Senior Thesis Final Report

West Village Housing Phases III & IV

Towson, Maryland

Mark Bland [Structural Option]

Advisor: Dr. Aly Said

April 8th, 2016

West Village Housing Phases III & IV

Located in: Towson, MD

Project Team

Owner	Towson University
General Contractor and CM	Whiting Turner Contracting
Architect & Landscape Architect	Ayers/Saint/Gross
Civil Engineer	Site Resources
Structural Engineer	Hope Furrer Associates
MEP/FP Engineers	Newcomb & Boyd
Electrical/Plumbing	WFT Engineering

Architecture

These two high-rise apartment buildings will add approximately 700 student beds to Towson University's campus. Floor arrangements included a mix of two and four bedroom apartments with shared bathrooms, kitchen and living areas create an adult living environment for the students. The exterior façades are a mix of brick and steel plate veneers adding personality to the area.

Construction

The buildings began construction simultaneously in September of 2014 to address the continued demand for on-campus housing and are planned to be finished in the summer of 2016.

Sustainability

Similar to most buildings on Towson University's campus, these two new facilities will be built in adherence with sustainable design and are expected to achieve LEED Silver certification.

Central lawn adjacent to building
(courtesy of Towson University Construction Services)

General Information

Building occupant name	Towson University Students
Occupancy	Residence Hall
Size	170,000 sq. ft.
Full Height	92 ft.
Total number of stories	9
Date of construction	September 2014 - July 2016
Cost information	Figures are being withheld
Delivery method	CM at risk

Birds-eye view of project site
(courtesy of Towson University Construction Services)

Electrical/Lighting Systems

Generally, interior lighting consists of fluorescent type T5. Downlights and decorative fixtures will be LED. Normal power for the building will originate from a pad-mount service transformer located outside each building.

Mechanical Systems

The heating is provided by a hot water system that consists of two boilers, hot water pumps, and piping. The hot water plant will be located in the penthouse on the roof. Chilled water will be distributed to coils in individual fan coil units via vertical distribution for each suite. Energy Recovery Units (ERU) will be single-zone medium pressure type to provide air conditioning. In addition, all units have occupant operable windows.

Structural Systems

Framing ---------The structural system will be 8" thick two-way post-tensioned concrete flat plats supported by reinforced concrete columns.

Foundations------In order to utilize conventional spread footings, Rammed Aggregate Piers (RAPs) will be used.

Lateral System---12" thick concrete shear walls will effectively resist the forces imposed on the building from wind and seismic loading.

For more information, visit my CPEP site
http://mab6037.wix.com/thesis

Mark Bland [Structural Option]
Advisor: Dr. Aly Said

Acknowledgements

I would like to thank the following people for all their help and support this past year:

- My friends and family, especially my Mom and Dad, for always being there for me through my college career and senior thesis. Without their love and support, I would not be the person I am today.

- The engineers and mentors at Hope Furrer Associates who encouraged me to choose the West Village Housing project for my senior thesis. Thanks especially to Tom Barabas who provided me with all the knowledge and plans that I needed to make this report a success.

- Mr. R Timothy Sandruck of Towson Universities Construction Services for his generosity to show me the project site and provide me with any information that I requested.

- My thesis advisor, Dr. Aly Said, who was always willing to meet with me and answer questions at any time of day. Thanks for the "real world" advice as I begin my professional career.

- The AE Faculty, especially Professor Kevin Parfitt, who has helped and guided me to where I am today. His honesty and genuine care for the AE students is something to be treasured.

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To all the many people who have influenced me throughout my life and time at college, I am eternally grateful. Thank you to all.

Executive Summary

This is the final report in a yearlong research study of the structural system for Towson University's West Village Housing project. It is an accumulation of five years of study that has been concluded in a yearlong in-depth analysis of architectural engineering systems. The focus of this report is to analyze the existing structural system and propose a new system in an attempt to improve the project cost, schedule and maintain the structural integrity of the building. West Village Housing Phases III & IV is located in Towson, Maryland on The Towson University campus. The project consists of two residential halls which will contain approximately 325,000 gross square feet of apartment-style accommodations for upper level students. The north building will be the building of study for this report.

Structurally, the existing gravity system is composed of 8" thick two-way post-tensioned concrete flat plates supported by reinforced concrete columns. Bays are approximately 27' by 24' with slight variances as the buildings shape changes. They are reinforced with ½" diameter un-bonded tendons in each direction and typical reinforcing, as required. The lateral system is made up of 12" thick concrete shear walls located around the stair and elevator shafts which will effectively resist the forces imposed on the building from all lateral loads. A structural overview of the existing concrete system is presented in greater detail within the first portion of this report. The remainder of the report will focus on the steel redesign of the building.

The primary structural redesign of the building was accomplished by implementing a non-composite steel beam system with hollow core planks. The goal to reduce project schedule and cost while maintaining a minimal floor depth were driving factors for a lot of the design decisions made throughout the redesign process. Bay sizes were limited to cooperate with the architecture of the apartments and exterior façade as well as to minimize beam depth.

The main lateral forces on the building including wind and seismic loads, were altered due to the change in stiffness and deflection from the proposed steel system. A conversion from concrete to steel results in a reduction of building mass. In order to accommodate this change, buckling restrained braced frames have been proposed to act as the main lateral system for West Village Housing's north building.

Two breadth topics were investigated to further the study on West Village Housing's north building. One breadth involves a critical path schedule analysis of the proposed structure as well as a cost comparison between the two systems. In addition, an acoustical analysis was performed for the second breadth. This investigates the sound transmitted between apartments and how the new structure effects that.

The final result of this thesis program is that the proposed steel system is a feasible and relatively cost effective system. It would fast track the project schedule and potentially save some of the upfront costs that are typical on a construction project. Both systems achieve all aspects of favorable structural design. Despite the success of the proposed steel system, it is important to note that the existing system is still acceptable.

The continuation of this report will cover all of the above elements of this project and more in greater detail.

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1 Introduction

1.1 Building Overview

The North building of West Village Housing Phases III & IV is a high-rise residential apartment building located off of Emerson Drive on Towson University's campus. This 9 story building will reach a height of 92' when it is completed in the summer of 2016. The residential units are two and four-bedroom student apartments with shared kitchen, living and bathroom areas. Each main floor contains a quiet study as well as an open lounge area to break up the floor plan which creates a common area for the students. The 1st floor opens up to a lobby with large store front windows to allow light through the building. A large multipurpose room is located on the North West corner with great views and has close proximity to the housing and residents life suite. The HRL suite is purposefully located on the main floor for easy access to any student who needs aid but also provides privacy for the resident's life tenants. The building is located in the West Village area of campus, which already contains six residence halls with more than 2,100 student beds; a Commons facility with dining areas, student service space and a 1,500-space parking garage. The educational facilities are located just a short walk east of the project site.

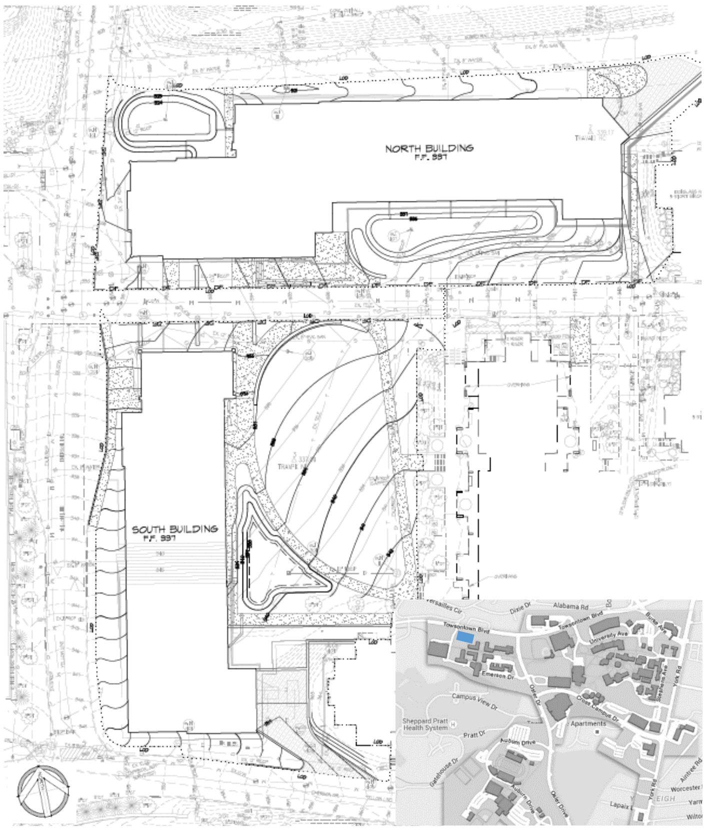


Figure 1: Building Location on Site

Ayers/Saint/Gross based out of Baltimore, Maryland were the lead architects on the project and worked closely with Towson University Construction Services to develop these apartment buildings on campus. Hope Furrer Associates, the structural engineers on the project, simultaneously worked with the architects to produce an efficient structural system that molded well with the architecture and met the client's needs.

