced fully grouted section or perhaps a partially grouted section if all assumptions are true. It was originally assumed that the exterior walls are unreinforced. This may not be the case however. In order for this wall to pass current code some level of grouting or reinforcement would have to be identified. Alternatively a more precise method of calculating the shear capacity could be used, such as a perforated method where the spandrel masonry sections would contribute to the overall shear capacity of the wall.
Member Capacities: Shear Walls Line A

Calculate Shear Capacity in Segments 2 and 3

\[ N_{v23} = p_{block} \cdot h_1 \cdot d_{23} \cdot 0.6 = 3.55 \text{ kip} \]

\[ f_v := 1.5 \cdot \left( \frac{V_{23\text{total}}}{A_{\text{shear} \cdot d_{23}}} \right) = 8.5 \text{ psi} \]

\[ F_{v1} := 1.5 \cdot \sqrt{f'm} = 28.6 \text{ (psi)} \]

\[ F_{v1} := F_{v1} \cdot \text{psi} = 28.6 \text{ psi} \]

\[ F_{v2} := 120 \text{ psi} \]

\[ F_{v3} := 37 \text{ psi} + 0.45 \cdot \left( \frac{N_{v23}}{A_{\text{shear} \cdot d_{23}}} \right) = 41.4 \text{ psi} \]

\[ F_v := \min (F_{v1}, F_{v2}, F_{v3}) = 28.6 \text{ psi} > f_v = 8.5 \text{ psi} \]

therefore OK for shear

Calculate Moment Capacity in Segment 2 and 3

\[ I_{23} := \frac{(2.5 \text{ in} \cdot 2) \cdot (d_{23})^3}{12} = 155520 \text{ in}^4 \]

\[ N_{v23} := p_{block} \cdot h_2 \cdot d_{23} \cdot 0.6 = 5.14 \text{ kip} \]

\[ f_{1\text{ten}} := \frac{M_{23}}{I_{23}} - \frac{N_{v23}}{A_{\text{shear} \cdot d_{23}}} = 137.2 \text{ psi} \]

\[ F_{\text{ungrouted}} := 33 \text{ psi} \]

\[ F_{\text{grouted}} := 86 \text{ psi} \]

This shear wall segment does not pass code as an unreinforced ungrouted section or as an unreinforced fully grouted section. In order for these wall segments to be utilized as shear walls some level of reinforcement would have to be identified.

Discussion:

No further analysis will be conducted on the shear walls for the expansions. The expansions will transfer shear loads from the wind through the roof diaphragm and into the existing masonry segments. It appears, from these basic calculations, that the possibility exists that the current walls do not meet the current design codes. Considering this, it is highly likely that additional stories would overstress the existing masonry shear walls. Investigation would have to be conducted in order to determine more precisely the composition of the exterior masonry walls. If no reinforcing is identified it may be prudent to re-evaluate the wind loads, considering the sheltered nature of the building or use a more precise perforated method to determine the capacity of the shear walls.
Member Capacities: Shear Walls Line A

Assumption modification (Method 2):

Shear resistance seems overly conservative. Modify assumptions to include wall segments between windows and at corner. Include corner returns (CR) as well.

![Figure 4. South Elevation (Method 2)](image)

Calculated Relative Rigidity of Wall Segments to Distribute Shear Load

- Governing rigidity equation: \( \Delta = \left( \frac{h^3}{12I} \right) + \left( \frac{3h}{Av} \right) \).
- Calculate moment resistance; assume shear resistance is OK based on method 1 calculations.

\[
\begin{align*}
\delta_1 &:= 23 \text{ ft} & \delta_{23} &:= 6 \text{ ft} & \delta_{1abc} &:= 2.5 \text{ ft} & \delta_4 &:= 4 \text{ ft} \\
h &:= 13 \text{ ft} & CR_1 &:= 4 \text{ ft} & CR_4 &:= 10 \text{ ft} \\
\end{align*}
\]

Segment 1

\[
A_{v1} := (2.5 \text{ in} \cdot 2) \cdot (d_1 + CR_1 - 8 \text{ in}) = 1580 \text{ in}^2
\]

Determine I (See Figure 5)

\[
\begin{align*}
A_{p1} &:= 2.5 \text{ in} \cdot 276 \text{ in} & A_{p2} &:= 2.5 \text{ in} \cdot 270.5 \text{ in} & A_{p3} &:= 2.5 \text{ in} \cdot 45.5 \text{ in} & A_{p4} &:= 2.5 \text{ in} \cdot 40 \text{ in} \\
Y_{bar1} &:= \frac{(A_{p1} \cdot 138 \text{ in} + A_{p2} \cdot 135.25 \text{ in} + A_{p3} \cdot 274.75 \text{ in} + A_{p4} \cdot 269.25 \text{ in})}{A_{p1} + A_{p2} + A_{p3} + A_{p4}} = 155 \text{ in}
\end{align*}
\]
Member Capacities: Shear Walls Line A

\[
I_{xx1} := \frac{2.5 \text{ in} \cdot (276 \text{ in})^3}{12} + A_{p1} \cdot (138 \text{ in})^2 = (1.75 \cdot 10^7) \text{ in}^4
\]

\[
I_{xx2} := \frac{2.5 \text{ in} \cdot (270.5 \text{ in})^3}{12} + A_{p2} \cdot (135.25 \text{ in})^2 = (1.65 \cdot 10^7) \text{ in}^4
\]

\[
I_{xx3} := \frac{45.5 \text{ in} \cdot (2.5 \text{ in})^3}{12} + A_{p3} \cdot (274.75 \text{ in})^2 = (8.59 \cdot 10^6) \text{ in}^4
\]

\[
I_{xx4} := \frac{40 \text{ in} \cdot (2.5 \text{ in})^3}{12} + A_{p4} \cdot (269.25 \text{ in})^2 = (7.25 \cdot 10^6) \text{ in}^4
\]

\[
I_{cc1} := (I_{xx1} + I_{xx2} + I_{xx3} + I_{xx4}) - (A_{p1} + A_{p2} + A_{p3} + A_{p4}) \cdot y_{bar1}^2 = (1.19 \cdot 10^7) \text{ in}^4
\]

Relative Rigidity

\[
\Delta_1 := \left( \frac{h^3}{12 \cdot I_{cc1}} + \frac{3 \cdot h}{A_{v1}} \right) \cdot 1 \text{ in} = 0.323 \quad R_1 := \frac{1}{\Delta_1} = 3.1
\]

Segment 1a, 1b, 1c

\[
A_{v1a} := 2 \cdot 2.5 \text{ in} \cdot d_{1abc} = 150 \text{ in}^2
\]

\[
I_{cc1a} := \frac{2.5 \text{ in} \cdot (d_{1abc})^3}{12} = 5625 \text{ in}^4
\]

\[
\Delta_{1a} := \left( \frac{h^3}{12 \cdot I_{cc1a}} + \frac{3 \cdot h}{A_{v1a}} \right) \cdot 1 \text{ in} = 59.363 \quad R_{1a} := \frac{1}{\Delta_{1a}} = 0.017 \quad R_{1b} := R_{1a} \quad R_{1c} := R_{1a}
\]

Segment 2, 3

\[
A_{v2} := 2 \cdot 2.5 \text{ in} \cdot d_{23} = 360 \text{ in}^2
\]

\[
I_{cc2} := \frac{2.5 \text{ in} \cdot (d_{23})^3}{12} = 77760 \text{ in}^4
\]

\[
\Delta_2 := \left( \frac{h^3}{12 \cdot I_{cc2}} + \frac{3 \cdot h}{A_{v2}} \right) \cdot 1 \text{ in} = 5.369 \quad R_2 := \frac{1}{\Delta_2} = 0.186 \quad R_3 := R_2
\]
Member Capacities: Shear Walls Line A

\[ A_{v4} := (2.5 \text{ in} \cdot 2) \cdot (d_4 + CR_4 - 8 \text{ in}) = 800 \text{ in}^2 \]

**Determine I (See Figure 6)**

\[ A_{p1} := 2.5 \text{ in} \cdot 48 \text{ in} \quad A_{p2} := 2.5 \text{ in} \cdot 42.5 \text{ in} \quad A_{p3} := 2.5 \text{ in} \cdot 45.5 \text{ in} \quad A_{p4} := 2.5 \text{ in} \cdot 40 \text{ in} \]

\[ y_{\text{bar}4} := \frac{(A_{p1} \cdot 24 \text{ in} + A_{p2} \cdot 21.25 \text{ in} + A_{p3} \cdot 46.75 \text{ in} + A_{p4} \cdot 41.25 \text{ in})}{A_{p1} + A_{p2} + A_{p3} + A_{p4}} = 33.1 \text{ in} \]

\[ I_{xx1} := \frac{2.5 \text{ in} \cdot (48 \text{ in})^3}{12} + A_{p1} \cdot (24 \text{ in})^2 = (9.22 \cdot 10^4) \text{ in}^4 \]

\[ I_{xx2} := \frac{2.5 \text{ in} \cdot (42.5 \text{ in})^3}{12} + A_{p2} \cdot (21.25 \text{ in})^2 = (6.4 \cdot 10^4) \text{ in}^4 \]

\[ I_{xx3} := \frac{45.5 \text{ in} \cdot (2.5 \text{ in})^3}{12} + A_{p3} \cdot (46.75 \text{ in})^2 = (2.49 \cdot 10^5) \text{ in}^4 \]

\[ I_{xx4} := \frac{40 \text{ in} \cdot (2.5 \text{ in})^3}{12} + A_{p4} \cdot (41.25 \text{ in})^2 = (1.7 \cdot 10^5) \text{ in}^4 \]

\[ I_{cc4} := (I_{xx1} + I_{xx2} + I_{xx3} + I_{xx4}) - (A_{p1} + A_{p2} + A_{p3} + A_{p4}) \cdot y_{\text{bar}4}^2 = (9.18 \cdot 10^4) \text{ in}^4 \]

**Relative Rigidity**

\[ \Delta_4 := \frac{h^3}{12 \cdot I_{cc4}} + \frac{3 \cdot h}{A_{v4}} \cdot 1 \text{ in} = 4.03 \quad R_4 := \frac{1}{\Delta_4} = 0.248 \]

**Load Distribution Based on Rigidity**

\[ R_{\text{total}} := R_1 + R_{1a} + R_{1b} + R_{1c} + R_2 + R_3 + R_4 = 3.769 \]

\[ F_{R1} := \frac{R_1}{R_{\text{total}}} = 0.822 \quad F_{R2} := \frac{R_2}{R_{\text{total}}} = 0.049 \quad F_{R3} := \frac{R_3}{R_{\text{total}}} = 0.049 \quad F_{R4} := \frac{R_4}{R_{\text{total}}} = 0.066 \]

\[ F_{R1a} := \frac{R_{1a}}{R_{\text{total}}} = 0.004 \quad F_{R1b} := \frac{R_{1b}}{R_{\text{total}}} = 0.004 \quad F_{R1c} := \frac{R_{1c}}{R_{\text{total}}} = 0.004 \]
Member Capacities: Shear Walls Line A

\[ \text{adjust}_1 := F_{R1} \cdot 0.6 \cdot 0.5 = 0.247 \quad \text{adjust}_2 := F_{R2} \cdot 0.6 \cdot 0.5 = 0.015 \]

\[ \text{adjust}_{1a} := F_{R1a} \cdot 0.6 \cdot 0.5 = 0.001 \quad \text{adjust}_4 := F_{R4} \cdot 0.6 \cdot 0.5 = 0.02 \]

**Segment 1**

\[ V_{11} := \text{adjust}_1 \cdot (58.5 \text{ psf} \cdot 3.5 \text{ ft} \cdot w + 23.2 \text{ psf} \cdot 7.5 \text{ ft} \cdot w + 22.7 \text{ psf} \cdot 5.25 \text{ ft} \cdot w) = 9.82 \text{ kip} \]

\[ V_{21} := \text{adjust}_1 \cdot (22.7 \text{ psf} \cdot 12.75 \text{ ft} \cdot w + 21.4 \text{ psf} \cdot \left(\frac{13 \text{ ft}}{2}\right) \cdot w) = 8.45 \text{ kip} \]

\[ V_{1\text{total}} := V_{11} + V_{21} = 18.28 \text{ kip} \quad M_1 := V_{11} \cdot (38.5 \text{ ft}) + V_{21} \cdot (13 \text{ ft}) = 5856525 \text{ lbf} \cdot \text{in} \]

\[ N_{v1} := p_{\text{block}} \cdot h_2 \cdot (d_1 + CR_1 - 8 \text{ in}) \cdot 0.6 = 22.56 \text{ kip} \]

\[ f_{\text{ten}} := \frac{M_1 \cdot \langle y_{\text{bar1}} \rangle}{I_{cc1}} - \frac{N_{v1}}{A_{v1}} = 62 \text{ psi} \]

\( f_{\text{ten}} = 62 \text{ psi} < 33 \text{ psi} \) but > 86 psi, therefore some level of grouting would need to be assumed in order to be OK.