Figure 2: Birds-eye view of north building

The University has contracted with Whiting Turner Contracting Company to provide pre-construction services during the design phases, and to be the Construction Manager at Risk during construction. Similar to most new buildings on Towson University's campus, the two new residence facilities will be built in adherence with sustainable design and construction standards and are expected to achieve LEED Silver certification.

West Village Housing's new North building consists of 8" thick two-way post-tensioned concrete flat plates supported by concrete columns. The foundation of the structure is supported by concrete columns and rammed aggregate piers (RAPs) along with spread footings, and foundation walls. Concrete shear walls located on the central axis of the building on both East and West sides are the primary lateral force resisting system and span the entire height of the building. These elements shall be founded on 48" thick mat foundations and shall extend out from the face of the wall. Concrete shear walls shall be assumed at all stair and elevator walls. Roof construction, shall be post-tensioned concrete flat plate similar to the typical floors noted above. In the pages to follow, each component of the structural system will be explained in more detail.

1.2 Foundation System

A geotechnical study was done by D. W. Kozera, Inc. who was able to provide useful recommendations for the foundation to the design team and structural engineers. Foundation levels are found at nominal structural depths or frost depth, and the encountered soil at footing subgrade are natural soils, newly placed compacted structural fill, or existing fill soils. The existing fill soils were found to be insufficient to support the foundation due to the unknown nature of their locations on site. In order to utilize the conventional spread footings at the North Building, Rammed Aggregate Piers (RAPs) are required.

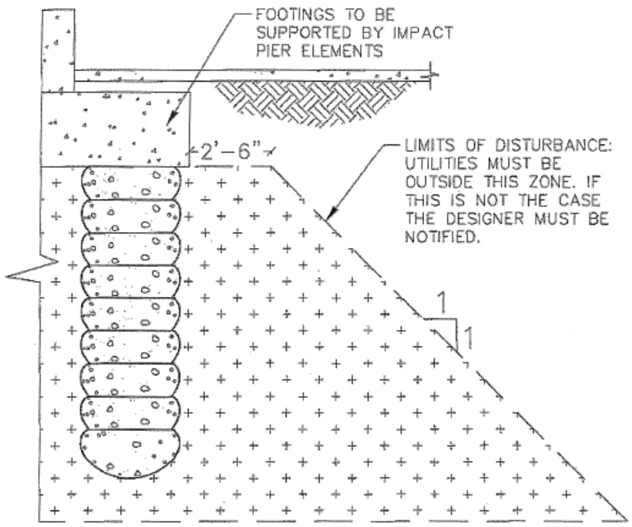


Figure 3: Example of RAP disturbance zone

RAPs are installed by constructing successive layers of densely compacted aggregate in a drilled vertical shaft measuring between 24 and 36 inches in diameter. The aggregate is then tamped making a ramming action which produces lateral prestraining and prestressing in the adjacent soils. Additional lifts of graded aggregate are then successively placed, creating a continuous shaft. The use of traditional shallow spread footing foundations can be then used because of the increase in strength and stiffness due to the compacted aggregate shaft. These piers are designed for an allowable bearing pressure of 5ksf. A simple RAP disturbance zone diagram can be seen in Figure 3.

The RAP ground improvement is typically designed and installed by a qualified contractor, in conjunction with the information provided by the structural engineer. Considerations that affect the design of RAPs include ground water elevations above the RAP tip elevation, soft or loose soils that may collapse during excavation, and the potential for construction debris in the fill soils. RAPs should fully penetrate the existing fill and loose residual soils and extend into dense residual soils. Therefore, RAP shaft lengths ranging from 15 to 30 feet below the finished floor exist for West Village Housing's north building. Once rammed aggregate piers are installed, the compacted/confining zone surrounding the RAPs cannot be disturbed. The disturbance cannot occur after the RAPs are installed regardless of whether the building load is applied yet or not.

The column footings (Figure 4) were recommended to be designed to have a maximum load of 700 kips. All footings are at least 16 inches deep for shear considerations. In addition, they are placed at least 30 inches below final exterior grade for frost protection. At the east end of the North Building, up to 15 feet of the wall is below final exterior grade. Therefore, the perimeter walls were required to resist lateral earth pressure loads. Designs and parameters can be seen in Figure 5. Walls shall be 12" thick reinforced with #5@12" vertical bars on each face and #4@12" horizontal bars on each face and be supported by a continuous wall footing.

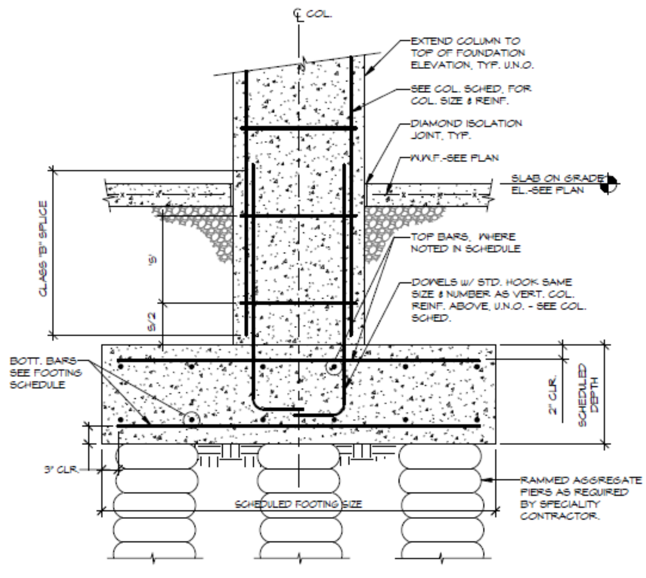


Figure 4: Typical column footing detail

A subgrade drainage system is required because the floor slab of the lower portion of the North building is below the surface of the groundwater table. It is designed to collect groundwater around the perimeter of the building below grade. Figure 6 shows the subgrade drainage detail and dimensions.

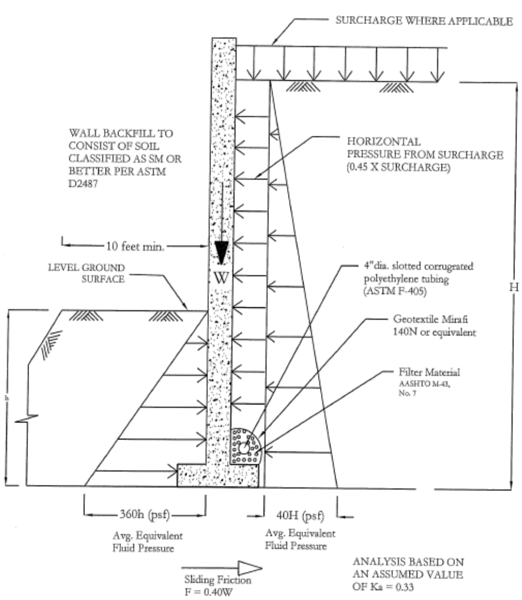


Figure 5: Lateral earth pressure diagram for cantilevered walls

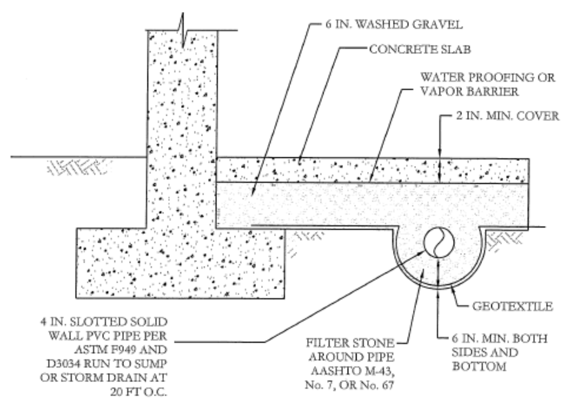


Figure 6: Typical sub drainage detail

1.3 Gravity System

The gravity system is comprised of the floor system and vertical columns in order to take the load to the foundation. The 3 sections below provide further detail.

1.3.1 Floor System

As previously mentioned, West Village Housing North Building is a concrete structure utilizing 8" thick two-way post-tensioned concrete flat plate supported by concrete columns. The slab is reinforced with ½" diameter un-bonded tendons in each direction and typical grade 60 reinforcement, as required, in other locations. The average pre-stress force in the slab shall be 18 kips/ft in each direction and the average weight of mild reinforcement shall be 2.5 lb/s.f. For the majority of the building, banded tendons run along the long axis of the building with uniform tendons running perpendicular. A painted textured finish with a UL 2-hour fire rated assembly is applied to the underside of the concrete. Below is a section showing reinforcement locations for a slab-column connection.