**Segment 2**

\[ V_{12} := \text{adjust}_2 \cdot (58.5 \text{ psf} \cdot 3.5 \text{ ft} \cdot w + 23.2 \text{ psf} \cdot 7.5 \text{ ft} \cdot w + 22.7 \text{ psf} \cdot 5.25 \text{ ft} \cdot w) = 0.59 \text{ kip} \]

\[ V_{22} := \text{adjust}_2 \cdot (22.7 \text{ psf} \cdot 12.75 \text{ ft} \cdot w + 21.4 \text{ psf} \cdot \left(\frac{13 \text{ ft}}{2}\right) \cdot w) = 0.51 \text{ kip} \]

\[ V_{2\text{total}} := V_{12} + V_{22} = 1.1 \text{ kip} \quad M_2 := V_{12} \cdot (38.5 \text{ ft}) + V_{22} \cdot (13 \text{ ft}) = 352122 \text{ lbf} \cdot \text{in} \]

\[ N_{v2} := p_{\text{block}} \cdot h_2 \cdot d_{23} \cdot 0.6 = 5.14 \text{ kip} \]

\[ I_{23} := \frac{(2.5 \text{ in} \cdot 2) \cdot (d_{23})^3}{12} = 155520 \text{ in}^4 \quad f_{\text{ten}} := \frac{M_2 \cdot \left(\frac{d_{23}}{2}\right)}{I_{23}} - \frac{N_{v2}}{A_{v2}} = 67.2 \text{ psi} \]

\( f_{\text{ten}} = 67.2 \text{ psi} < 33 \text{ psi} \) but > 86 psi, therefore some level of grouting would need to be assumed in order to be OK.
Segment 1a

\[ V_{11a} := \text{adjust}_{1a} \cdot (58.5 \text{ psf} \cdot 3.5 \text{ ft} \cdot \text{w} + 23.2 \text{ psf} \cdot 7.5 \text{ ft} \cdot \text{w} + 22.7 \text{ psf} \cdot 5.25 \text{ ft} \cdot \text{w}) = 0.05 \text{ kip} \]

\[ V_{21a} := \text{adjust}_{1a} \cdot \left(22.7 \text{ psf} \cdot 12.75 \text{ ft} \cdot \text{w} + 21.4 \text{ psf} \cdot \frac{13 \text{ ft}}{2} \cdot \text{w} \right) = 0.05 \text{ kip} \]

\[ V_{1a_{total}} := V_{11a} + V_{21a} = 0.1 \text{ kip} \quad M_{1a} := V_{11a} \cdot (38.5 \text{ ft}) + V_{21a} \cdot (13 \text{ ft}) = 31844 \text{ lbf} \cdot \text{in} \]

\[ N_{v1a} := p_{\text{block}} \cdot h_2 \cdot d_{labc} \cdot 0.6 = 2.14 \text{ kip} \]

\[ I_{1a} := \frac{(2.5 \text{ in} \cdot 2) \cdot (d_{labc})^3}{12} = 11250 \text{ in}^4 \]

\[ f_{1ten} := \frac{M_{1a} \cdot \frac{d_{labc}}{2}}{I_{23}} - \frac{N_{v1a}}{A_{v1a}} = -11.2 \text{ psi} \]

\[ f_{1ten} = \text{negative therefore axial load greater than moment, so OK.} \]

Segment 4

\[ V_{14} := \text{adjust}_{4} \cdot (58.5 \text{ psf} \cdot 3.5 \text{ ft} \cdot \text{w} + 23.2 \text{ psf} \cdot 7.5 \text{ ft} \cdot \text{w} + 22.7 \text{ psf} \cdot 5.25 \text{ ft} \cdot \text{w}) = 0.79 \text{ kip} \]

\[ V_{24} := \text{adjust}_{4} \cdot \left(22.7 \text{ psf} \cdot 12.75 \text{ ft} \cdot \text{w} + 21.4 \text{ psf} \cdot \frac{13 \text{ ft}}{2} \cdot \text{w} \right) = 0.68 \text{ kip} \]

\[ V_{4_{total}} := V_{14} + V_{24} = 1.46 \text{ kip} \quad M_{4} := V_{14} \cdot (38.5 \text{ ft}) + V_{24} \cdot (13 \text{ ft}) = 469089 \text{ lbf} \cdot \text{in} \]

\[ N_{v4} := p_{\text{block}} \cdot h_2 \cdot (d_{4} + CR_{4} - 8 \text{ in}) \cdot 0.6 = 11.42 \text{ kip} \]

\[ f_{1ten} := \frac{M_{4} \cdot (y_{\text{bar}_{4}})}{I_{ccl}} - \frac{N_{v4}}{A_{v4}} = 155 \text{ psi} \]

\[ f_{1ten} = 155 \text{ psi} > 86 \text{ psi}, \text{ therefore would need reinforcement if this pier were to contribute to overall shear resistance of the wall.} \]

Note: This is an estimate based on the contributions of all the piers only. A more detailed analysis could be conducted to determine if the spandrel could contribute to the overall moment resistance of the wall. Also, keep in mind that this is an urban building and the wind loads are likely not to be full intensity. Finally the brick most likely contributes to the overall shear resistance of the wall, but it is difficult to calculate the contribution.
Figure 5. Segment 1 Geometry (Method 2)
Assumption modification (Method 3):

Still have issues with wall segment 4 passing unreinforced and segments 1, 2, and 3 have fairly high stress levels and would require a high grout percentage to pass. Try to get an upper bound estimate of the moment capacity of the wall considering no openings and no corner returns.

\[ w := 80 \text{ ft} \quad l := 80 \text{ ft} \]

\[
V_1 := 0.6 \cdot 0.5 \left( 58.5 \text{ psf} \cdot 3.5 \text{ ft} \cdot w + 23.2 \text{ psf} \cdot 7.5 \text{ ft} \cdot w + 22.7 \text{ psf} \cdot 5.25 \text{ ft} \cdot w \right) = 11.95 \text{ kip}
\]

\[
V_2 := 0.6 \cdot 0.5 \left( 22.7 \text{ psf} \cdot 12.75 \text{ ft} \cdot w + 21.4 \text{ psf} \cdot \left( \frac{13 \text{ ft}}{2} \right) \cdot w \right) = 10.28 \text{ kip}
\]

\[
V_{\text{total}} := V_1 + V_2 = 22.23 \text{ kip} \quad M := V_1 \cdot (38.5 \text{ ft}) + V_2 \cdot (13 \text{ ft}) = 7125390 \text{ lbf \cdot in}
\]

\[
N_v := p_{\text{block}} \cdot h_2 \cdot 1 \cdot 0.6 = 68.54 \text{ kip}
\]

\[
I := \left( \frac{2.5 \text{ in} \cdot 2}{12} \right) \cdot \left( \frac{1}{3} \right)^3 = 368640000 \text{ in}^4 \quad A_v := (2.5 \text{ in} \cdot 2) \cdot 1
\]

\[
f_{\text{ten}} := -5 \text{ psi, therefore OK as unreinforced.}
\]

The upperbound estimate shows that if all openings were removed and the wall behaved as a single segment there would be adequate moment capacity.
Member Capacities: Column Pad Footing

Review Bearing Capacity of Frame Line 3 Pad Footings

Notes:
- Bearing capacity is only reviewed preliminarily in this calculation.
- Footing strength is not reviewed, because limited reinforcing data is available on the design drawings making it difficult to determine, with any level of confidence the composition of the footing.

![Figure 1. Frame line 3 pad footing](image)

Weight of Footing

\[\gamma_c := \frac{150 \text{ lb}}{\text{ft}^3}\]

\[P_{\text{footing}} := \gamma_c \cdot (8.67 \text{ ft} \cdot 8.67 \text{ ft} \cdot 2 \text{ ft} + 3.5 \text{ ft} \cdot 3.5 \text{ ft} \cdot 1 \text{ ft}) = 24388 \text{ lb}\]

\[P_{\text{footing}} := \frac{P_{\text{footing}}}{8.67 \text{ ft} \cdot 8.67 \text{ ft}} = 324 \frac{\text{lb}}{\text{ft}^2}\]

\[A_{\text{footing}} := 8.67 \text{ ft} \cdot 8.67 \text{ ft} = 75.2 \text{ ft}^2\]

Existing Building Footing Load

\[P_{\text{asd1}} := 182000 \text{ lb}\]

\[q_{\text{asd}} := \frac{P_{\text{asd1}} + P_{\text{footing}}}{A_{\text{footing}}} = 2745.7 \frac{\text{lb}}{\text{ft}^2}\]

2-Story Extended Building Footing Load (Two-Story Corner Bearing)

\[P_{\text{asd2}} := 278900 \text{ lb}\]

\[q_{\text{asd}} := \frac{P_{\text{asd2}} + P_{\text{footing}}}{A_{\text{footing}}} = 4034.8 \frac{\text{lb}}{\text{ft}^2}\]
Member Capacities: Roof Trusses

Top Chord Strength (Controlling Member)

Member Specifications:

- (2)-8 x 8 x 7/8" L shape with 18"x 3/4" flate plate, bolted
- A9 Steel

Assumptions:

- Assume that the strength of the connection points exceeds that of the member.
- Assume plate and angles act compositely.
- Angles are idealized for purposes of calculating section properties.

![Figure 1. Top chord dimensions](image)

Section Properties (Pieces)

**Part I**

\[ A_1 := 0.75 \text{ in} \cdot 18 \text{ in} = 13.5 \text{ in}^2 \]

\[ I_{yy1} := \frac{18 \text{ in} \cdot (0.75 \text{ in})^3}{12} = 0.63 \text{ in}^4 \]

\[ I_{xx1} := \frac{0.75 \text{ in} \cdot (18 \text{ in})^3}{12} = 365 \text{ in}^4 \]

**Part II**

\[ A_2 := \frac{7}{8} \text{ in} \cdot 8 \text{ in} = 7 \text{ in}^2 \]

\[ I_{yy2} := \frac{0.875 \text{ in} \cdot (8 \text{ in})^3}{12} = 37.3 \text{ in}^4 \]

\[ I_{xx2} := \frac{8 \text{ in} \cdot (0.875 \text{ in})^3}{12} = 0.45 \text{ in}^4 \]
Part III

\[ A_3 := (8 \text{ in} - 0.875 \text{ in}) \cdot 0.875 \text{ in} = 6.23 \text{ in}^2 \]

\[ I_{yy3} := \frac{(8 \text{ in} - 0.875 \text{ in}) \cdot (0.875 \text{ in})^3}{12} = 0.4 \text{ in}^4 \]

\[ I_{xx3} := \frac{0.875 \text{ in} \cdot (8 \text{ in} - 0.875 \text{ in})^3}{12} = 26.4 \text{ in}^4 \]

Section Properties (Whole Part)

\[ y_1 := 0.375 \text{ in} \quad y_2 := 4.75 \text{ in} \quad y_3 := 1.19 \text{ in} \quad A_8 := A_1 + 2 \cdot A_2 + 2 \cdot A_3 = 40 \text{ in}^2 \]

\[ Y_{\text{bar}} := \frac{A_1 \cdot y_1 + A_2 \cdot y_2 + A_3 \cdot y_3 \cdot 2}{A_1 + 2 \cdot A_2 + 2 \cdot A_3} = 2.16 \text{ in} \]

\[ I_{xx} := I_{xx1} + 2 \left( I_{xx2} + A_2 \left( \frac{0.375 \text{ in}}{2} + \frac{0.875 \text{ in}}{2} \right)^2 \right) + 2 \left( I_{xx3} + A_3 \left( \frac{0.375 \text{ in}}{2} + 0.875 \text{ in} + \frac{7.13 \text{ in}}{2} \right)^2 \right) \]

\[ I_{xx} = 690.6 \text{ in}^4 \]

\[ I_{yy} := I_{yy1} + A_1 \left( 2.16 \text{ in}^2 - \frac{3 \text{ in}}{8} \right)^2 + 2 \left( I_{yy2} + A_2 \left( 4.75 \text{ in} - 2.16 \text{ in} \right)^2 \right) + 2 \left( I_{yy3} + A_3 \left( 0.97 \text{ in} \right)^2 \right) \]

\[ I_{yy} = 224.8 \text{ in}^4 \]
Member Capacities: Roof Trusses

\[ r_{xx} := \sqrt{\frac{I_{xx}}{A_g}} = 4.16 \text{ in} \quad r_{yy} := \sqrt{\frac{I_{yy}}{A_g}} = 2.37 \text{ in} \]

Calculate Controlling Effective Length

\[ K_y := 1.0 \quad L_y := 7.75 \text{ ft} \quad K_x := 0.65 \quad L_x := 15.5 \text{ ft} \]

\[ \frac{K_y \cdot L_y}{r_{yy}} = 39.2 \quad \text{* Slenderness along "y" axis controls} \]

\[ \frac{K_x \cdot L_x}{r_{xx}} = 29.1 \]

Check Compactness (AISC SCM 14th Table B4.1a)

\[ E_s := 29000 \text{ ksi} \quad f_y := 30 \text{ ksi} \]

Part I (stiffened)

\[ b := 12 \text{ in} \quad t := 0.75 \text{ in} \]

\[ \frac{b}{t} = 16 \leq 1.4 \cdot \sqrt{\frac{E_s}{f_y}} = 43.5 \quad \text{therefore, non-slender O.K for compression member} \]

Part II (stiffened)

\[ \text{O.K. by inspection.} \]

Part III (unstiffened)

\[ b := 8 \text{ in} \quad t := 0.875 \text{ in} \]

\[ \frac{b}{t} = 9.1 \leq 0.45 \cdot \sqrt{\frac{E_s}{f_y}} = 14 \quad \text{therefore, non-slender O.K for compression member} \]
Member Capacities: Roof Trusses

Design Strength (AISC SCM 14th ed.)

- Non-slender, apply section E3.
- As a note sections E6 and E4 may also apply and should be checked in a design application.

\[ 4.71 \cdot \sqrt{\frac{E_s}{f_y}} = 146.4 \geq \frac{K_y \cdot L_y}{r_{yy}} = 39.2 \] therefore, inelastic buckling will occur

\[
F_e := \frac{\pi^2 \cdot E_s}{2} = 186.1 \text{ksi} \quad F_{cr} := \left( \frac{f_y}{F_e} \right) \cdot f_y = 28 \text{ksi} \quad \phi_c := 0.90
\]

\[ \phi_c p_n := \phi_c \cdot F_{cr} \cdot A_g = 1009 \text{kip} \]

Bottom Chord Strength (Controlling Member)

Member Specifications:

- (2)-8 x 8 x 7/8" L shape with 18"x 3/4" flate plate, bolted
- A9 Steel

Figure 1. Top chord dimensions
Section Properties (Pieces)

Part I

\[ A_1 := 0.625 \text{ in} \cdot 18 \text{ in} = 11.3 \text{ in}^2 \]

\[ I_{yy1} := \frac{18 \text{ in} \cdot (0.625 \text{ in})^3}{12} = 0.37 \text{ in}^4 \]

\[ I_{xx1} := \frac{0.625 \text{ in} \cdot (18 \text{ in})^3}{12} = 304 \text{ in}^4 \]

Part II

\[ A_2 := \frac{3}{4} \text{ in} \cdot 8 \text{ in} = 6 \text{ in}^2 \]

\[ I_{yy2} := \frac{0.75 \text{ in} \cdot (8 \text{ in})^3}{12} = 32 \text{ in}^4 \]

\[ I_{xx2} := \frac{8 \text{ in} \cdot (0.75 \text{ in})^3}{12} = 0.28 \text{ in}^4 \]

Part III

\[ A_3 := (8 \text{ in} - 0.75 \text{ in}) \cdot 0.75 \text{ in} = 5.44 \text{ in}^2 \]

\[ I_{yy3} := \frac{(8 \text{ in} - 0.75 \text{ in}) \cdot (0.75 \text{ in})^3}{12} = 0.25 \text{ in}^4 \]

\[ I_{xx3} := \frac{0.75 \text{ in} \cdot (8 \text{ in} - 0.75 \text{ in})^3}{12} = 23.8 \text{ in}^4 \]

Figure 2. Top chord, location of centroids
Section Properties (Whole Part)

\[ y_1 := 0.313 \text{ in} \quad y_2 := 4.63 \text{ in} \quad y_3 := 1.00 \text{ in} \quad A_g := A_1 + 2 \cdot A_2 + 2 \cdot A_3 = 34.1 \text{ in}^2 \]

\[ Y_{\text{bar}} := \frac{A_1 \cdot y_1 + A_2 \cdot y_2 \cdot 2 + A_3 \cdot y_3 \cdot 2}{A_1 + 2 \cdot A_2 + 2 \cdot A_3} = 2.05 \text{ in} \]

\[ I_{\text{xx}} := I_{\text{xx}1} + 2 \cdot \left( I_{\text{xx}2} + A_2 \cdot (0.563 \text{ in})^2 \right) + 2 \cdot \left( I_{\text{xx}3} + A_3 \cdot (4.63 \text{ in})^2 \right) = 588.9 \text{ in}^4 \]

\[ I_{\text{yy}} := I_{\text{yy}1} + A_1 \cdot (1.74 \text{ in})^2 + 2 \cdot \left( I_{\text{yy}2} + A_2 \cdot (2.58 \text{ in})^2 \right) + 2 \cdot \left( I_{\text{yy}3} + A_3 \cdot (1.05 \text{ in})^2 \right) \]

\[ I_{\text{yy}} = 190.8 \text{ in}^4 \]

\[ r_{\text{xx}} := \sqrt{\frac{I_{\text{xx}}}{A_g}} = 4.15 \text{ in} \quad r_{\text{yy}} := \sqrt{\frac{I_{\text{yy}}}{A_g}} = 2.36 \text{ in} \]

Check Capacity of Bottom Chord Tension Member

- Assume (2) rows of 3/4” Rivets for net section fracture calculations.
- Calculate for gross section yielding and net section fracture.
- Block shear will no be considered for this calculation.

**Gross Section Yield**

\[ \phi_{ty} := 0.90 \quad f_y = 30000 \text{ psi} \quad A_g = 34.1 \text{ in}^2 \quad \text{Dia} := \frac{3}{4} \text{ in} + \frac{1}{8} \text{ in} = 0.875 \text{ in} \]

\[ \phi_{tP_{ny}} := \phi_{ty} \cdot f_y \cdot A_g = 921 \text{ kip} \quad x_{\text{bar}} := 0.5 \cdot 0.375 \text{ in} = 0.188 \text{ in} \quad L := 8 \text{ in} \]

**Net Section Fracture**

\[ A_n := A_g - 2 \cdot (\text{Dia} \cdot 0.75 \text{ in}) = 32.8 \text{ in}^2 \quad U := 1 - \frac{x_{\text{bar}}}{L} = 0.977 \]

\[ A_e := U \cdot A_n = 32.04 \text{ in}^2 \quad F_u := 55000 \text{ psi} \quad \phi_{tf} := 0.75 \]

\[ \phi_{tP_{nr}} := \phi_{tf} \cdot F_u \cdot A_e = 1322 \text{ kip} \]
Member Capacities: Roof Trusses

Check Tensile Strength of Diagonal Member

- Check 2L 6"x6"x3/4"
- Engineering properties from SCM 14th edition.

\[ \phi_{ty} := 0.90 \quad f_y = 30000 \text{ psi} \quad A_g := 2 \cdot 8.46 \text{ in}^2 = 16.9 \text{ in}^2 \quad \text{Dia} := \frac{3}{4} \text{ in} + \frac{1}{8} \text{ in} = 0.875 \text{ in} \]

\[ \phi_t P_{ny} := \phi_{ty} \cdot f_y \cdot A_g = 457 \text{ kip} \quad x_{bar} := 0.5 \cdot 0.375 \text{ in} = 0.188 \text{ in} \quad L := 8 \text{ in} \]

Net Section Fracture

\[ A_n := A_g - 2 \cdot (\text{Dia} \cdot 0.75 \text{ in}) = 15.6 \text{ in}^2 \quad U := 1 - \frac{x_{bar}}{L} = 0.977 \]

\[ A_e := U \cdot A_n = 15.24 \text{ in}^2 \quad F_u := 55000 \text{ psi} \quad \phi_{tf} := 0.75 \]

\[ \phi_t P_{nf} := \phi_{tf} \cdot F_u \cdot A_e = 629 \text{ kip} \]

Check Compressive Strength of Vertical Strut

- Check 2L 6"x6"x1/2"

Calculate Controlling Effective Length

\[ L_x := 7.5 \text{ ft} \quad L_y := L_x = 7.5 \text{ ft} \quad r_{xx} := 2.63 \text{ in} \quad r_{yy} := 1.86 \text{ in} \]

\[ K_y := 1.0 \quad K_x := 1.0 \]

\[ \frac{K_y \cdot L_y}{r_{yy}} = 48.4 \quad \text{* Slenderness along } \''y'' \text{ axis controls} \quad \frac{K_x \cdot L_x}{r_{xx}} = 34.2 \]

Check Compactness (AISC SCM 14th Table B4.1a)

- All leg the same and unstiffened.

\[ E_s := 29000 \text{ ksi} \quad f_y := 30 \text{ ksi} \quad b := 6 \text{ in} \quad t := 0.50 \text{ in} \]

\[ \frac{b}{t} = 12 \leq 0.45 \sqrt{\frac{E_s}{f_y}} = 14 \quad \text{therefore, non-slender O.K for compression member} \]
Member Capacities: Roof Trusses

**Design Strength (AISC SCM 14th ed.)**

- Non-slender, apply section E3.
- As a note sections E6 and E4 may also apply and should be checked in a design application.

\[
4.71 \cdot \sqrt{\frac{E_s}{f_y}} = 146.4 \geq \frac{K_y \cdot L_y}{r_{yy}} = 48.4 \quad \text{therefore, inelastic buckling will occur}
\]

\[
F_e := \frac{\pi^2 \cdot E_s}{(K_y \cdot L_y)} = 122.2 \text{ ksi} \quad F_{cr} := \left( \frac{f_y}{F_e} \right) \cdot f_y = 27.1 \text{ ksi} \quad \phi_c := 0.90
\]

\[
\phi_{cp} := \phi_c \cdot F_{cr} \cdot A_g = 412 \text{ kip}
\]
Appendix I

Module Definition
Description of Calculation:

- The purpose of this calculation is to estimate, empirically, the modulus of elasticity of the gypsum sheathed 8'-0" x 8'-0" wall coupon used in the racking test described in ASTM E-72. The calculated modulus of elasticity will be compared to one which is calculated from the actual test data of the coupon (5/8" Dense-Glass Gold, 2003). The two are compared and a value is selected for use in SAP modeling of the wall assembly and ultimately in the ETAB model of the expanded building.
- Description of Wall Assembly: The wall assembly consists of 2x4 Southern Yellow Pine studs spaced at 16" O.C. sheathed with 5/8" Dense Glass Gold Gypsum sheathing. The sheathing is attached and the wall was tested per ASTM E-72.
- For the purposes of modeling the assembly the 4 1/8" thick wall will be idealized by a 5/8" thin-shell.

Assumptions:

- A similar coupon constructed from cold-formed steel studs would behave similar to the wood framed coupon as far as racking is concerned.
- Poisson's ratio = 0.25 and \( G = 0.4 \cdot E \)
- In regards to shear deflection the wall will be simulated most accurately by providing fixed constraints at both the top and base.

Rigidity Calculation:

- \( h = b, A = b \cdot t = h \cdot t, l = (t \cdot b^3)/12 = (t \cdot h^3)/12 \)
- Delta = \( (P \cdot h^3)/(12 \cdot E \cdot I) + (1.2 \cdot P \cdot h)/(A \cdot G) \);

\[
\nu := 0.25 \quad G := \frac{E}{2 \cdot (1 + \nu)} \rightarrow 0.4 \cdot E
\]

\[
P \cdot h^3 + \frac{1.2 \cdot P \cdot h}{h \cdot t \cdot G} \Delta \text{ solve } P \rightarrow 0.25 \cdot E \cdot t \cdot \Delta
\]

Considering \( P = k \cdot \Delta \) and \( k \) in this instance = 0.25*E*t; rearranging gives provides:

\[
t := \frac{5}{8} \text{ in} \quad k_{avg} := 3674 \frac{lb}{\text{in}} \quad k := k_{avg} \quad 0.25 \cdot E \cdot t - k \text{ solve } E \rightarrow 23153.6 \cdot \frac{lb}{\text{in}^2}
\]

Therefore using the average \( k \) value calculated graphically from the Timber Products racking test empiracally \( E = 23,514 \) psi. Additionally solving for \( E \) provides the following relationship:

\[
\frac{P \cdot h^3}{12 \cdot E \cdot \left( \frac{t \cdot h^3}{12} \right)} + \frac{1.2 \cdot P \cdot h}{h \cdot t \cdot G} \Delta \text{ solve } E \rightarrow \frac{6.4 \cdot P}{\text{in} \cdot \Delta}
\]

\[
E(P, \Delta) := \frac{6.4 \cdot P}{\Delta \cdot \text{in}} \quad \text{(Eq. #1)}
\]
Raw Test Data (Timber Products Inspections, 1990)

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<th>Δ_2 (in)</th>
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Calculate Modulus of Elasticity

- Calculate Modulus of Elasticity using Eq. #1; Calculate at each point and average to compare with the empirical calculation that was based on the average stiffness calculated graphically.

\[
E_1 := \overrightarrow{E(P_1, \Delta_i)} \quad E_2 := \overrightarrow{E(P_2, \Delta_2)} \quad E_3 := \overrightarrow{E(P_3, \Delta_3)}
\]

\[
\begin{bmatrix}
40000 & 35887.9 & 41290.3 \\
40000 & 36374.1 & 51200 \\
38787.9 & 35359.1 & 32268.9 \\
39183.7 & 34133.3 & 35111.1 \\
39811 & 33684.2 & 36994.2 \\
40506.3 & 33382.1 & 39384.6 \\
40000 & 34594.6 & 38620.7 \\
26826.3 & 34594.6 & 38646.2 \\
40031.9 & 34594.6 & 37524.4 \\
36000 & 37572.1 & 37101.4 \\
36056.3 & 34522.8 & 36102.6 \\
36476.7 & 33621 & 35455.4
\end{bmatrix}
\]

\[
E_1 = \begin{bmatrix} 139851.9 \text{ psi} \end{bmatrix} \quad E_2 = \begin{bmatrix} 37281.6 \text{ psi} \end{bmatrix} \quad E_3 = \begin{bmatrix} 34028.6 \text{ psi} \end{bmatrix}
\]
Module Definition for Analytical Model

\[ P_1 \ (lb) = k_1 = \text{slope} (\Delta_1, P_1) = 3760.9 \ \frac{lb}{in} \]

\[ P_2 \ (lb) = k_2 = \text{slope} (\Delta_2, P_2) = 3937.3 \ \frac{lb}{in} \]

\[ P_3 \ (lb) = k_3 = \text{slope} (\Delta_3, P_3) = 3324.4 \ \frac{lb}{in} \]
Module Definition for Analytical Model

Compute Averages of Both Methods and Compare

Average Stiffness Calculation

Provided

\[ k_{\text{avg}} := \text{mean} (k_1, k_2, k_3) = 3674.2 \ \frac{\text{lbf}}{\text{in}} \]

\[ E_k := 23514 \ \text{psi} \]

Using the Empirically Derived Eq. #1

\[ E_{1\text{avg}} := \text{mean} (E_1) = 39736.8 \ \text{psi} \]
\[ E_{2\text{avg}} := \text{mean} (E_2) = 33183.8 \ \text{psi} \]

\[ E_{3\text{avg}} := \text{mean} (E_3) = 33176.9 \ \text{psi} \]
\[ E_e := \text{mean} (E_1, E_2, E_3) = 35138.9 \ \text{psi} \]

Average of Both Methods

\[ E := \text{mean} (E_k, E_e) = 29326.5 \ \text{psi} \]

*Conclude that the range for the actual Modulus of Elasticity is between 23,514 psi and 35,139 psi. Correlate test data to SAP wall model to refine estimate for use in ETABS model.*
Module Definition for Analytical Model

SAP Model (Using \( E = 31300 \) psi and poissons ratio = 0.25)

\[
P_s \quad \Delta_s
\]

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\[
k_s := \text{slope} (\Delta_s, P_s) = \frac{5633.7 \text{ lb}}{237 \text{ in}} \quad \frac{k_s}{k_{av}} = 1.53
\]

\*

Note that the response of the SAP model is linear and approximately 53% stiffer than average stiffness of the non-linear coupon testing response.

Breaking the data set for test 2 into two ranges the average stiffness of the low range of loading (E2L) and the high range of loading (E2H) is as follows:

\[
P_{2L} := \text{submatrix} (P_2, 0, 11, 0, 0) \quad \Delta_{2L} := \text{submatrix} (\Delta_2, 0, 11, 0, 0)
\]

\[
k_{2L} := \text{slope} (\Delta_{2L}, P_{2L}) = \frac{5410.7 \text{ lb}}{\text{in}}
\]

\[
P_{2H} := \text{submatrix} (P_2, 12, 28, 0, 0) \quad \Delta_{2H} := \text{submatrix} (\Delta_2, 12, 28, 0, 0)
\]

\[
k_{2L} := \text{slope} (\Delta_{2H}, P_{2H}) = \frac{3006.4 \text{ lb}}{\text{in}}
\]

The stiffness in the low range of the data set which would be representative of the service conditions of the module walls tracks with the average modulus of elasticity calculated (E). This value will be used in ETABS modeling. Please note however that using this Modulus of Elasticity in load conditions near ultimate capacity is likely lower and may not accurately represent the response of the wall panels.
Module Definition for Analytical Model

Figure 1 shows the SAP model that was used to test the empirically derived Modulus of Elasticity. The thin shell is meshed into approximate 6"x6" 4-noded elements. In order to measure only the shear deflection of the coupon the base was fixed in all direction and the top was fixed with translation in the right to left direction released.

Define the Weight of the Wall Assembly for use in ETABS Modeling

- 8'x8'x4.625" Coupon
- Dead Load:

\[
\begin{align*}
\text{Material} & \quad \text{Weight} \\
\text{CFS Track and Studs} & \quad 1.70 \text{ lb/ft} \\
5/8" \text{ Gypsum Sheathing} & \quad 2.75 \text{ psf} \\
1/2" \text{ Gypsum Wallboard} & \quad 2.20 \text{ psf}
\end{align*}
\]

\[
V = \frac{H \cdot L \cdot t}{24.7 \cdot 3} = 24.7 \text{ ft}^3
\]

\[
W_t := 1.7 \frac{lbf}{ft} \cdot (7 \cdot H + 2 \cdot L) + (2.75 \text{ psf} + 2.20 \text{ psf}) \cdot L \cdot H = 439.2 \text{ lbf}
\]

Walls in Etabs are modeled with 5/8" thick thin shells to represent the rigidity of the gympum sheathing only based on the ITS test data. Determine appropriate weight to account for change in actual thickness from 4.625" to a 5/8" model thickness.

\[
V := \frac{5}{8} \text{ in} \cdot H \cdot L = 3.3 \text{ ft}^3 \\
\gamma_{\text{wall}} := \frac{W_t}{V} = 131.8 \text{ pcf}
\]
Define Stiffness of Module Floor System

- References:
  1. NAHB, 1999.
  2. Ghugal and Sharma, 2011

Description of Calculation

- Calculate Modulus of Elasticity for the floor system tested in Reference 1. It is not clear in the test setup whether the floor system restraint are most accurately represented by a fixed support or pinned support. The analytical expression for both conditions were developed (Ref. 2) and a SAP model created to test which value will likely be more accurate for use in ETAB module modeling.

Testing Data (Reference 1)

- Test measured midspan deflection for a 12’x24’ floor system.
- Constructed with 8” cold formed steel joists and 22/32” OSB.
- Estimate load and deflection values from Figure 2 in order to calculate diaphragm stiffness.