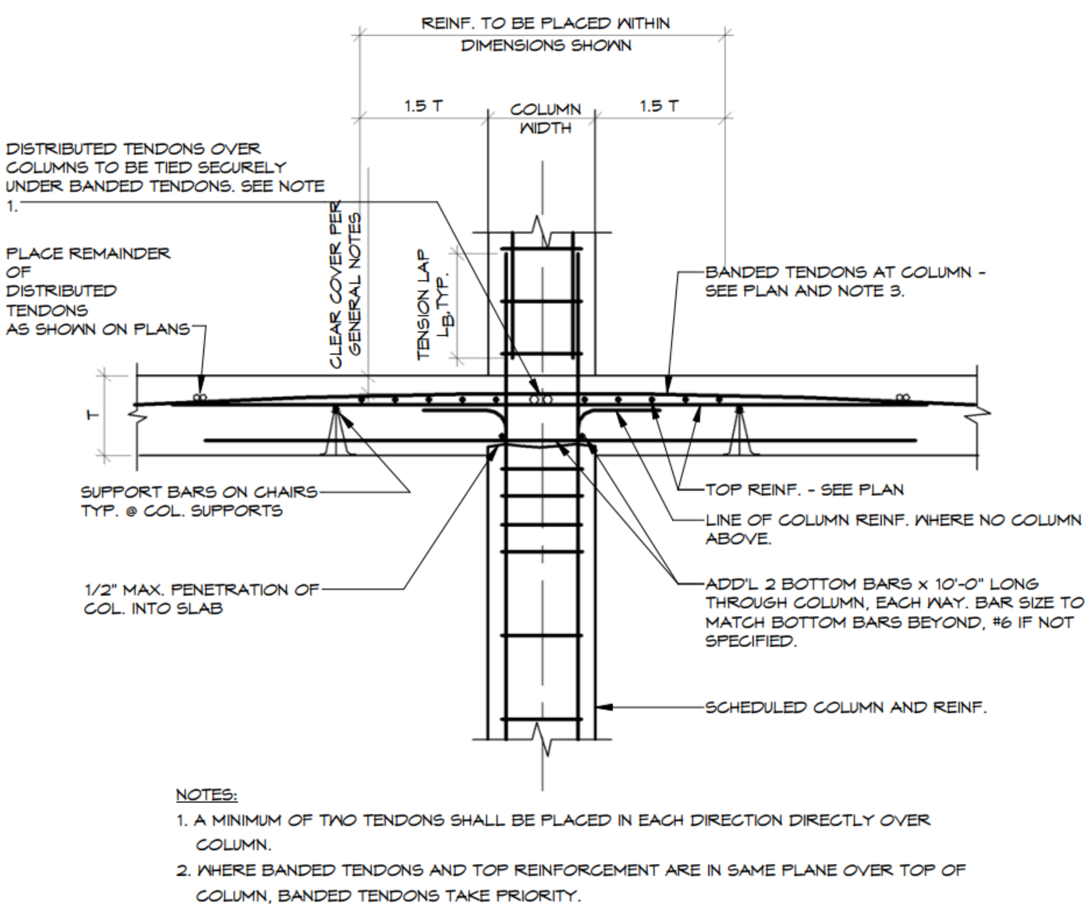


Figure 5: Typical column-slab connection

Concrete columns are another main aspect of the gravity system. As shown on the following figures, all the buildings columns have a 26" diameter and reinforced as follows:

Floors 1-4:
8#10 vertical bars with #3 ties at 18" o.c.

Floors 6 and up:
8#9 vertical bars with #3 ties at 18" o.c.

All column splices are Class B lap splices.

Figure 8 shows the complexity that goes into the construction of a typical column. Banded tendons must be aligned correct to line up with column lines.

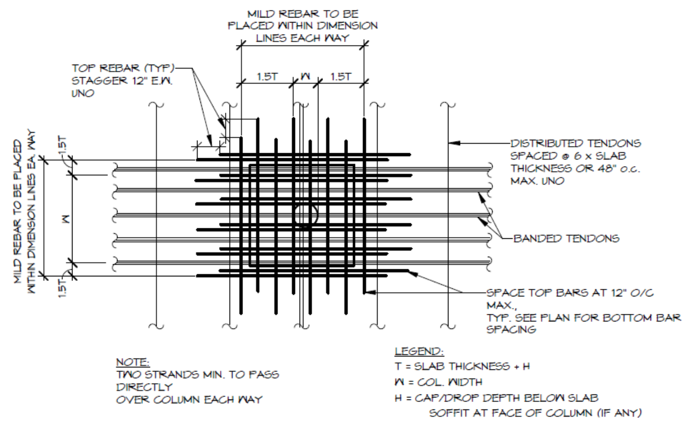


Figure 6: Top reinforcement at interior columns

1.3.2 Typical Bay

Due to the unique shape of the building and architecturally driven exterior, there are not "typical" bays. Multiple grid lines and non-symmetrical column spacing adds to the diverse bay layouts. Dimensions may vary per bay but the floor system described above is common between all of them. Figures 9 and 10, show the locations of the post-tensioned cables on the typical residence hall framing plan. The banded tendons running parallel to the long axis vary in

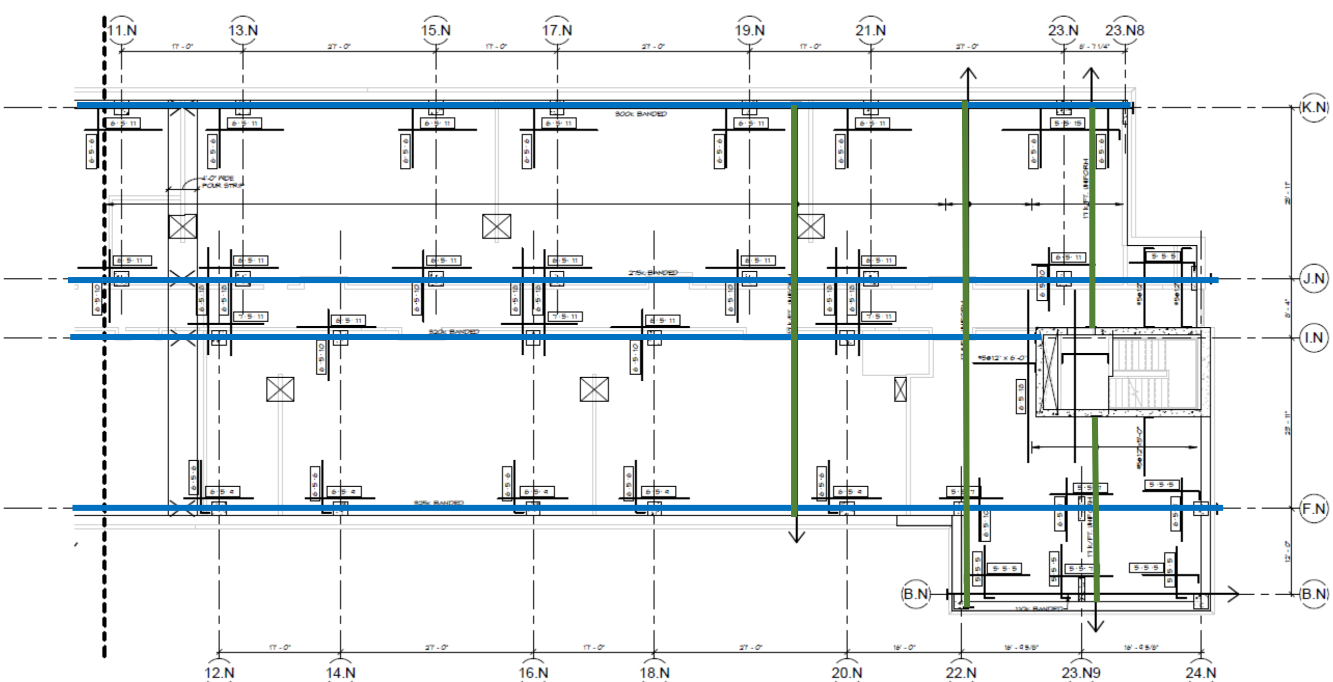


Figure 7: Typical Framing for East half of North residence hall

= Banded Tendons = Uniformly Distributed Tendons force from 200 kip to 325 kip while the distributed tendons have a typical strength of 17 kip/ft. There locations were guided by the layout of the hallway and adjacent apartments in the East-West direction. The South West part of the building has tendons that swap grid lines in order to connect all column lines as shown below.