![Load-Deflection Curve - 8" Joists](image)

Figure 2. Load deflection data for FD8-43-1 sample (Reference 1)
Because of highly non-linear response of diaphragm, estimate an appropriate load range.

Expected module diaphragm load based on wind load calculations.

\[ P_{w\text{max}} := 27 \text{ psf} \quad h_{\text{module}} := 10 \text{ ft} \quad w_{\text{wind}} := P_{w\text{max}} \cdot h_{\text{module}} = 270 \frac{\text{lbf}}{\text{ft}} \]

Expected module diaphragm load based on wind load calculations.

\[ L_f := 24 \text{ ft} \quad \text{(Floor specimen length)} \quad w(x, y) := \frac{\text{mean}(x, y)}{L_f} \]

\[
\begin{bmatrix}
\text{title} := & \left[ \begin{array}{c}
\text{“w”} \\
\text{“plf”} \\
\text{“kip”} \\
\text{“in.”}
\end{array} \right] \\
\text{w} := & \begin{bmatrix}
w(0 \text{ kip}, 16 \text{ kip}) \\
w(16 \text{ kip}, 22.5 \text{ kip}) \\
w(22.5 \text{ kip}, 26.5 \text{ kip}) \\
w(26.5 \text{ kip}, 28.0 \text{ kip}) \\
w(28.0 \text{ kip}, 29.0 \text{ kip})
\end{bmatrix} = \begin{bmatrix}
333.3 \\
802.1 \\
1020.8 \\
1135.4 \\
1187.5
\end{bmatrix} \quad \text{plf} \quad K := \begin{bmatrix}
K_1 \\
K_2 \\
K_3 \\
K_4 \\
K_5
\end{bmatrix} = \begin{bmatrix}
32 \\
13 \\
8 \\
3 \\
2
\end{bmatrix} \quad \frac{\text{kip}}{\text{in}}
\end{bmatrix}
\]
Rigidity Calculation (Reference 2):

- Formulation for deflection @ midspan for both a simple and fixed end boundary condition deep beam subjected to distributed load.
- $t_f$=thickness of OSB, $h_f$=Width of Floor System (depth of beam)
- $L_f$=Length of Floor System
- $K$=Stiffness
- $I=(t_f*h_f^3)/12$, $q=P_{fl}/L_f$

Simple Supports:

\[
\delta_{fl} = \left[\frac{5*q*L^4}{384*E*I}\right] \left[1 + 1.92*(1+\nu_f)*(h_f^2/L_f^2)\right]
\]

Fixed Supports:

\[
\delta_{fl} = \left[\frac{q*L^4}{384*E*I}\right] \left[1 + 9.60*(1+\nu_f)*(h_f^2/L_f^2)\right]
\]

\[\nu_f := 0.25 \quad t_f := \frac{23}{32} \text{ in} \quad h_f := 12 \text{ ft} \quad L_f = 24 \text{ ft}\]

Simple boundary condition (Reference 4)

\[
E(K) := \frac{2.78*K}{\text{in}} \quad \text{(Eq. #2)} \quad E(K) = \begin{bmatrix} 88960 \\ 36140 \\ 22240 \\ 8340 \\ 5560 \end{bmatrix} \text{ psi}
\]

For a fixed boundary condition (Reference 4)

\[
E(K) := \frac{1.39*K}{\text{in}} \quad \text{(Eq. #3)} \quad E(K) = \begin{bmatrix} 44480 \\ 18070 \\ 11120 \\ 4170 \\ 2780 \end{bmatrix} \text{ psi}
\]

$E_{fl} := \text{mean}(88960 \text{ psi, 44480 psi}) = 66720 \text{ psi}$
Discussion:

Note that the expected maximum load of 270 plf is less than the 0.5" deflection reading on Figure 2. The mean load of 333 plf is closest to this value. Expect that the stiffness for this range would be appropriate to use for the module modeling. Assume that the actual boundary condition of the floor specimen is somewhere between a pinned and fixed support. Therefore, choose the median stiffness value for the fixed and pinned condition. Confirm and refine with SAP model.

Figure 3. SAP model; floor diaphragm pinned @ center of floor system along the longitudinal axis, but remaining edges free to translate longitudinally.

Figure 4. SAP model; floor diaphragm both translation and rotation restrained @ edges.
Discussion regarding SAP model:

Tests were conducted with boundary conditions shown in Figures 3 and Figures 4. E=60,000 psi produced average deflection values similar to the test data, when considering the average of the two boundary conditions. The model floor system was loaded with 16 kip (in the same manner as the test sample). The target deflection was 0.5" as in the test data.

- Pinned condition: Mid-span deflection = 0.673"
- Fixed condition: Mid-span deflection = 0.318"
- Average: Mid-span deflection = 0.496"

The average modeled mid-span deflection of 0.496" is comparable to the tested mid-span deflection of the assembly (0.500"). Use E=60,000 psi for the module floor systems. As with the gypsum wall panels the floor system will be represented by the floor sheathing only. It will be modeled with a 23/32" thick thin shell. The shell will have no weight associated with it. A dead load of 10 psf will be added as a surface load to represent the weight of the floor assembly.

Development of a Test Module

- In general, the purpose of the modules to be developed to be used for the expansion is not to determine internal loading of the module components, but to transfer the external loads accurately to the existing structure so that the effects of the expansion on the existing structure may be assessed.
- It is understood that the behavior of light-framed diaphragm systems is complex and the framed walls and floors typically exhibit orthotropic properties rather than the isotropic properties being proposed in this experiment. If accurate internal loadings and deformations were required for the experiment than this type of rigidity modeling would not be appropriate.
- Because of the nature of the research, modeling the modules as shells is appropriate. By doing this information regarding gravity load transfer, Drift and lateral load transfer to the transfer structure and existing building can be gathered from the model and reviewed.
- The next step was to develop a complete module model to determine whether the wall and floor systems developed previously would function as expected when assembled to form the modules.
Module Definition for Analytical Model

Module ETABS Test Specimen

Description of Model:

- $p_1 := 10 \text{ psf}$ (Dead)
- $p_2 := 20 \text{ psf}$ (Wind E/W Loading, WES)
- $p_3 := 20 \text{ psf}$ (Wind N/S Loading, WNS)
- $L := 60 \text{ ft}$
- $H := 20 \text{ ft}$
- $W := 12 \text{ ft}$
- $\gamma_{\text{wall}} := 131.8 \text{ pcf}$

Figure 5. Test Module Specifications
Module Definition for Analytical Model

Figure 6. Test module isometric in its deformed shape due to the Wind E/W load case

Module Construction:

- Walls and roofs are constructed from the shell elements properties determined previously.
  - Walls and floors meshed at approximately 16” widths to simulate typical stud framed reactions.
  - Floor meshed into smaller squares to simulate load sharing (transferred through screws and diaphragm out of plane bending) and tie floor system into endwalls.
  - Semi-rigid diaphragms applied to both the 2nd floor and roof to define the extents of the diaphragm.
- Structural steel transfer grid defined (see page 3) and modules tied into grid.
- Pinned reactions specified to simulate field connections to existing building.
- The loads shown in Figure 5 Elevation A were applied to the test module along with a 3 psf cladding load.
Figure 7. (a) Floor system at transfer grid. (b) Zoomed in view of link connection.

Transfer Grid:

- Figure 7(a) shows a more detailed view of the module floor system.
- Notice on the indirect connection of the module to the steel frame on the right hand side of Figure 7(a). This will simulate the connection to the transfer grid in locations where modules are placed side by side. It is important to simulate that the walls and floors are rigidly connected (bottom plates fasten at intervals to steel framing by screws or pneumatic fasteners) to the steel transfer structure, but also disconnect the adjacent modules from each other. The separation was accomplished through a rigid link as shown in both Figures 7(a) and (b).
- Floor shells are special one-way span, therefore distribute gravity load in the left to right direction only.
- W12x53 were chosen for the steel framing elements because of their convenient dimensions as well as their adequate capacity (See calculations on page 4).
- Referencing the Plan View in Figure 5: Members from nodes 1-5, 2-6, 3-7 and 4-8 are representative of the steel framing in line with the main roof trusses, whereas the orthogonal members are those that are in line with the diaphragm trusses.
- Ends were released to simulate simple connections.
Transfer Grid Beam Calculations:

- The transfer beam members running parallel to the main trusses are continuously supported by the main truss structural members therefore are not experiencing any bending due to the module loads.

- However, beams in-line with the main trussed are assumed to act compositely with the main truss and will experience axial loading. The members are assumed to be through-bolted to the main truss top-chord to strengthen the member.

- It is assumed that the diaphragm trusses running orthogonal to the main trusses do not support gravity loads.

- Walls above the diaphragm truss require continuous support therefore a structural member must be placed beneath the walls. Transfer beam A is sized for this purpose. The depth of the W-shaped grid is dependent on the depth of this member.

![Figure 8. Module schematic used to size transfer beam A](image)

Load estimates used to size transfer beam A:

- Dead load: 38 psf for structural components and 20 psf surcharge for mechanical equipment and furniture.
- Assume load combination 1.2D + 1.6L + 0.5S controls.
Module Definition for Analytical Model

\[ p_s := 33 \text{ psf} \quad p_L := 40 \text{ psf} \quad p_D := 38 \text{ psf} + 20 \text{ psf} \]

\[ W := 15.5 \text{ ft} \quad L := 19.5 \text{ ft} \quad d := 12.1 \text{ in} \quad b_t := 10 \text{ in} \]

\[ w_D := p_D \cdot 4 \cdot \frac{W}{2} = 1798 \frac{\text{lbf}}{\text{ft}} \quad w_L := p_L \cdot 4 \cdot \frac{W}{2} = 1240 \frac{\text{lbf}}{\text{ft}} \quad w_s := p_s \cdot 2 \cdot \frac{W}{2} = 512 \frac{\text{lbf}}{\text{ft}} \]

\[ w_u := 1.2 \cdot w_D + 1.6 \cdot w_L + 0.5 \cdot w_s = 4397 \frac{\text{lbf}}{\text{ft}} \]

\[ M_u := \frac{w_u \cdot (L^2)}{8} = 209 \text{ kip ft} \]

\[ L_b := 0 \quad \text{(Top chord fully braced by modules)} \quad C_b := 1 \]

Try W12x53, from the Steel Construction Manual 14th edition

\[ \phi M_n := 292 \text{ kip ft} \quad \geq \quad M_n = 209 \text{ kip ft} \quad \text{Therefore OK for strength, check deflection under live loading} \]

\[ I_x := 425 \text{ in}^4 \quad \quad E_s := 29000 \text{ ksi} \]

\[ \Delta := \frac{5 \cdot (w_s + w_L) \cdot L^4}{384 \cdot E_s \cdot I_x} = 0.462 \text{ in} \quad \leq \quad \frac{L}{480} = 0.488 \text{ in} \]
Module Definition for Analytical Model

Reactions (From ETABS Analysis):

<table>
<thead>
<tr>
<th>Row</th>
<th>Node</th>
<th>Case</th>
<th>$F_x$ (kip)</th>
<th>$F_y$ (kip)</th>
<th>$F_z$ (kip)</th>
</tr>
</thead>
<tbody>
<tr>
<td>“0”</td>
<td>“1”</td>
<td>“Dead”</td>
<td>0.195</td>
<td>-0.512</td>
<td>5.665</td>
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<tr>
<td>“1”</td>
<td>“1”</td>
<td>“Wind E/W”</td>
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<td>0.755</td>
<td>-10.32</td>
</tr>
<tr>
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<td>“1”</td>
<td>“Wind N/S”</td>
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<td>-0.191</td>
<td>0.286</td>
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<tr>
<td>“3”</td>
<td>“1”</td>
<td>“D+WNS”</td>
<td>0.2</td>
<td>-0.703</td>
<td>5.95</td>
</tr>
<tr>
<td>“4”</td>
<td>“1”</td>
<td>“D+WES”</td>
<td>-5.012</td>
<td>0.244</td>
<td>-4.656</td>
</tr>
<tr>
<td>“5”</td>
<td>“2”</td>
<td>“Dead”</td>
<td>0.043</td>
<td>-0.134</td>
<td>8.906</td>
</tr>
<tr>
<td>“6”</td>
<td>“2”</td>
<td>“Wind E/W”</td>
<td>-1.402</td>
<td>0.887</td>
<td>0.409</td>
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<td>“7”</td>
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<td>“Wind N/S”</td>
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<td>-0.481</td>
<td>0.139</td>
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</tr>
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<td>“D+WES”</td>
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<td>0.753</td>
<td>9.315</td>
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<tr>
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<td>0.177</td>
<td>8.938</td>
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<tr>
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</tr>
<tr>
<td>“12”</td>
<td>“3”</td>
<td>“Wind N/S”</td>
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<td>-0.73</td>
<td>0.1</td>
</tr>
<tr>
<td>“13”</td>
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<td>“D+WNS”</td>
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<td>-0.553</td>
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</tr>
<tr>
<td>“14”</td>
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<tr>
<td>“15”</td>
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<tr>
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<td>-0.701</td>
<td>-10.191</td>
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<tr>
<td>“17”</td>
<td>“4”</td>
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<td>-1.019</td>
<td>-0.525</td>
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<tr>
<td>“18”</td>
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<td>“D+WNS”</td>
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<td>5.174</td>
</tr>
<tr>
<td>“19”</td>
<td>“4”</td>
<td>“D+WES”</td>
<td>-4.989</td>
<td>-0.169</td>
<td>-4.492</td>
</tr>
<tr>
<td>“20”</td>
<td>“5”</td>
<td>“Dead”</td>
<td>-0.213</td>
<td>-0.519</td>
<td>5.522</td>
</tr>
<tr>
<td>“21”</td>
<td>“5”</td>
<td>“Wind E/W”</td>
<td>-4.828</td>
<td>-0.866</td>
<td>10.269</td>
</tr>
<tr>
<td>“22”</td>
<td>“5”</td>
<td>“Wind N/S”</td>
<td>-0.008</td>
<td>-0.19</td>
<td>0.267</td>
</tr>
<tr>
<td>“23”</td>
<td>“5”</td>
<td>“D+WNS”</td>
<td>-0.22</td>
<td>-0.709</td>
<td>5.789</td>
</tr>
<tr>
<td>“24”</td>
<td>“5”</td>
<td>“D+WES”</td>
<td>-5.041</td>
<td>-1.385</td>
<td>15.791</td>
</tr>
<tr>
<td>“25”</td>
<td>“6”</td>
<td>“Dead”</td>
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<td>-0.152</td>
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<td>“26”</td>
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<td>“Wind E/W”</td>
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<td>-0.832</td>
<td>-0.321</td>
</tr>
<tr>
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<td>“6”</td>
<td>“Wind N/S”</td>
<td>-0.002</td>
<td>-0.475</td>
<td>-0.131</td>
</tr>
<tr>
<td>“28”</td>
<td>“6”</td>
<td>“D+WNS”</td>
<td>-0.04</td>
<td>-0.628</td>
<td>8.766</td>
</tr>
</tbody>
</table>
Module Definition for Analytical Model

"29"  "6"  "D+WES"  −0.635  −0.984  8.314
"30"  "7"  "Dead"  −0.039  0.078  8.636
"31"  "7"  "Wind E/W"  −0.598  0.65  −0.533
"32"  "7"  "Wind N/S"  −0.002  −0.725  0.1
"33"  "7"  "D+WNS"  −0.041  −0.647  8.736
"34"  "7"  "D+WES"  −0.637  −0.728  8.102
"35"  "8"  "Dead"  −0.193  0.529  5.477
"36"  "8"  "Wind E/W"  −4.777  0.885  10.177
"37"  "8"  "Wind N/S"  −0.005  −0.988  −0.499
"38"  "8"  "D+WNS"  −0.198  −0.459  4.978
"39"  "8"  "D+WES"  −4.971  1.414  15.653

Overturning:

• Estimate node 4 as a roller for the purposes of calculating an overturning force.
• Reviewing the Wind E/W load case.
• ETABS result are reasonable, Tc approximately equal to T ETAB.

\[
M_{ot} := (p_2 \cdot H \cdot L) \cdot (0.5 \cdot H) = 78 \text{ ft} \cdot \text{kip} \\
T \cdot W - M_{ot} \xrightarrow{\text{solve, float }, 3} 5.03 \cdot \frac{\text{ft} \cdot \text{kip}}{\text{ft}}
\]

\[
T_c := -20 \text{ kip} \quad C_c := -T_c = 20 \text{ kip} \quad (\text{Resisting Moment})
\]

\[
T_{ETAB} := F_{z_1} + F_{z_6} + F_{z_{11}} + F_{z_{16}} = -19.6 \text{ kip}
\]

<table>
<thead>
<tr>
<th>(5) 10.269K</th>
<th>(6) -0.321K</th>
<th>(7) -0.539K</th>
<th>(8) 10.177K</th>
</tr>
</thead>
<tbody>
<tr>
<td>(1) -10.32K</td>
<td>(2) 0.409K</td>
<td>(3) 0.511K</td>
<td>(4) -10.191K</td>
</tr>
</tbody>
</table>

Figure 9. Base reactions Winds E/W Load Case, Fz
Module Definition for Analytical Model

Qualitatively:

- Referencing Figure 9: The wind load is distributed mostly to the end walls, as would be expected when using a diaphragm/shear wall lateral force resisting system. The shear walls located on the North and South distribute the force from the diaphragm as a force couple to the base support.
- Of interest is the middle reactions. The front two are acting compressively and the rear two are acting in tension. This is different than what was expected. It’s possible that these values are small in relation to the end reactions and should essentially be considered zero.

Base Shear:

- Reviewing the Wind E/W load case.
- ETABS result match calculated results

\[ V_{\text{base}} := p_2 \cdot L \cdot H = 7.8 \text{ kip} \]

\[ V_{\text{ETABS}} := F_{x_1} + F_{x_6} + F_{x_{11}} + F_{x_{16}} + F_{x_{21}} + F_{x_{26}} + F_{x_{31}} + F_{x_{36}} = -24 \text{ kip} \]

Figure 10. Base reactions Winds E/W Load Case, Fx

Qualitatively:

- Referencing Figure 10: The base shear reaction appear reasonable. The majority of load is distributed to the North/South endwalls with the windward side having a slightly higher magnitude.
- Links were used to attach the module to the transfer structure. It is possible that the links considering the geometry, although rigid, are actually less stiff than the direct node restraint used on the windward wall.
Module Definition for Analytical Model

Gravity Loading:

- Reviewing the Dead load case.
- ETABS result match approximately with calculated results

\[ W_{d1} := 3 \cdot (W \cdot L) \cdot p_1 = 9.1 \text{ kip} \]
\[ W_{d2} := 53 \text{ lb/ft} \cdot (2 \cdot L + 4 \cdot W) = 5.4 \text{ kip} \]
\[ W_{d3} := \gamma_{wall} \cdot \frac{5}{8} \text{ in} \cdot (2 \cdot L \cdot H + 2 \cdot W \cdot H) = 9.6 \text{ kip} \]
\[ W_{d4} := 3 \text{ psf} \cdot (2 \cdot L \cdot H + 2 \cdot W \cdot H) = 4.2 \text{ kip} \]
\[ W := W_{d1} + W_{d2} + W_{d3} + W_{d4} = 28230.9 \text{ lbf} \]
\[ W_{ETABS} := F_{z_0} + F_{z_5} + F_{z_{10}} + F_{z_{15}} + F_{z_{20}} + F_{z_{25}} + F_{z_{30}} + F_{z_{35}} = 57.5 \text{ kip} \]

Figure 11. Base reactions Dead Load Case, Fz

Qualatatively:

- Referencing Figure 11: The gravity load distribution appears to be reasonable. The interior reactions are greater than the endwall reactions. This is inline with what should be expected with a greater tributary area.
- As in the base shear qualitative commentary, the reaction in the front are slightly larger than those in the rear. It would appear that the direct node restraints in the front do create a slightly stiffer scenario than the links in the rear.
Module Definition for Analytical Model

Figure 12. Module isometric showing major axis moment diagram on frame members; D+WES

Qualitative commentary regarding Figure 6 and Figure 12:

- Figure 6: Deflected shape appears to be representative of the structure under the given loads. Wind load appears to be distributing from the walls to the diaphragm and the diaphragm appears to be deflecting and distributing to the endwalls appropriately.
- Figure 12:
  - Frame members located along the Y-axis, origin side:
    - Releases are accurately representing the simple connection; zero moment node locations.
    - Middle bay frame member: loaded by gravity loads only, it can be observed that the special one-way span action appears to be functioning. The loads are distributed to those members parallel to the Y-axis. The members parallel to the X-axis have very small moments due to self-weight only.
    - Members on either side of middle bay: This is the windward side. Some uplift appears to be transferring from the wall shell elements to the transfer framing near the ends; noticeable by the negative moment.
  - Frame members located along the Y-axis, opposite side:
    - The links appear to be transferring the load to the transfer frame as intended.
    - The moments are, however, larger in the end spans. This would indicate some load is being transfer by axial rigidity of the wall elements to the beam, rather than pure diaphragm action to the end walls.
Appendix J

Structural Analysis Results; Expanded Building
<table>
<thead>
<tr>
<th>Structural Element</th>
<th>Value</th>
<th>Limits</th>
<th>Members</th>
<th>Load Case</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Frame 3</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Max. SL Deflection: ( S )</td>
<td>0.12&quot;</td>
<td>L/360 ( ^1 ) = 2.58&quot;</td>
<td>Roof Member SL Deflection</td>
<td></td>
</tr>
<tr>
<td>Max. LL Deflection: ( L )</td>
<td>0.23&quot;</td>
<td>L/360 ( ^1 ) = 2.58&quot;</td>
<td>L</td>
<td></td>
</tr>
<tr>
<td>Max. TL Deflection: ( D+S+L )</td>
<td>0.75&quot;</td>
<td>L/240 ( ^1 ) = 3.88&quot;</td>
<td>D+S+L</td>
<td></td>
</tr>
<tr>
<td><strong>Roof Truss Frame 3</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Max. Comp. Force (^2)</td>
<td></td>
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<td></td>
</tr>
<tr>
<td>Top Chord</td>
<td>38K</td>
<td>B426</td>
<td>Max Envelope</td>
<td></td>
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<td>Bottom Chord</td>
<td>6K</td>
<td>B413</td>
<td>Max Envelope</td>
<td></td>
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<td>Vertical Strut</td>
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<td>C134</td>
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<td>Max. Tens. Force</td>
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<td>360K</td>
<td>B427</td>
<td>Max Envelope</td>
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</tr>
<tr>
<td>Diagonal Tie</td>
<td>185K</td>
<td>D11</td>
<td>Max Envelope</td>
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<tr>
<td><strong>Column A3</strong></td>
<td></td>
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<tr>
<td>( L_{bx_{b-1}} ) (ft.)</td>
<td>12</td>
<td></td>
<td></td>
<td>LC2</td>
</tr>
<tr>
<td>( Mu ) (K*ft.)</td>
<td>26.2</td>
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<td></td>
<td></td>
</tr>
<tr>
<td>( Pu ) (K)</td>
<td>283.0</td>
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<td></td>
</tr>
<tr>
<td>( L_{bx_{1,2}} ) (ft.)</td>
<td>13</td>
<td></td>
<td></td>
<td>LC2</td>
</tr>
<tr>
<td>( Mu ) (K*ft.)</td>
<td>8</td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>( Pu ) (K)</td>
<td>226.0</td>
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</tr>
<tr>
<td>( L_{bx_{2,BOT}} ) (ft.)</td>
<td>18</td>
<td></td>
<td></td>
<td>LC2</td>
</tr>
<tr>
<td>( Mu ) (K*ft.)</td>
<td>19.5</td>
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<td></td>
<td></td>
</tr>
<tr>
<td>( Pu ) (K)</td>
<td>170.0</td>
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<tr>
<td>Base Reaction (K)</td>
<td>196.7 (+Z)</td>
<td></td>
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Table 2. Two-Story Wall-Bearing Expansion

<table>
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<th>Structural Element</th>
<th>Value</th>
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<th>Load Case</th>
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<tr>
<td><strong>Frame 3</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Max. SL Deflection: ( S )</td>
<td>0.12”</td>
<td>L/360 ( ^1 ) = 2.58”</td>
<td>Roof Member SL Deflection</td>
<td></td>
</tr>
<tr>
<td>Max. LL Deflection: ( L )</td>
<td>0.36”</td>
<td>L/360 ( ^1 ) = 2.58”</td>
<td>L</td>
<td></td>
</tr>
<tr>
<td>Max. TL Deflection: ( D+S+L )</td>
<td>1.00”</td>
<td>L/240 ( ^1 ) = 3.88”</td>
<td>D+S+L</td>
<td></td>
</tr>
<tr>
<td><strong>Roof Truss Frame 3</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Max. Comp. Force ( 2 )</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Top Chord</td>
<td>56K</td>
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<td>B426</td>
<td>Max Envelope</td>
</tr>
<tr>
<td>Bottom Chord</td>
<td>9K</td>
<td></td>
<td>B413</td>
<td>Max Envelope</td>
</tr>
<tr>
<td>Vertical Strut</td>
<td>183K</td>
<td></td>
<td>C134</td>
<td>Max Envelope</td>
</tr>
<tr>
<td>Max. Tens. Force</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Top Chord</td>
<td>100K</td>
<td></td>
<td>B413</td>
<td>Max Envelope</td>
</tr>
<tr>
<td>Bottom Chord</td>
<td>519K</td>
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<td>B427</td>
<td>Max Envelope</td>
</tr>
<tr>
<td>Diagonal Tie</td>
<td>265K</td>
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<td>D11</td>
<td>Max Envelope</td>
</tr>
<tr>
<td><strong>Column A3</strong></td>
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<td></td>
</tr>
<tr>
<td>( L_{bxx, b-1} (ft.) ):</td>
<td>12</td>
<td></td>
<td></td>
<td>LC2</td>
</tr>
<tr>
<td>( Mu (K*ft.) ):</td>
<td>28</td>
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<td></td>
<td></td>
</tr>
<tr>
<td>( Pu (K) ):</td>
<td>355</td>
<td></td>
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<td></td>
</tr>
<tr>
<td>( L_{bxx, 1-2} (ft.) ):</td>
<td>13</td>
<td></td>
<td></td>
<td>LC2</td>
</tr>
<tr>
<td>( Mu (K*ft.) ):</td>
<td>6</td>
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<td></td>
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<tr>
<td>( Pu (K) ):</td>
<td>300</td>
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</tr>
<tr>
<td>( L_{bxx, 2-BOT} (ft.) ):</td>
<td>18</td>
<td></td>
<td></td>
<td>LC2</td>
</tr>
<tr>
<td>( Mu (K*ft.) ):</td>
<td>24</td>
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<td></td>
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<tr>
<td>( Pu (K) ):</td>
<td>242</td>
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<tr>
<td>Base Reaction (K)</td>
<td>248.2 (+Z)</td>
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<td>ASD2</td>
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### Table 3. Single-Story Corner-Post Bearing Expansion

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<th>Members</th>
<th>Load Case</th>
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<tbody>
<tr>
<td><strong>Frame 3</strong></td>
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<td></td>
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<tr>
<td>Max. SL Deflection:</td>
<td>0.12”</td>
<td>L/360 (^{-1}) = 2.58”</td>
<td></td>
<td>Roof Member SL Deflection</td>
</tr>
<tr>
<td>S</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Max. LL Deflection:</td>
<td>0.22”</td>
<td>L/360 (^{-1}) = 2.58”</td>
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<td>L</td>
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<td>L</td>
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<tr>
<td>Max. TL Deflection:</td>
<td>0.76”</td>
<td>L/240 (^{-1}) = 3.88”</td>
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<td>D+S+L</td>
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<td>D+S+L</td>
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</tbody>
</table>

**Roof Truss Frame 3**

- **Max. Comp. Force** \(^2\)
  - Top Chord: 28K B426 Max Envelope
  - Bottom Chord: 6K B413 Max Envelope
  - Vertical Strut: 134K C134 Max Envelope

- **Max. Tens. Force**
  - Top Chord: 57K B413 Max Envelope
  - Bottom Chord: 377K B427 Max Envelope
  - Diagonal Tie: 195K D11 Max Envelope

**Column A3**

<table>
<thead>
<tr>
<th>Prefix</th>
<th>Value</th>
<th>Limit</th>
<th>Member</th>
<th>Envelope</th>
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<tr>
<td>(L_{bx\ b-1} (ft.):)</td>
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<td>LC2</td>
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<td>(Mu (K*ft.):)</td>
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<td>(Pu (K):)</td>
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<td>(L_{bx\ 1-2} (ft.):)</td>
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<td>LC2</td>
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<td>(Pu (K):)</td>
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<td>(L_{bx\ 2-BOT} (ft.):)</td>
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<td>LC2</td>
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<td>(Mu (K*ft.):)</td>
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<td>(Pu (K):)</td>
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</table>

**Base Reaction (K)** 204.0 (+Z) ASD2
<table>
<thead>
<tr>
<th>Structural Element</th>
<th>Value</th>
<th>Limits</th>
<th>Members</th>
<th>Load Case</th>
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<td><strong>Frame 3</strong></td>
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<td>Max. SL Deflection:</td>
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<td>L/360/1 = 2.58”</td>
<td>Roof Member SL Deflection</td>
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<td>L/360/1 = 2.58”</td>
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<td>Max. TL Deflection:</td>
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<td>D+S+L</td>
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<td><strong>Roof Truss Frame 3</strong></td>
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<tr>
<td>Max. Comp. Force 2</td>
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<tr>
<td>Bottom Chord</td>
<td>0K</td>
<td>B413</td>
<td>Max Envelope</td>
<td></td>
</tr>
<tr>
<td>Vertical Strut</td>
<td>202K</td>
<td>C134</td>
<td>Max Envelope</td>
<td></td>
</tr>
<tr>
<td>Max. Tens. Force</td>
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<td></td>
<td></td>
<td></td>
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<tr>
<td>Top Chord</td>
<td>84K</td>
<td>B413</td>
<td>Max Envelope</td>
<td></td>
</tr>
<tr>
<td>Bottom Chord</td>
<td>568K</td>
<td>B427</td>
<td>Max Envelope</td>
<td></td>
</tr>
<tr>
<td>Diagonal Tie</td>
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<td>Max Envelope</td>
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<td><strong>Column A3</strong></td>
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<tr>
<td>(L_{bx b-1} (ft.)):</td>
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<td>LC2</td>
<td></td>
<td></td>
</tr>
<tr>
<td>(Mu (K*ft.)):</td>
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<td></td>
<td></td>
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<tr>
<td>(Pu (K)):</td>
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<td>(L_{bx 1-2} (ft.)):</td>
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<td>LC2</td>
<td></td>
<td></td>
</tr>
<tr>
<td>(Mu (K*ft.)):</td>
<td>6</td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>(Pu (K)):</td>
<td>339</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(L_{bx 2-BOT} (ft.)):</td>
<td>18</td>
<td>LC2</td>
<td></td>
<td></td>
</tr>
<tr>
<td>(Mu (K*ft.)):</td>
<td>26</td>
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<td>(Pu (K)):</td>
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<tr>
<td><strong>Base Reaction (K)</strong></td>
<td>278.9 (+Z)</td>
<td>ASD2</td>
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<td></td>
</tr>
</tbody>
</table>

**Table Notes:**

1. Plaster ceilings.
2. Composite action with transfer beam above. A portion of the load is now resisted by the transfer beam.
Appendix K

Load Distribution Data
The figure below displays the coordinates that were used for the accompanying axial load distribution graphs.