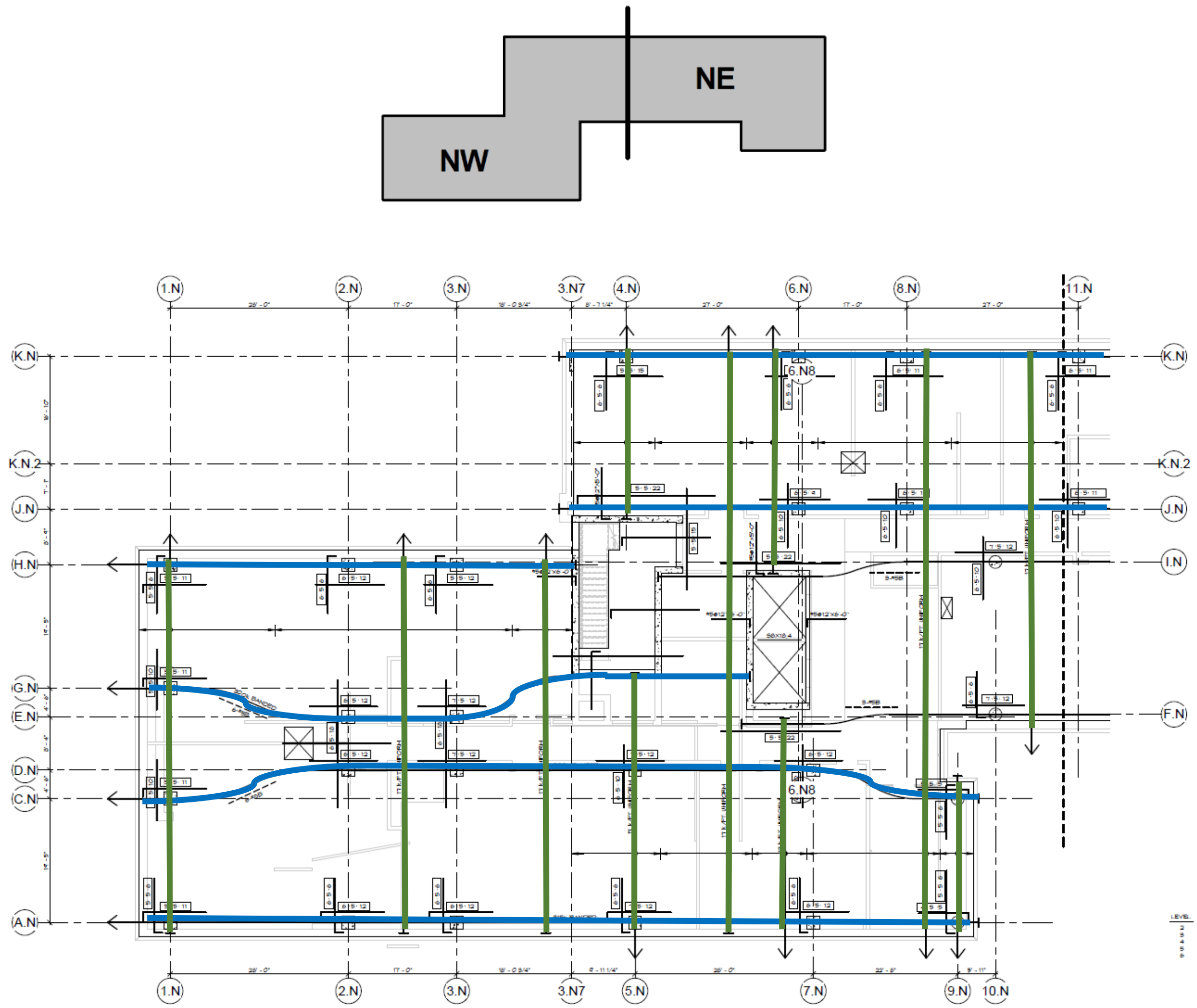


Figure 8: Typical framing for West half of North residence hall

1.3.3 Roof framing

Typical roof construction should be post-tensioned concrete flat plate similar to the typical floors noted above. As shown below, the slab is topped with a typical insulating concrete system. A green roof is also located on the South West portion of the building. It is comprised of hot fluid applied rubberized asphalt membrane that is fully adhered to the concrete deck, insulation, drainage retention mat and other planting materials noted in Figure 12.

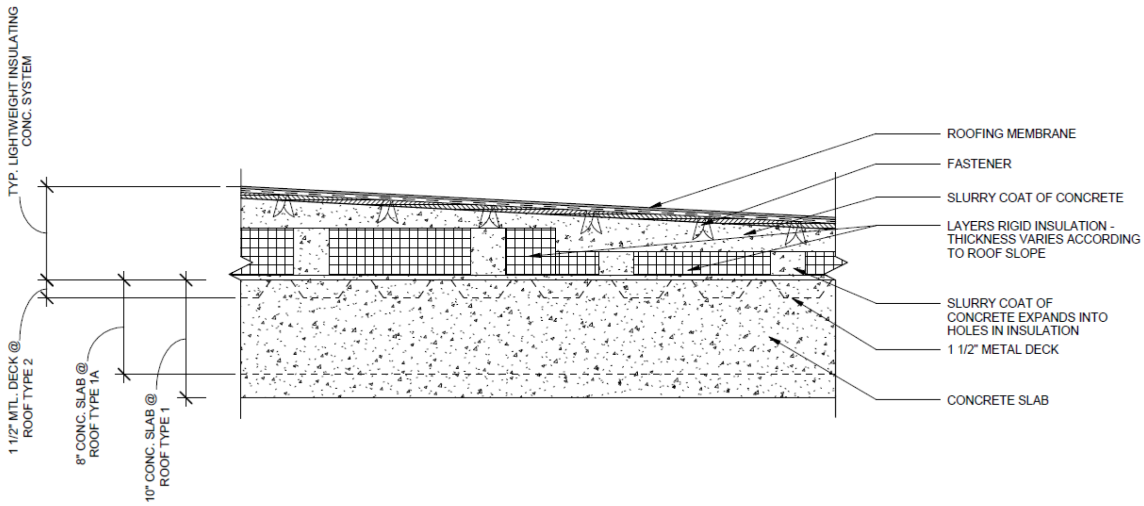


Figure 9: Lightweight insulating concrete roof detail

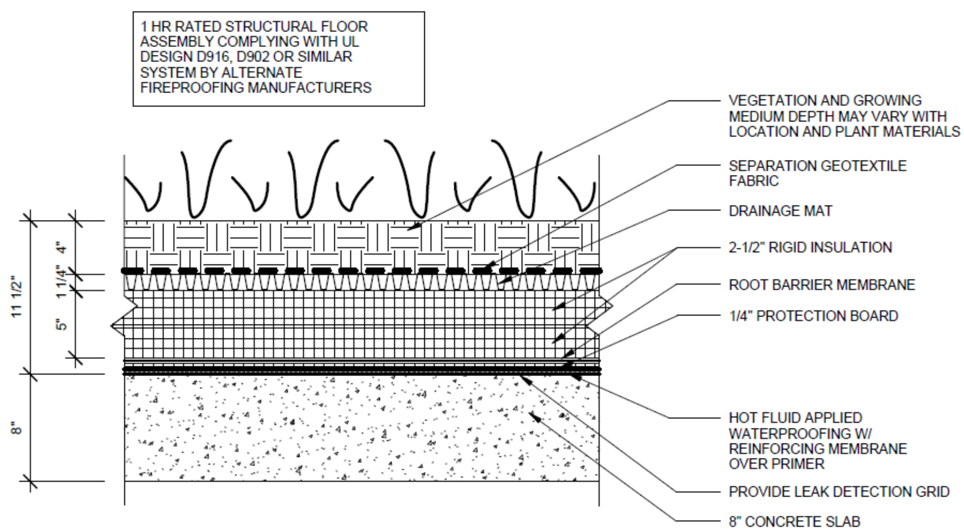
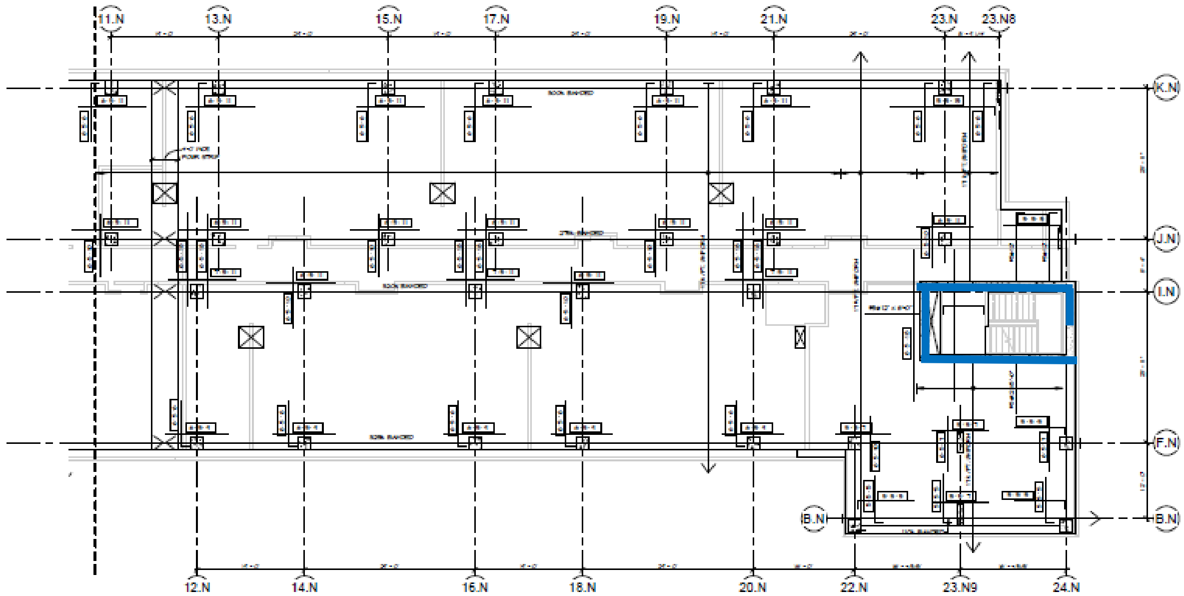
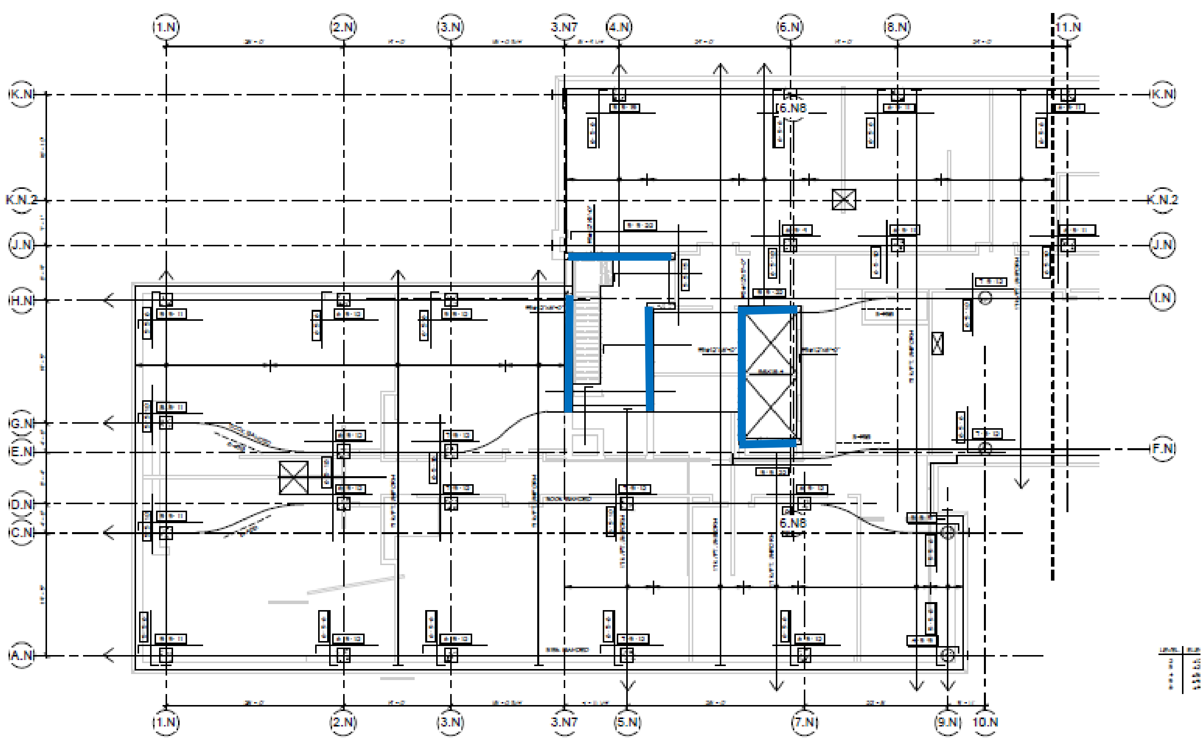


Figure 10: Green roof detail

1.4 Lateral system

The lateral force resisting system (LFRS) of West Village Housing North Building consists of 10 regular concrete shear walls that are 12" thick. They are located at the East and West ends of the building to effectively resist the forces imposed on the building due to the wind and seismic loads. All stair and elevator walls are concrete shear walls with an average weight of reinforcing of 7 lb/s.f.





The reinforcement in each wall consisted of #5 bars at 12" O.C. both vertically and horizontally. In industry this is common reinforcement for shear walls and is uniformly applied to all the shear walls in the building regardless of height or location. Below is Figure 13, showing elevations of two shear walls located on the west side of the building.

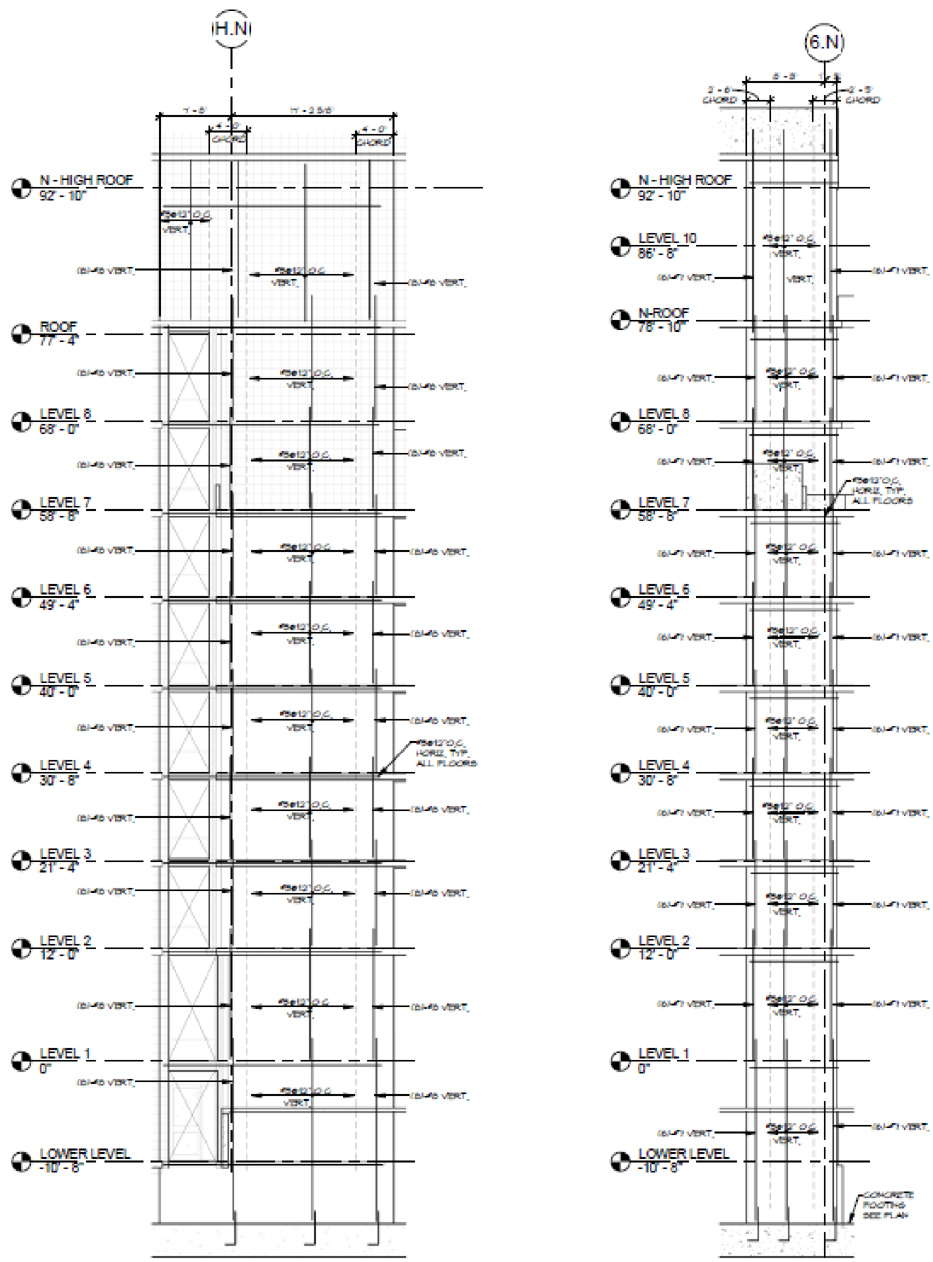


Figure 11: Shear wall elevations on west side of building

1.5 Joint Details

The figures on the right show some of the typical construction joint placements for the concrete slabs. The placement of a contraction joint seen in Figure 14 of is to control cracking for good overall structural behavior. The reinforcement holds random cracks tightly, keeping them small. Contraction joints are located on column centerlines with intermediate joints located between column lines, as required, to provide a maximum distance between joints of 15'. Construction joints (Figure 15 and 16) shall be placed in the slab where the contractors concreting operations are to conclude or be interrupted. This means that every time a concrete slab is finished pouring a construction joint is place to separate it from the next adjacent slab that still needs to be poured.