**Grid Map:**

- Lines 1,2,3,4 and A,B,C,D,E,F are the main grid lines in which the corners of the modules bear. A.5,B.5,C.5,D.5 and E.5 are intermediate bearing locations.
- **Note 1:** Steel W-shape cross-members. These members are only used for the wall bearing module expansions to bridge between the trusses. This is necessary because it is assumed that the diaphragm trusses running perpendicular to the main trusses are not meant to be load bearing.
- **Load Combinations Checked:** LC2 (Max. Gravity), LC4a (High gravity, winds from the North), LC4d (High gravity, winds from the West).

![Figure 1. Transfer grid map](image-url)
Expansion Load Distribution: Axial Load @ Truss Joints; LC2 Single-Story, Wall-Bearing

Coordinates:

\[
\begin{bmatrix}
0 \\
7.75 \\
15.5 \\
23.25 \\
31 \\
38.75 \\
46.5 \\
54.25 \\
62 \\
69.75 \\
77.5
\end{bmatrix}
\]

\[
x := 
\begin{bmatrix}
0 \\
19.5 \\
39 \\
58.5
\end{bmatrix}
\]

Lines 1 through 4 Axial Load @ Link

\[
L1 := 
\begin{bmatrix}
14.21 \\
0.825 \\
17.32 \\
3.825 \\
17.087 \\
5.196 \\
17.044 \\
3.869 \\
17.308 \\
0.821 \\
14.21
\end{bmatrix} \text{kip}
\]

\[
L2 := 
\begin{bmatrix}
34.604 \\
3.191 \\
48.31 \\
3.955 \\
35.372 \\
4.133 \\
35.011 \\
4.133 \\
48.339 \\
3.192 \\
34.604
\end{bmatrix} \text{kip}
\]

\[
L3 := 
\begin{bmatrix}
35.168 \\
3.558 \\
48.145 \\
4.09 \\
35.544 \\
4.027 \\
34.612 \\
4.027 \\
48.149 \\
3.642 \\
35.25
\end{bmatrix} \text{kip}
\]

\[
L4 := 
\begin{bmatrix}
15.564 \\
1.125 \\
17.56 \\
2.958 \\
16.442 \\
3.234 \\
15.938 \\
2.958 \\
17.557 \\
1.118 \\
21.986
\end{bmatrix} \text{kip}
\]

Lines A through F Axial load @ Links

\[
A := 
\begin{bmatrix}
14.21 \\
34.604 \\
35.168 \\
15.564
\end{bmatrix} \text{kip}
\]

\[
A.5 := 
\begin{bmatrix}
0.825 \\
3.191 \\
3.558 \\
1.125
\end{bmatrix} \text{kip}
\]

\[
B := 
\begin{bmatrix}
17.32 \\
48.31 \\
48.145 \\
17.56
\end{bmatrix} \text{kip}
\]

\[
B.5 := 
\begin{bmatrix}
3.825 \\
3.558 \\
4.09 \\
2.958
\end{bmatrix} \text{kip}
\]

\[
C := 
\begin{bmatrix}
17.087 \\
35.372 \\
35.544 \\
16.442
\end{bmatrix} \text{kip}
\]

\[
C.5 := 
\begin{bmatrix}
5.196 \\
5.181 \\
6.524 \\
4.015
\end{bmatrix} \text{kip}
\]

\[
D := 
\begin{bmatrix}
17.044 \\
35.011 \\
34.612 \\
15.938
\end{bmatrix} \text{kip}
\]

\[
D.5 := 
\begin{bmatrix}
3.869 \\
4.133 \\
4.027 \\
3.234
\end{bmatrix} \text{kip}
\]

\[
E := 
\begin{bmatrix}
17.308 \\
48.339 \\
48.149 \\
17.557
\end{bmatrix} \text{kip}
\]

\[
E.5 := 
\begin{bmatrix}
0.821 \\
3.192 \\
3.642 \\
3.118
\end{bmatrix} \text{kip}
\]

\[
F := 
\begin{bmatrix}
14.21 \\
34.604 \\
35.25 \\
21.986
\end{bmatrix} \text{kip}
\]
Lines 1 through 4 Axial Load @ Link Graphs (Figure 2)
Expansion Load Distribution: Axial Load @ Truss Joints; LC2 Single-Story, Wall-Bearing

Lines A through C.5 Axial Load @ Link Graphs (Figures 3)
Expansion Load Distribution: Axial Load @ Truss Joints; LC2 Single-Story, Wall-Bearing

Lines D through F Axial Load @ Link Graphs (Figures 4)
Expansion Load Distribution: Axial Load @ Truss Joints; LC4a Single-Story, Wall-Bearing

Coordinates:

\[
x := \begin{bmatrix} 0 \\ 7.75 \\ 15.5 \\ 23.25 \\ 31 \\ 38.75 \\ 46.5 \\ 54.25 \\ 62 \\ 69.75 \\ 77.5 \end{bmatrix}
\]

\[
y := \begin{bmatrix} 19.5 \\ 39 \\ 58.5 \end{bmatrix}
\]

Lines 1 through 4 Axial Load @ Link

\[
L1 := \begin{bmatrix} 8.659 \\ 0.786 \\ 9.778 \\ 2.704 \\ 9.283 \end{bmatrix} \text{kip}
\]

\[
L2 := \begin{bmatrix} 19.425 \\ 2.26 \\ 24.886 \\ 2.29 \\ 18.25 \end{bmatrix} \text{kip}
\]

\[
L3 := \begin{bmatrix} 19.608 \\ 2.54 \\ 24.792 \\ 2.236 \\ 17.856 \end{bmatrix} \text{kip}
\]

\[
L4 := \begin{bmatrix} 8.837 \\ 1.308 \\ 9.835 \\ 3.105 \\ 17.856 \end{bmatrix} \text{kip}
\]

Lines A through F Axial Load @ Links

\[
A := \begin{bmatrix} 8.659 \\ 19.425 \\ 19.608 \end{bmatrix} \text{kip}
\]

\[
A.5 := \begin{bmatrix} 0.786 \\ 2.26 \\ 2.54 \end{bmatrix} \text{kip}
\]

\[
B := \begin{bmatrix} 9.778 \\ 24.886 \\ 24.792 \end{bmatrix} \text{kip}
\]

\[
B.5 := \begin{bmatrix} 2.704 \\ 2.29 \\ 2.236 \end{bmatrix} \text{kip}
\]

\[
C := \begin{bmatrix} 9.283 \\ 18.25 \end{bmatrix} \text{kip}
\]

\[
C.5 := \begin{bmatrix} 3.552 \\ 17.98 \end{bmatrix} \text{kip}
\]

\[
D := \begin{bmatrix} 9.266 \\ 25.129 \end{bmatrix} \text{kip}
\]

\[
D.5 := \begin{bmatrix} 2.799 \\ 2.318 \end{bmatrix} \text{kip}
\]

\[
E := \begin{bmatrix} 9.842 \\ 19.145 \end{bmatrix} \text{kip}
\]

\[
E.5 := \begin{bmatrix} 0.850 \\ 1.453 \end{bmatrix} \text{kip}
\]

\[
F := \begin{bmatrix} 8.306 \\ 19.37 \end{bmatrix} \text{kip}
\]
Lines 1 through 4 Axial Load @ Link Graphs (Figure 5)
Expansion Load Distribution: Axial Load @ Truss Joints; LC4a Single-Story, Wall-Bearing

Lines A through C.5 Axial Load @ Link Graphs (Figure 6)
Expansion Load Distribution: Axial Load @ Truss Joints; LC4a Single-Story, Wall-Bearing

Lines D through F Axial Load @ Link Graphs (Figure 7)
Expansion Load Distribution: Axial Load @ Truss Joints; LC4d Single-Story, Wall-Bearing

Coordinates:

\[
\begin{bmatrix}
0 \\
7.75 \\
15.5 \\
23.25 \\
31 \\
38.75 \\
46.5 \\
54.25 \\
62 \\
69.75 \\
77.5
\end{bmatrix}
\]

\[
x := \begin{bmatrix}
7.75 \\
15.5 \\
23.25 \\
31 \\
38.75 \\
46.5 \\
54.25 \\
62 \\
69.75 \\
77.5
\end{bmatrix}
\]

\[
y := \begin{bmatrix}
19.5 \\
39 \\
58.5
\end{bmatrix}
\]

Lines 1 through 4 Axial Load @ Link

\[
L1 := \begin{bmatrix}
8.821 \\
0.781 \\
10.083 \\
2.731 \\
9.531 \\
3.594 \\
9.503 \\
2.763 \\
10.075 \\
0.778 \\
8.814
\end{bmatrix}
\]

\[
L2 := \begin{bmatrix}
19.384 \\
2.247 \\
24.925 \\
2.252 \\
18.214 \\
2.945 \\
17.973 \\
2.372 \\
24.95 \\
2.249 \\
19.383
\end{bmatrix}
\]

\[
L3 := \begin{bmatrix}
19.554 \\
2.531 \\
24.89 \\
2.243 \\
17.863 \\
3.531 \\
17.898 \\
2.201 \\
24.893 \\
2.243 \\
19.608
\end{bmatrix}
\]

\[
L4 := \begin{bmatrix}
8.54 \\
1.423 \\
9.517 \\
3.087 \\
8.725
\end{bmatrix}
\]

Lines A through F Axial load @ Links

\[
A := \begin{bmatrix}
8.821 \\
19.384 \\
19.554 \\
8.54
\end{bmatrix}
\]

\[
A.5 := \begin{bmatrix}
0.781 \\
2.247 \\
2.531 \\
1.423
\end{bmatrix}
\]

\[
B := \begin{bmatrix}
10.083 \\
24.925 \\
24.89 \\
9.517
\end{bmatrix}
\]

\[
B.5 := \begin{bmatrix}
2.731 \\
2.531 \\
2.201 \\
2.267
\end{bmatrix}
\]

\[
C := \begin{bmatrix}
9.531 \\
18.214 \\
17.863 \\
8.725
\end{bmatrix}
\]

\[
C.5 := \begin{bmatrix}
3.594 \\
2.945 \\
3.531 \\
3.087
\end{bmatrix}
\]

\[
D := \begin{bmatrix}
9.503 \\
17.973 \\
17.898 \\
8.450
\end{bmatrix}
\]

\[
D.5 := \begin{bmatrix}
2.763 \\
17.973 \\
17.898 \\
2.418
\end{bmatrix}
\]

\[
E := \begin{bmatrix}
10.075 \\
24.95 \\
24.893 \\
9.516
\end{bmatrix}
\]

\[
E.5 := \begin{bmatrix}
0.778 \\
2.249 \\
2.519 \\
1.416
\end{bmatrix}
\]

\[
F := \begin{bmatrix}
8.814 \\
19.383 \\
19.608 \\
10.532
\end{bmatrix}
\]
Expansion Load Distribution: Axial Load @ Truss Joints; LC4d Single-Story, Wall-Bearing

Lines 1 through 4 Axial Load @ Link Graphs (Figure 8)
Expansion Load Distribution: Axial Load @ Truss Joints; LC4d Single-Story, Wall-Bearing

Lines A through C.5 Axial Load @ Link Graphs (Figure 9)
Expansion Load Distribution: Axial Load @ Truss Joints; LC4d Single-Story, Wall-Bearing

**Lines D through F Axial Load @ Link Graphs** (Figure 10)
Expansion Load Distribution: Axial Load @ Truss Joints; LC2 Two-Story, Wall-Bearing

Coordinates:

\[
\begin{bmatrix}
0 \\
7.75 \\
15.5 \\
23.25 \\
31 \\
38.75 \\
46.5 \\
54.25 \\
62 \\
69.75 \\
77.5
\end{bmatrix}
\]

\[
x := \begin{bmatrix} 0 \\ 19.5 \\ 39 \\ 58.5 \end{bmatrix} \text{ ft}
\]

Lines 1 through 4 Axial Load @ Link

\[
\begin{bmatrix}
26.323 \\
2.296 \\
32.584 \\
7.143 \\
32.676 \\
9.343 \\
32.615 \\
2.29 \\
26.317
\end{bmatrix}
\]

\[
y := \begin{bmatrix} 5.345 \\ 4.608 \\ 78.742 \\ 6.401 \\ 60.945 \\ 8.383 \\ 60.411 \\ 6.669 \\ 78.761 \end{bmatrix} \text{ ft}
\]

\[
L1 := \begin{bmatrix}
9.343 \\
32.615 \\
7.21 \\
32.566 \\
2.29 \\
26.317
\end{bmatrix} \text{ kip}
\]

\[
L2 := \begin{bmatrix}
8.383 \\
60.411 \\
6.669 \\
78.761 \\
4.607 \\
51.347
\end{bmatrix} \text{ kip}
\]

\[
L3 := \begin{bmatrix}
9.938 \\
60.211 \\
6.364 \\
78.794 \\
5.274 \\
52.076
\end{bmatrix} \text{ kip}
\]

\[
L4 := \begin{bmatrix}
9.343 \\
32.615 \\
7.21 \\
32.566 \\
2.29 \\
26.317
\end{bmatrix} \text{ kip}
\]

Lines A through F Axial load @ Links

\[
A := \begin{bmatrix}
L1_0 \\
L2_0 \\
L3_0 \\
L4_0
\end{bmatrix}
\]

\[
A.5 := \begin{bmatrix}
L1_1 \\
L2_1 \\
L3_1 \\
L4_1
\end{bmatrix}
\]

\[
B := \begin{bmatrix}
L1_2 \\
L2_2 \\
L3_2 \\
L4_2
\end{bmatrix}
\]

\[
B.5 := \begin{bmatrix}
L1_3 \\
L2_3 \\
L3_3 \\
L4_3
\end{bmatrix}
\]

\[
C := \begin{bmatrix}
L1_4 \\
L2_4 \\
L3_4 \\
L4_4
\end{bmatrix}
\]

\[
C.5 := \begin{bmatrix}
L1_5 \\
L2_5 \\
L3_5 \\
L4_5
\end{bmatrix}
\]

\[
D := \begin{bmatrix}
L1_6 \\
L2_6 \\
L3_6 \\
L4_6
\end{bmatrix}
\]

\[
D.5 := \begin{bmatrix}
L1_7 \\
L2_7 \\
L3_7 \\
L4_7
\end{bmatrix}
\]

\[
E := \begin{bmatrix}
L1_8 \\
L2_8 \\
L3_8 \\
L4_8
\end{bmatrix}
\]

\[
E.5 := \begin{bmatrix}
L1_9 \\
L2_9 \\
L3_9 \\
L4_9
\end{bmatrix}
\]

\[
F := \begin{bmatrix}
L1_{10} \\
L2_{10} \\
L3_{10} \\
L4_{10}
\end{bmatrix}
\]
Expansion Load Distribution: Axial Load @ Truss Joints; LC2 Two-Story, Wall-Bearing

Lines 1 through 4 Axial Load @ Link Graphs  (Figure 11)
Expansion Load Distribution: Axial Load @ Truss Joints; LC2 Two-Story, Wall-Bearing

Lines A through C.5 Axial Load @ Link Graphs (Figure 12)
Expansion Load Distribution: Axial Load @ Truss Joints; LC2 Two-Story, Wall-Bearing

Lines D through F Axial Load @ Link Graphs (Figure 13)
Expansion Load Distribution: Axial Load @ Truss Joints; LC4a Two-Story, Wall-Bearing

Coordinates:

\[
\begin{bmatrix}
0 \\
7.75 \\
15.5 \\
23.25 \\
31 \\
38.75 \\
46.5 \\
54.25 \\
62 \\
69.75 \\
77.5
\end{bmatrix} \quad ft
\]

\[
\begin{bmatrix}
y_0 \\
y_1 \\
y_2 \\
y_3 \\
y_4 \\
y_5 \\
y_6 \\
y_7 \\
y_8 \\
y_9 \\
y_{10}
\end{bmatrix} \quad ft
\]

Lines 1 through 4 Axial Load @ Link

\[
\begin{bmatrix}
17.392 \\
2.027 \\
19.36 \\
5.023 \\
18.967 \\
6.357 \\
18.972 \\
5.182 \\
19.513 \\
2.106 \\
14.734
\end{bmatrix} \quad \text{kip}
\]

\[
\begin{bmatrix}
30.101 \\
3.158 \\
43.001 \\
3.714 \\
33.66 \\
4.896 \\
33.20 \\
4.01 \\
44.877 \\
3.854 \\
27.833
\end{bmatrix} \quad \text{kip}
\]

\[
\begin{bmatrix}
30.285 \\
3.569 \\
42.974 \\
3.663 \\
33.218 \\
5.644 \\
33.258 \\
3.717 \\
44.848 \\
3.763 \\
28.091
\end{bmatrix} \quad \text{kip}
\]

\[
\begin{bmatrix}
17.967 \\
2.889 \\
19.316 \\
4.476 \\
18.336 \\
5.682 \\
18.007 \\
4.82 \\
19.498 \\
3.065 \\
17.16
\end{bmatrix} \quad \text{kip}
\]

Lines A through F Axial Load @ Links

\[
\begin{bmatrix}
L_{11} \\
L_{12} \\
L_{13} \\
L_{14} \\
L_{15}
\end{bmatrix}
\]

\[
\begin{bmatrix}
L_{21} \\
L_{22} \\
L_{23} \\
L_{24} \\
L_{25}
\end{bmatrix} \quad \text{kip}
\]

\[
\begin{bmatrix}
L_{31} \\
L_{32} \\
L_{33} \\
L_{34} \\
L_{35}
\end{bmatrix} \quad \text{kip}
\]

\[
\begin{bmatrix}
L_{41} \\
L_{42} \\
L_{43} \\
L_{44} \\
L_{45}
\end{bmatrix} \quad \text{kip}
\]
Expansion Load Distribution: Axial Load @ Truss Joints; LC4a Two-Story, Wall-Bearing

Lines 1 through 4 Axial Load @ Link Graphs (Figure 14)
Expansion Load Distribution: Axial Load @ Truss Joints; LC4a Two-Story, Wall-Bearing

Lines A through C.5 Axial Load @ Link Graphs (Figure 15)
Expansion Load Distribution: Axial Load @ Truss Joints; LC4a Two-Story, Wall-Bearing

Lines D through F Axial Load @ Link Graphs (Figure 16)
Expansion Load Distribution: Axial Load @ Truss Joints; LC4b Two-Story, Wall-Bearing

Coordinates:

\[
\begin{bmatrix}
0 \\
7.75 \\
15.5 \\
23.25 \\
31 \\
38.75 \\
46.5 \\
54.25 \\
62 \\
69.75 \\
77.5
\end{bmatrix}
\]

\[
x := 
\begin{bmatrix}
0 \\
19.5 \\
39 \\
58.5
\end{bmatrix}
\]

Lines 1 through 4 Axial Load @ Link

\[
L1 := \begin{bmatrix} 17.341 \\ 2.043 \\ 20.576 \\ 5.111 \\ 19.954 \\ 6.450 \\ 19.914 \\ 5.158 \\ 20.564 \\ 2.039 \\ 17.335 \end{bmatrix} \text{kip}
\]

\[
L2 := \begin{bmatrix} 29.859 \\ 3.090 \\ 43.240 \\ 3.744 \\ 33.569 \\ 4.937 \\ 33.223 \\ 3.920 \\ 43.252 \\ 3.089 \\ 29.858 \end{bmatrix} \text{kip}
\]

\[
L3 := \begin{bmatrix} 33.569 \\ 3.744 \\ 43.308 \\ 3.671 \\ 33.305 \\ 5.606 \\ 33.223 \\ 3.920 \\ 43.252 \\ 3.089 \\ 30.070 \end{bmatrix} \text{kip}
\]

\[
L4 := \begin{bmatrix} 16.08 \\ 2.953 \\ 18.044 \\ 4.253 \\ 17.189 \\ 5.467 \\ 16.839 \\ 4.452 \\ 18.034 \\ 2.947 \\ 18.04 \end{bmatrix} \text{kip}
\]

Lines A through F Axial load @ Links

\[
A := \begin{bmatrix} L1_0 \\ L2_0 \\ L3_0 \\ L4_0 \end{bmatrix} \text{kip}
\]

\[
A.5 := \begin{bmatrix} L1_1 \\ L2_1 \\ L3_1 \\ L4_1 \end{bmatrix} \text{kip}
\]

\[
B := \begin{bmatrix} L1_2 \\ L2_2 \\ L3_2 \\ L4_2 \end{bmatrix} \text{kip}
\]

\[
B.5 := \begin{bmatrix} L1_3 \\ L2_3 \\ L3_3 \\ L4_3 \end{bmatrix} \text{kip}
\]

\[
C := \begin{bmatrix} L1_4 \\ L2_4 \\ L3_4 \\ L4_4 \end{bmatrix} \text{kip}
\]

\[
C.5 := \begin{bmatrix} L1_5 \\ L2_5 \\ L3_5 \\ L4_5 \end{bmatrix} \text{kip}
\]
Expansion Load Distribution: Axial Load @ Truss Joints; LC4b Two-Story, Wall-Bearing

Lines 1 through 4 Axial Load @ Link Graphs (Figure 17)
Expansion Load Distribution: Axial Load @ Truss Joints; LC4b Two-Story, Wall-Bearing

Lines A through C.5 Axial Load @ Link Graphs (Figure 18)
Expansion Load Distribution: Axial Load @ Truss Joints; LC4b Two-Story, Wall-Bearing

Lines D through F Axial Load @ Link Graphs (Figure 19)
Expansion Load Distribution: Axial Load @ Truss Joints; LC2 Single-Story, Corner-Bearing

Coordinates:

\[
\begin{bmatrix}
0 \\
7.75 \\
15.5 \\
23.25 \\
31 \\
38.75 \\
46.5 \\
54.25 \\
62 \\
69.75 \\
77.5
\end{bmatrix} = \begin{bmatrix}
0 \\
19.5 \\
39 \\
58.5
\end{bmatrix}
\]

Lines 1 through 4 Axial Load @ Link

\[
\begin{bmatrix}
14.442 \\
1.495 \\
23.125 \\
3.818 \\
19.482 \\
2.696 \\
20.574 \\
2.172 \\
22.805 \\
1.936 \\
14.655
\end{bmatrix} = \begin{bmatrix}
37.101 \\
5.773 \\
49.958 \\
3.419 \\
40.736 \\
3.812 \\
39.636 \\
3.438 \\
49.631 \\
5.845 \\
36.666
\end{bmatrix} = \begin{bmatrix}
36.666 \\
6.727 \\
49.325 \\
3.214 \\
39.125 \\
4.692 \\
39.283 \\
3.179 \\
49.324 \\
6.675 \\
17.433
\end{bmatrix} = \begin{bmatrix}
17.433 \\
2.986 \\
22.441 \\
1.63 \\
18.259 \\
1.841 \\
17.861 \\
1.71 \\
22.427 \\
3.004 \\
23.8
\end{bmatrix}
\]

Lines A through F Axial load @ Links

\[
\begin{bmatrix}
L1_0 \\
L2_0 \\
L3_0 \\
L4_0
\end{bmatrix} = \begin{bmatrix}
L1_1 \\
L2_1 \\
L3_1 \\
L4_1
\end{bmatrix} = \begin{bmatrix}
L1_2 \\
L2_2 \\
L3_2 \\
L4_2
\end{bmatrix} = \begin{bmatrix}
L1_3 \\
L2_3 \\
L3_3 \\
L4_3
\end{bmatrix} = \begin{bmatrix}
L1_4 \\
L2_4 \\
L3_4 \\
L4_4
\end{bmatrix} = \begin{bmatrix}
L1_5 \\
L2_5 \\
L3_5 \\
L4_5
\end{bmatrix}
\]

\[
A := \begin{bmatrix}
L1_0 \\
L2_0 \\
L3_0 \\
L4_0
\end{bmatrix}; \quad A.5 := \begin{bmatrix}
L1_1 \\
L2_1 \\
L3_1 \\
L4_1
\end{bmatrix}; \quad B := \begin{bmatrix}
L1_2 \\
L2_2 \\
L3_2 \\
L4_2
\end{bmatrix}; \quad B.5 := \begin{bmatrix}
L1_3 \\
L2_3 \\
L3_3 \\
L4_3
\end{bmatrix}; \quad C := \begin{bmatrix}
L1_4 \\
L2_4 \\
L3_4 \\
L4_4
\end{bmatrix}; \quad C.5 := \begin{bmatrix}
L1_5 \\
L2_5 \\
L3_5 \\
L4_5
\end{bmatrix}
\]

\[
D := \begin{bmatrix}
L1_6 \\
L2_6 \\
L3_6 \\
L4_6
\end{bmatrix}; \quad D.5 := \begin{bmatrix}
L1_7 \\
L2_7 \\
L3_7 \\
L4_7
\end{bmatrix}; \quad E := \begin{bmatrix}
L1_8 \\
L2_8 \\
L3_8 \\
L4_8
\end{bmatrix}; \quad E.5 := \begin{bmatrix}
L1_9 \\
L2_9 \\
L3_9 \\
L4_9
\end{bmatrix}; \quad F := \begin{bmatrix}
L1_{10} \\
L2_{10} \\
L3_{10} \\
L4_{10}
\end{bmatrix}
\]
Expansion Load Distribution: Axial Load @ Truss Joints; LC2 Single-Story, Corner-Bearing

Lines 1 through 4 Axial Load @ Link Graphs (Figure 20)
Lines A through C.5 Axial Load @ Link Graphs (Figure 21)
Expansion Load Distribution: Axial Load @ Truss Joints; LC2 Single-Story, Corner-Bearing

Lines D through F Axial Load @ Link Graphs (Figure 22)
Expansion Load Distribution: Axial Load @ Truss Joints; LC4a Single-Story, Corner-Bearing

Coordinates:

\[
\begin{bmatrix}
0 \\
7.75 \\
15.5 \\
23.25 \\
31 \\
38.75 \\
46.5 \\
54.25 \\
62 \\
69.75 \\
77.5 \\
\end{bmatrix}
\]

\[
\begin{bmatrix}
0 \\
19.5 \\
39 \\
58.5 \\
\end{bmatrix}
\]

Lines 1 through 4 Axial Load @ Link

\[
\begin{bmatrix}
11.430 \\
1.203 \\
17.658 \\
3.144 \\
14.598 \\
2.10 \\
15.440 \\
1.773 \\
17.557 \\
1.571 \\
11.015 \\
\end{bmatrix}
\]

\[
\begin{bmatrix}
25.704 \\
4.019 \\
33.556 \\
2.538 \\
29.292 \\
2.71 \\
28.459 \\
2.66 \\
34.486 \\
4.216 \\
24.141 \\
\end{bmatrix}
\]

\[
\begin{bmatrix}
25.257 \\
4.603 \\
33.409 \\
2.333 \\
28.272 \\
5.095 \\
28.341 \\
2.414 \\
34.316 \\
4.718 \\
24.469 \\
\end{bmatrix}
\]

\[
\begin{bmatrix}
12.462 \\
2.445 \\
17.123 \\
1.771 \\
14.183 \\
2.268 \\
13.934 \\
2.541 \\
17.279 \\
2.541 \\
13.728 \\
\end{bmatrix}
\]

Lines A through F Axial load @ Links

\[
\begin{bmatrix}
L1_0 \\
L2_0 \\
L3_0 \\
L4_0 \\
\end{bmatrix}
\]

\[
\begin{bmatrix}
L1_1 \\
L2_1 \\
L3_1 \\
L4_1 \\
\end{bmatrix}
\]

\[
\begin{bmatrix}
L1_2 \\
L2_2 \\
L3_2 \\
L4_2 \\
\end{bmatrix}
\]

\[
\begin{bmatrix}
L1_3 \\
L2_3 \\
L3_3 \\
L4_3 \\
\end{bmatrix}
\]

\[
\begin{bmatrix}
L1_4 \\
L2_4 \\
L3_4 \\
L4_4 \\
\end{bmatrix}
\]

\[
\begin{bmatrix}
L1_5 \\
L2_5 \\
L3_5 \\
L4_5 \\
\end{bmatrix}
\]

\[
\begin{bmatrix}
L1_6 \\
L2_6 \\
L3_6 \\
L4_6 \\
\end{bmatrix}
\]

\[
\begin{bmatrix}
L1_7 \\
L2_7 \\
L3_7 \\
L4_7 \\
\end{bmatrix}
\]

\[
\begin{bmatrix}
L1_8 \\
L2_8 \\
L3_8 \\
L4_8 \\
\end{bmatrix}
\]

\[
\begin{bmatrix}
L1_9 \\
L2_9 \\
L3_9 \\
L4_9 \\
\end{bmatrix}
\]

\[
\begin{bmatrix}
L1_{10} \\
L2_{10} \\
L3_{10} \\
L4_{10} \\
\end{bmatrix}
\]
Expansion Load Distribution: Axial Load @ Truss Joints; LC4a Single-Story, Corner-Bearing