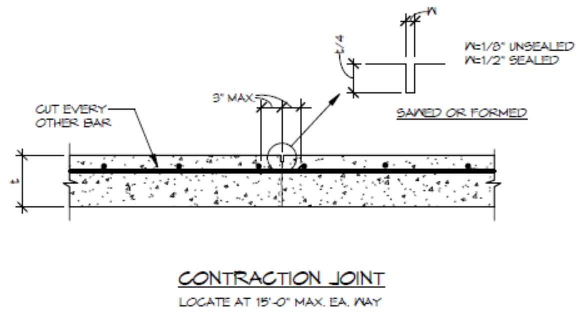


Figure 12: Typical contraction joint used to control random cracking in the floor slab

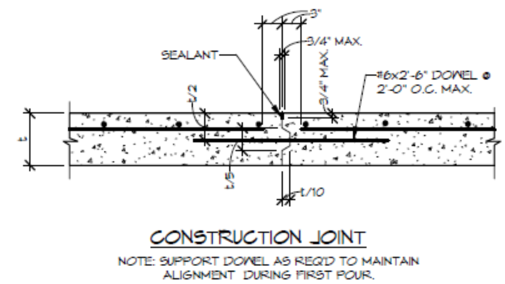


Figure 13: Typical construction joint

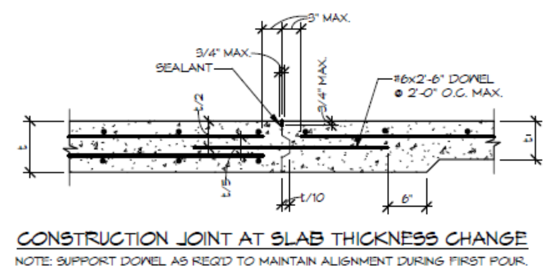


Figure 14: Typical joint when the slab alters its size at the exterior of the building

Joints at the exterior walls require sealants due to moisture. Silicone sealant is applied at exterior non-traffic joints and multi-component urethane is applied as traffic sealants. The joints that are between systems are faced with sheet membrane tape between cladding materials and at all penetrations to seal the air barrier.

1.6 Determination of Design Loads

This projects design live loads, wind loads and seismic loads are in accordance with IBC 2012 and ASCE 7-10. Through standard practice over many years, engineers have accumulated standard live and dead loads that can be found in both building codes. Some typical loads used on this project are shown below. The below information does not cover all loads applied to West Village Housing's North building. It should however, provide a basis for typical loads seen

LOADING SCHEDULE (PSF)

LOADING \ LOCATION	TYPICAL FLOOR	TYPICAL ROOF	PENTHOUSE FLOOR	PENTHOUSE ROOF		
CONCRETE SLAB	100	100	125			
METAL DECK	-	2	2	2		
M/E/C/L	8	8	8	8		
MEMBRANE	-	3	3	3		
ROOFING	-	6	6	6		
INSULATION	-	6	6	6		
PARTITION (LIVE LOAD)	15	-	-			
TOTAL DEAD LOAD	108	123	150	25		
LIVE LOAD	55	30	100	30		
TOTAL LOAD	163	153	250	55		

NOTES:
1. ALL LIVE LOADS ARE IN ACCORDANCE WITH INTERNATIONAL BUILDING CODE 2012 EDITION.
2. LIVE LOAD REDUCTION IS NOT SHOWN. HOWEVER, LIVE LOADS HAVE BEEN REDUCED WHERE ALLOWED BY CODE.
3. TOTAL DEAD LOADS DO NOT INCLUDE WEIGHT OF STEEL OR PRIMARY FRAMING MEMBERS.
4. LOADS IN SCHEDULE DO NOT INCLUDE WEIGHTS OF ROOF TOP MECHANICAL UNITS. THE PROVISION FOR THE SUPPORT OF THESE UNITS HAVE BEEN MADE ON AN INDIVIDUAL BASIS. ANY CHANGE FROM SPECIFIED MECHANICAL UNIT (SIZE, WEIGHT AND LOCATION) SHALL BE BROUGHT TO THE ATTENTION OF THE STRUCTURAL ENGINEER.
5. SEE PLANS FOR LOCALIZED CONCENTRATED LOADS.
6. DRIFTED AND SLIDING SNOW LOADS ARE ACCOUNTED FOR IN ACCORDANCE WITH INTERNATIONAL BUILDING CODE 2006 EDITION, BUT ARE NOT INCLUDED IN THE LIVE LOADS INDICATED ABOVE.

Figure 15: Loading schedule

in industry that apply to this building in particular. They are based on industry standards as well as engineering judgment from Hope Furrer Associates, the structural engineer on the project.

The structural dead loads include the load from slab, concrete topping and finishing, columns and beams. In addition, superimposed dead loads are applied to certain areas such as mechanical rooms for MEP units, roofing systems accounting for additional roofing materials and any rooftop concrete pavers that may be present.

Structural live loads vary upon occupancy and the use of the space. The loads used in this design can be found in ASCE 7-10.

Soil Loads for the building were calculated using the geotechnical report provided by Kozera D. W and will be detailed in further reports.

Snow Loads were determined from ground snow load maps in ASCE 7-10. Snow drift was calculated using the procedures given in the code.

1.7 Load Paths

The combination of dead and live loads results in a total gravity load that is applied to the building. The concrete floor slabs will resist these loads at every floor where load is applied or carried through the building. Each bay is defined or bound by either concrete columns or shear walls. These elements will take the load from the slab directly down to the base of the building into the foundations that they rest on. Through complex soil properties and engineered gravel the load is dispersed into the ground.

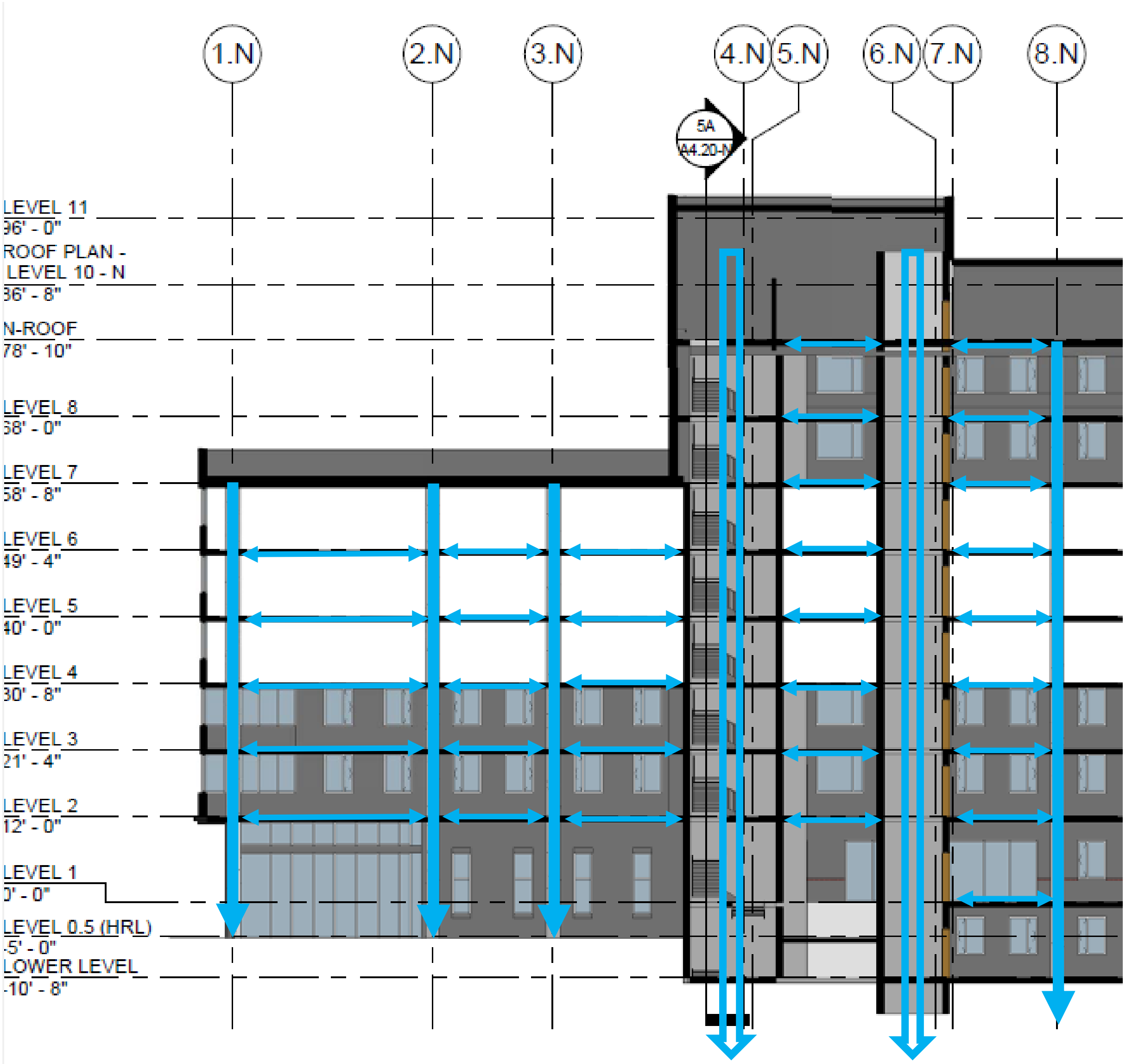


Figure 16: Longitudinal section through north building showing gravity load path

Lateral Loads are sometimes considered more complicated because they are more variable and abnormal than typical gravity loads. Wind forces are applied to the exterior façade of the building as a positive or negative pressure. Connections from the exterior façade to the main structure will transfer the load to the slab creating a horizontal force. In structural engineering, it is common knowledge that load follows stiffness. West Village Housing Building North can be considered a rigid diaphragm because it is comprised of concrete. Therefore, the load will be distributed to the LFRS elements based on their stiffness. Generally, the stiffer the shear wall is the more lateral load it will attract. After the load is distributed to the shear walls, it is then carried down to the foundations.

Earthquake shaking generates inertia forces that are considered seismic loads. These loads or lateral accelerations are applied anywhere there is mass in the building. Typically the higher the building the greater the seismic loads because of the greater amount of mass that is accumulated. The slab-frame connections previously mentioned in the report attracts the shear from these loads and distributed them to the reinforced concrete shear walls. When seismic loads are controlling, foundation design is more crucial to the project. This is because these forces are dependent on the mass of the building so the foundations have to be carefully designed to support the entirety of the load.

A RAM model of the existing concrete structure is shown in Figure 17.

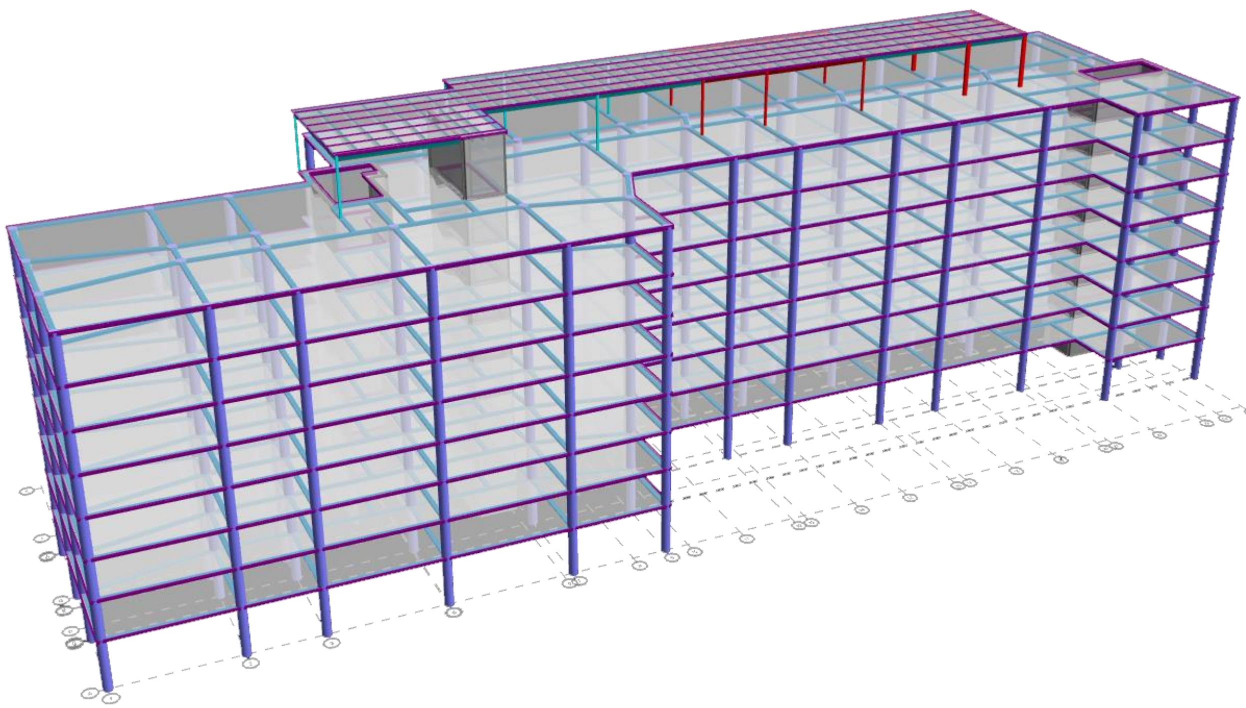


Figure 17: Existing concrete structure RAM model

2 Design Codes, Standards and References

The following is a list of some of the structural codes used on this project. These codes will be used and referenced in all future calculations and design work for this thesis.

International Code Council

- International Building Code, 2012 Edition
- International Building Code, 2006 Edition
  - Used for drift and sliding snow loads only

American Society of Civil Engineers

- ASCE 7-10: Minimum Design Loads for Building and Other Structures

American Concrete Institute

- ACI 318-11: Building Code Requirements for Structural Concrete

Post Tensioning Institute

- Post Tensioning Manual, 6th Edition

PCI Manual

- Design of Hollow Core Slabs, 2nd Edition

American Institute of Steel Construction

- Steel Construction Manual, 14th Edition, 2010

Structural Welding Code

- Steel ANSI/AWS D1.1-10:2010

RSMeans Cost Data

- Facilities Construction 2014, 29th edition

Towson University Construction Services (owner)

- Architectural drawings for project
- Geotechnical reports

Hope Furrer Associates (structural engineer)

- Structural drawings and details for project

3 Structural Design Alternative

The north building of West Village Housing Phases III & IV consists of a reinforced concrete flat plate post tensioned system as well as reinforced concrete shear walls. Previous reports and notebook submissions, have determined that the structure is acceptable for both strength and serviceability requirements.

A new theoretical scenario has been proposed in which the architect and owner prefer the use of a new system rather than post tensioned concrete. This system would utilize precast concrete planks and steel joists for the floors as well as light weight metal gauge bearing walls. The redesign must consider the floor heights as well as the bay sizes when evaluating the appropriate redesign. A detailed schedule will have to be determined as a new design and new materials would adjust the current construction schedule. In addition, an acoustical analysis should be performed to ensure the comfort of the occupants would not get worse due to the structural changes.

3.1 Design Proposal

The proposed solution for the design is a precast concrete system with steel joists and bearing walls used for the gravity system with the use of buckling restrained braced frames for the lateral system. In order to maintain the current stair/elevator towers, the lateral system configuration of the building will remain the same as it has been proven to function as an efficient lateral arrangement. RAM will be used to analyze the gravity system and lateral system in concurrence with hand spot checks.

Multiple factors based the decision to explore this new system. Precast concrete planks are an efficient way to construct longer clear spans and use the same deck because of prestressing. This economical design can reduce the schedule for construction, simultaneously reducing the project cost. Precast hollow core slabs generally allow for lower sound transmission, heating and air conditioning cost per square foot. In addition, the use of light weight metal gauge bearing walls could reduce the overall building mass and effect of seismic loads on the building. To simplify the layout and take advantage of the allowable precast clear spans, the current column grid will have to be adjusted which could help reduce the amount of columns.

A buildings structure can affect many aspects of the project. One issue that is common in residential halls is high noise levels being heard from room to room. Another, is the simple deadline of the project and when people can occupy the building. These two factors were chosen to be evaluated further. In order to facilitate this design proposal, two breadth areas (acoustics and construction management) will be covered, in addition to the depth, to enhance the design and scope of work for the building.

3.2 Construction Management Breadth

The alternate system will have an effect on both the cost and critical path schedule of the project. A critical path comparison will be conducted for the new system in hopes that the overall project schedule will be shortened. The current systems cost is not to be revealed because of the request of the owner. However, calculating if the new system will reduce overall cost can be determined. This cost and schedule data will be used to determine the feasibility of the alternate system.

3.3 Acoustics Breadth

Sound transmission through walls and floors are a big factor for the comfort of occupants especially in residence halls. The architects and owners on this project would like to maintain or even reduce their current sound transmission criterion (STC) values. Changing the structure of the building will alter the amount of noise that is heard within the building. An acoustical analysis of a typical floor in the building will be performed for the current design as well as the proposed design to ensure that the STC values do not exceed the limits. In addition, other ways of reducing the noise transmission between rooms will be investigated.