Lines 1 through 4 Axial Load @ Link Graphs (Figure 23)
Expansion Load Distribution: Axial Load @ Truss Joints; LC4a Single-Story, Corner-Bearing

Lines A through C 5 Axial Load @ Link Graphs (Figure 24)
Expansion Load Distribution: Axial Load @ Truss Joints; LC4a Single-Story, Corner-Bearing

Lines D through F Axial Load @ Link Graphs (Figure 25)
Expansion Load Distribution: Axial Load @ Truss Joints; LC4a Single-Story, Corner-Bearing

Coordinates:

\[
x := \begin{bmatrix} 0 \\ 7.75 \\ 15.5 \\ 23.25 \\ 31 \\ 38.75 \\ 46.5 \\ 54.25 \\ 62 \\ 69.75 \\ 77.5 \end{bmatrix} \quad y := \begin{bmatrix} 0 \\ 19.5 \\ 39 \\ 58.5 \end{bmatrix}
\]

Lines 1 through 4 Axial Load @ Link

\[
L1 := \begin{bmatrix} 11.654 \\ 1.192 \\ 17.943 \\ 3.134 \\ 14.831 \\ 2.091 \\ 15.645 \\ 1.713 \\ 17.719 \\ 1.564 \\ 11.829 \end{bmatrix} \quad \text{kip} \quad L2 := \begin{bmatrix} 25.522 \\ 4.018 \\ 33.695 \\ 2.528 \\ 29.271 \\ 2.687 \\ 28.447 \\ 2.531 \\ 33.715 \\ 4.076 \\ 25.28 \end{bmatrix} \quad \text{kip} \quad L3 := \begin{bmatrix} 25.071 \\ 4.607 \\ 33.587 \\ 2.33 \\ 28.285 \\ 3.067 \\ 28.381 \\ 2.297 \\ 33.585 \\ 4.564 \\ 25.575 \end{bmatrix} \quad \text{kip} \quad L4 := \begin{bmatrix} 12.029 \\ 2.441 \\ 16.795 \\ 1.76 \\ 13.822 \\ 2.246 \\ 13.822 \\ 1.813 \\ 16.79 \\ 2.445 \end{bmatrix} \quad \text{kip}
\]

Lines A through F Axial load @ Links

\[
A := \begin{bmatrix} L1 \\ L2 \\ L3 \\ L4 \end{bmatrix} \quad A.5 := \begin{bmatrix} L1 \\ L2 \\ L3 \\ L4 \end{bmatrix} \quad B := \begin{bmatrix} L1 \\ L2 \\ L3 \\ L4 \end{bmatrix} \quad B.5 := \begin{bmatrix} L1 \\ L2 \\ L3 \\ L4 \end{bmatrix} \quad C := \begin{bmatrix} L1 \\ L2 \\ L3 \\ L4 \end{bmatrix} \quad C.5 := \begin{bmatrix} L1 \\ L2 \\ L3 \\ L4 \end{bmatrix}
\]

\[
D := \begin{bmatrix} L1 \\ L2 \\ L3 \\ L4 \end{bmatrix} \quad D.5 := \begin{bmatrix} L1 \\ L2 \\ L3 \\ L4 \end{bmatrix} \quad E := \begin{bmatrix} L1 \\ L2 \\ L3 \\ L4 \end{bmatrix} \quad E.5 := \begin{bmatrix} L1 \\ L2 \\ L3 \\ L4 \end{bmatrix} \quad F := \begin{bmatrix} L1 \\ L2 \\ L3 \\ L4 \end{bmatrix}
\]
Expansion Load Distribution: Axial Load @ Truss Joints; LC4a Single-Story, Corner-Bearing

Lines 1 through 4 Axial Load @ Link Graphs  (Figure 26)
Expansion Load Distribution: Axial Load @ Truss Joints; LC4a Single-Story, Corner-Bearing

Lines A through C.5 Axial Load @ Link Graphs (Figure 27)
Lines D through F Axial Load @ Link Graphs  (Figure 28)
Expansion Load Distribution: Axial Load @ Truss Joints; LC2 Two-Story, Corner-Bearing

Coordinates:

\[
x := \begin{bmatrix}
0 \\
7.75 \\
15.5 \\
23.25 \\
31 \\
38.75 \\
46.5 \\
54.25 \\
62 \\
69.75 \\
77.5 \\
\end{bmatrix}
y := \begin{bmatrix}
0 \\
19.5 \\
39 \\
58.5 \\
\end{bmatrix}
\]

Lines 1 through 4 Axial Load @ Link

\[
L1 := \begin{bmatrix}
28.064 \\
2.873 \\
43.289 \\
6.221 \\
36.678 \\
4.78 \\
37.932 \\
3.756 \\
43.042 \\
3.524 \\
28.456 \\
\end{bmatrix}
\]

\[
L2 := \begin{bmatrix}
72.559 \\
9.542 \\
88.128 \\
5.89 \\
73.174 \\
6.755 \\
71.959 \\
5.904 \\
88.711 \\
5.904 \\
72.0 \\
\end{bmatrix}
\]

\[
L3 := \begin{bmatrix}
71.899 \\
10.778 \\
87.704 \\
5.51 \\
71.706 \\
7.705 \\
71.844 \\
5.439 \\
88.305 \\
10.439 \\
73.326 \\
\end{bmatrix}
\]

\[
L4 := \begin{bmatrix}
32.743 \\
5.226 \\
41.905 \\
3.375 \\
33.823 \\
4.318 \\
33.371 \\
3.507 \\
42.186 \\
5.245 \\
38.916 \\
\end{bmatrix}
\]

Lines A through F Axial load @ Links

\[
A := \begin{bmatrix}
L1_0 \\
L2_0 \\
L3_0 \\
L4_0 \\
\end{bmatrix}
A.5 := \begin{bmatrix}
L1_1 \\
L2_1 \\
L3_1 \\
L4_1 \\
\end{bmatrix}
B := \begin{bmatrix}
L1_2 \\
L2_2 \\
L3_2 \\
L4_2 \\
\end{bmatrix}
B.5 := \begin{bmatrix}
L1_3 \\
L2_3 \\
L3_3 \\
L4_3 \\
\end{bmatrix}
C := \begin{bmatrix}
L1_4 \\
L2_4 \\
L3_4 \\
L4_4 \\
\end{bmatrix}
C.5 := \begin{bmatrix}
L1_5 \\
L2_5 \\
L3_5 \\
L4_5 \\
\end{bmatrix}
\]

\[
D := \begin{bmatrix}
L1_6 \\
L2_6 \\
L3_6 \\
L4_6 \\
\end{bmatrix}
D.5 := \begin{bmatrix}
L1_7 \\
L2_7 \\
L3_7 \\
L4_7 \\
\end{bmatrix}
E := \begin{bmatrix}
L1_8 \\
L2_8 \\
L3_8 \\
L4_8 \\
\end{bmatrix}
E.5 := \begin{bmatrix}
L1_9 \\
L2_9 \\
L3_9 \\
L4_9 \\
\end{bmatrix}
F := \begin{bmatrix}
L1_{10} \\
L2_{10} \\
L3_{10} \\
L4_{10} \\
\end{bmatrix}
\]
Expansion Load Distribution: Axial Load @ Truss Joints; LC2 Two-Story, Corner-Bearing

Lines 1 through 4 Axial Load @ Link Graphs (Figure 29)
Expansion Load Distribution: Axial Load @ Truss Joints; LC2 Two-Story, Corner-Bearing

Lines A through C.5 Axial Load @ Link Graphs (Figure 30)
Expansion Load Distribution: Axial Load @ Truss Joints; LC2 Two-Story, Corner-Bearing

Lines D through F Axial Load @ Link Graphs  (Figure 31)

- **D (kip)**
- **D.5 (kip)**
- **E (kip)**
- **E.5 (kip)**
- **F (kip)**
Expansion Load Distribution: Axial Load @ Truss Joints; LC4a Two-Story, Corner-Bearing

Coordinates:

\[
x := \begin{bmatrix} 0 \\ 7.75 \\ 15.5 \\ 23.25 \\ 31 \\ 38.75 \\ 46.5 \\ 54.25 \\ 62 \\ 69.75 \\ 77.5 \end{bmatrix} \quad \text{ft} \\
y := \begin{bmatrix} 0 \\ 19.5 \\ 39 \\ 58.5 \end{bmatrix} \quad \text{ft}
\]

Lines 1 through 4 Axial Load @ Link

\[
L2 := \begin{bmatrix} 4.677 & 5.121 & 5.121 & 4.677 \\ 50.291 & 50.242 & 50.242 & 50.291 \\ 3.939 & 3.939 & 3.939 & 3.939 \\ 7.103 & 7.103 & 7.103 & 7.103 \end{bmatrix} \quad \text{kip} \\
L3 := \begin{bmatrix} 5.121 & 5.121 & 5.121 & 5.121 \\ 50.242 & 50.242 & 50.242 & 50.242 \\ 3.939 & 3.939 & 3.939 & 3.939 \\ 7.791 & 7.791 & 7.791 & 7.791 \end{bmatrix} \quad \text{kip} \\
\]

Lines A through F Axial Load @ Links

\[
A := \begin{bmatrix} L1_0 \\ L2_0 \\ L3_0 \\ L4_0 \end{bmatrix} \quad A.5 := \begin{bmatrix} L1_1 \\ L2_1 \\ L3_1 \\ L4_1 \end{bmatrix} \quad B := \begin{bmatrix} L1_2 \\ L2_2 \\ L3_2 \\ L4_2 \end{bmatrix} \quad B.5 := \begin{bmatrix} L1_3 \\ L2_3 \\ L3_3 \\ L4_3 \end{bmatrix} \quad C := \begin{bmatrix} L1_4 \\ L2_4 \\ L3_4 \\ L4_4 \end{bmatrix} \quad C.5 := \begin{bmatrix} L1_5 \\ L2_5 \\ L3_5 \\ L4_5 \end{bmatrix}
\]

\[
D := \begin{bmatrix} L1_6 \\ L2_6 \\ L3_6 \\ L4_6 \end{bmatrix} \quad D.5 := \begin{bmatrix} L1_7 \\ L2_7 \\ L3_7 \\ L4_7 \end{bmatrix} \quad E := \begin{bmatrix} L1_8 \\ L2_8 \\ L3_8 \\ L4_8 \end{bmatrix} \quad E.5 := \begin{bmatrix} L1_9 \\ L2_9 \\ L3_9 \\ L4_9 \end{bmatrix} \quad F := \begin{bmatrix} L1_10 \\ L2_10 \\ L3_10 \\ L4_10 \end{bmatrix}
\]
Expansion Load Distribution: Axial Load @ Truss Joints; LC4a Two-Story, Corner-Bearing

Lines 1 through 4 Axial Load @ Link Graphs (Figure 32)
Expansion Load Distribution: Axial Load @ Truss Joints; LC4a Two-Story, Corner-Bearing

Lines A through C.5 Axial Load @ Link Graphs (Figure 33)
Expansion Load Distribution: Axial Load @ Truss Joints; LC4a Two-Story, Corner-Bearing

Lines D through F Axial Load @ Link Graphs (Figure 34)
Coordinates:

\[
\begin{bmatrix}
0 \\
7.75 \\
15.5 \\
23.25 \\
31 \\
38.75 \\
46.5 \\
54.25 \\
62 \\
69.75 \\
77.5
\end{bmatrix} \quad x := \begin{bmatrix}
0 \\
19.5 \\
39 \\
58.5
\end{bmatrix} \quad ft
\]

Lines 1 through 4 Axial Load @ Link

\[
\begin{bmatrix}
22.441 & 42.905 & 47.229 & 22.023 \\
2.225 & 6.428 & 7.210 & 3.909 \\
33.212 & 57.642 & 57.711 & 59.712 \\
4.756 & 4.112 & 3.824 & 2.935 \\
27.732 & 51.083 & 50.387 & 24.268
\end{bmatrix} \quad L1 := 3.558 \text{ kip}
\]

\[
\begin{bmatrix}
4.659 & 5.079 & 5.079 & 3.885 \text{ kip} \\
28.654 & 50.194 & 50.477 & 23.979 \\
2.824 & 4.116 & 3.771 & 3.028 \\
33.103 & 58.237 & 58.323 & 29.999 \\
2.738 & 6.507 & 7.172 & 3.916 \\
22.747 & 47.687 & 48.077 & 23.882
\end{bmatrix} \quad L2 := kip
\]

Lines A through F Axial load @ Links

\[
\begin{bmatrix}
L1_0 \\
L2_0 \\
L3_0 \\
L4_0
\end{bmatrix} \quad A := \begin{bmatrix}
L1_1 \\
L2_1 \\
L3_1 \\
L4_1
\end{bmatrix} \quad B := \begin{bmatrix}
L1_2 \\
L2_2 \\
L3_2 \\
L4_2
\end{bmatrix} \quad B.5 := \begin{bmatrix}
L1_3 \\
L2_3 \\
L3_3 \\
L4_3
\end{bmatrix} \quad C := \begin{bmatrix}
L1_4 \\
L2_4 \\
L3_4 \\
L4_4
\end{bmatrix} \quad C.5 := \begin{bmatrix}
L1_5 \\
L2_5 \\
L3_5 \\
L4_5
\end{bmatrix}
\]

D := \begin{bmatrix}
L1_6 \\
L2_6 \\
L3_6 \\
L4_6
\end{bmatrix} \quad E := \begin{bmatrix}
L1_7 \\
L2_7 \\
L3_7 \\
L4_7
\end{bmatrix} \quad E.5 := \begin{bmatrix}
L1_8 \\
L2_8 \\
L3_8 \\
L4_8
\end{bmatrix} \quad F := \begin{bmatrix}
L1_9 \\
L2_9 \\
L3_9 \\
L4_9
\end{bmatrix} \quad F.5 := \begin{bmatrix}
L1_{10} \\
L2_{10} \\
L3_{10} \\
L4_{10}
\end{bmatrix}
Expansion Load Distribution: Axial Load @ Truss Joints; LC4d Two-Story, Corner-Bearing

Lines 1 through 4 Axial Load @ Link Graphs  (Figure 35)
Expansion Load Distribution: Axial Load @ Truss Joints; LC4d Two-Story, Corner-Bearing

Lines A through C.5 Axial Load @ Link Graphs (Figure 36)
Expansion Load Distribution: Axial Load @ Truss Joints; LC4d Two-Story, Corner-Bearing

Lines D through F Axial Load @ Link Graphs (Figure 37)