4 Structural Depth

The proposed structural redesign of West Village Housing's north building included the redesign of both the gravity and lateral system. This is to investigate the feasibility of a steel system for the project. Throughout the design process multiple constraints and architectural factors had to be considered. One of these factors is the height restriction and floor to floor height. A typical structural engineering goal is set to minimize the structural depth. This will be addressed in the gravity system redesign portion of the report. In addition, bay size and column spacing would ideally be maintained in the redesign. This would limit the architectural changes of the building allowing for a more ideal comparison between the original and redesigned system. One aspect that is required for a steel system and not for concrete that was taken into account in this report is fireproofing. Using steel beams and columns requires the use of spray on fireproofing in order to maintain safety and meet code requirements within the building. These factors are just a few of the considerations that were taking into account in the design process.

Through implementation of a typical professional structural design process, the following analysis was performed.

New gravity and seismic loads needed to first be determined. This allowed for the selection of a new roof and floor elements. The next step is to design the gravity system. A gravity model was created in RAM Structural System. The previously determined building loads were input into the program and member sizes were optimized by computer calculations. These sizes were verified through engineering judgement and hand calculations which can be found in Appendices portion of this report. A goal, as previously stated, was set to maintain the buildings bay sizes which are typically 24' x 27'.

Once the gravity system elements such as the floor type, beams and columns were determined, the building's lateral system was designed. Wind and seismic loads were determined for Towson, Maryland and then input into the RAM model. The layout of the braced frames or the lateral force resisting elements were then determined and checked through spread sheets and software to assure there locations were adequate to resist the given loads. Braces were designed and spot checks were conducted.

4.1 Gravity System Redesign

The gravity system redesign of West Village Housings north building takes advantage of steel and precast concrete to reduce the construction period of the project. The analysis and design of these elements including precast hollow core plank, steel beams, steel columns and foundations are noted in the following sections.

4.1.1 Precast Hollow core plank

A precast hollow-core slab was chosen to support the floor and roof areas. Hollow core slabs are precast and prestressed concrete members that have continuous voids throughout there length. These voids reduce weight and cost therefore resulting in a favorable option for a redesign of a heavier and more expensive post tensioned concrete floor system. Another benefit of having these voids is the possibility of using them for electrical or mechanical runs. This helps aesthetically reroute lighting and HVAC circuits through the building. This is an efficient use of the planks, however is not within the scope of this report and will not be further investigated. Structurally, precast hollow core plank provide additional load capacity, span range and deflection control due to the prestressing of the member.

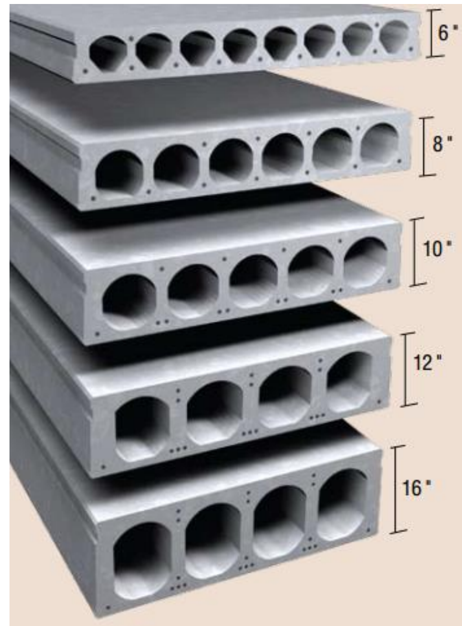


Figure 18: Elematic Hollow-core Plank

The location of the project site in Towson Maryland was taken into account when deciding what precast manufacturer would provide the planks. The nearest precast plant is Oldcastle Precast. The hollow core planks or trade name that they manufacturer are Elematic. Elematic's products were used in the design of the hollow core planks for West Village Housings north building. Examples of an Elematic plank can be seen in Figure 17 showing the possible plank depths and the prestressing within the slabs.

The redesigned system utilizes Elematic 4 foot wide planks that are 8 inches thick with a 2 inch topping. The slabs have a concrete 28 day strength of 5,000 psi with (6) $\frac{7}{16}$ inch strands. The strands are 270 ksi, low-relaxation steel. They span for approximate 24 ft which can be seen in Figure 18. The layout or direction of the slabs were chosen in order to direct the load to the edge beams and interior beams. RAM outputs showed a more problematic situation when the planks ran East-West in the building so the North-South orientation was chosen. The hollow

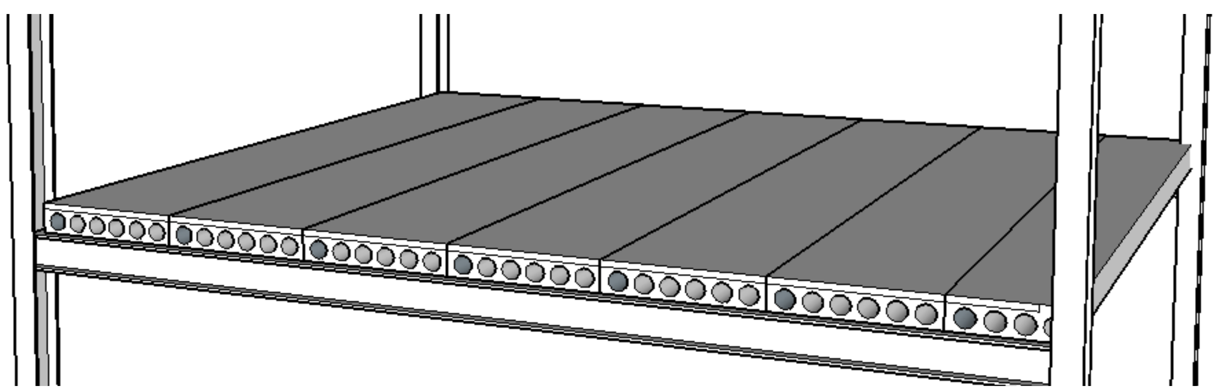


Figure 19: Example of proposed plank layout for typical bay

core planks with added topping that were chosen weigh 79 psf which is a reduction from the 108 psf used for the post tensioned slab design. In turn, this reduction in self weight will reduce the building structure and sizes of the required beams and columns in the following sections. The design and analysis of the precast hollow core slab accounted for all prestressing losses including elastic shortening, creep, shrinkages and steel relaxation. Detailed calculations and analysis can be found in Appendix B.

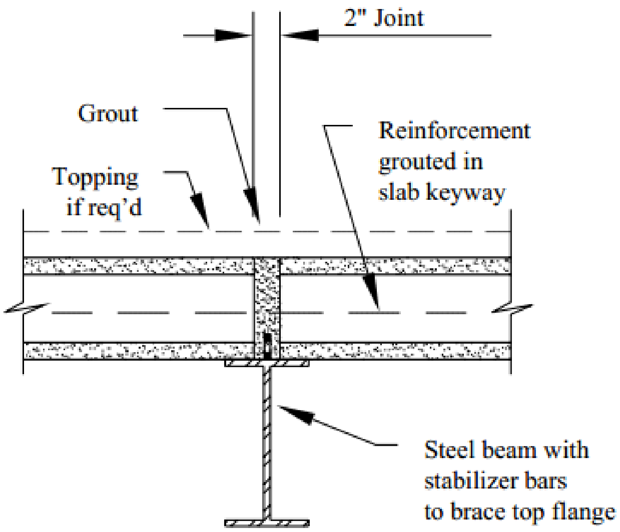


Figure 20: Hollow core to steel beam connection detail

In order to start the next step in the design process with gravity beams, a connection detail needs to be finalized. This can change the design method of the beams from composite to non-composite. After analysis and engineering knowledge from the architectural engineering faculty, the connection in Figure 19 was chosen. The connection can transfer internal diaphragm forces and provide lateral bracing to the steel beam. The grout adds stability and stiffness to the floor diaphragm. The floor acts as one unit but is not permanently connected to the steel beam. This allows the beam to be designed as non-composite.

4.1.2 Gravity Beams

Once the loads from the hollow core plank have been determined, live load values are found from ASCE 7-10 tables and a detailed connection is chosen, the gravity beams can be designed. The bay sizes remained the same but column lines had to slightly change in order to allow for a design that eliminates transfer beams which typically results in a higher cost. Column line I and column line E had to shift only a couple of feet to line up with the columns in the upper half of the building as seen in Figure 20. Figure 21, shows the current proposed structural grid. It allows us to optimize the steel performance by maintaining a typical bay size throughout the building. The columns are originally offset from one another because of the architectural layout of the building, the apartment configuration and the concrete system that allows this.

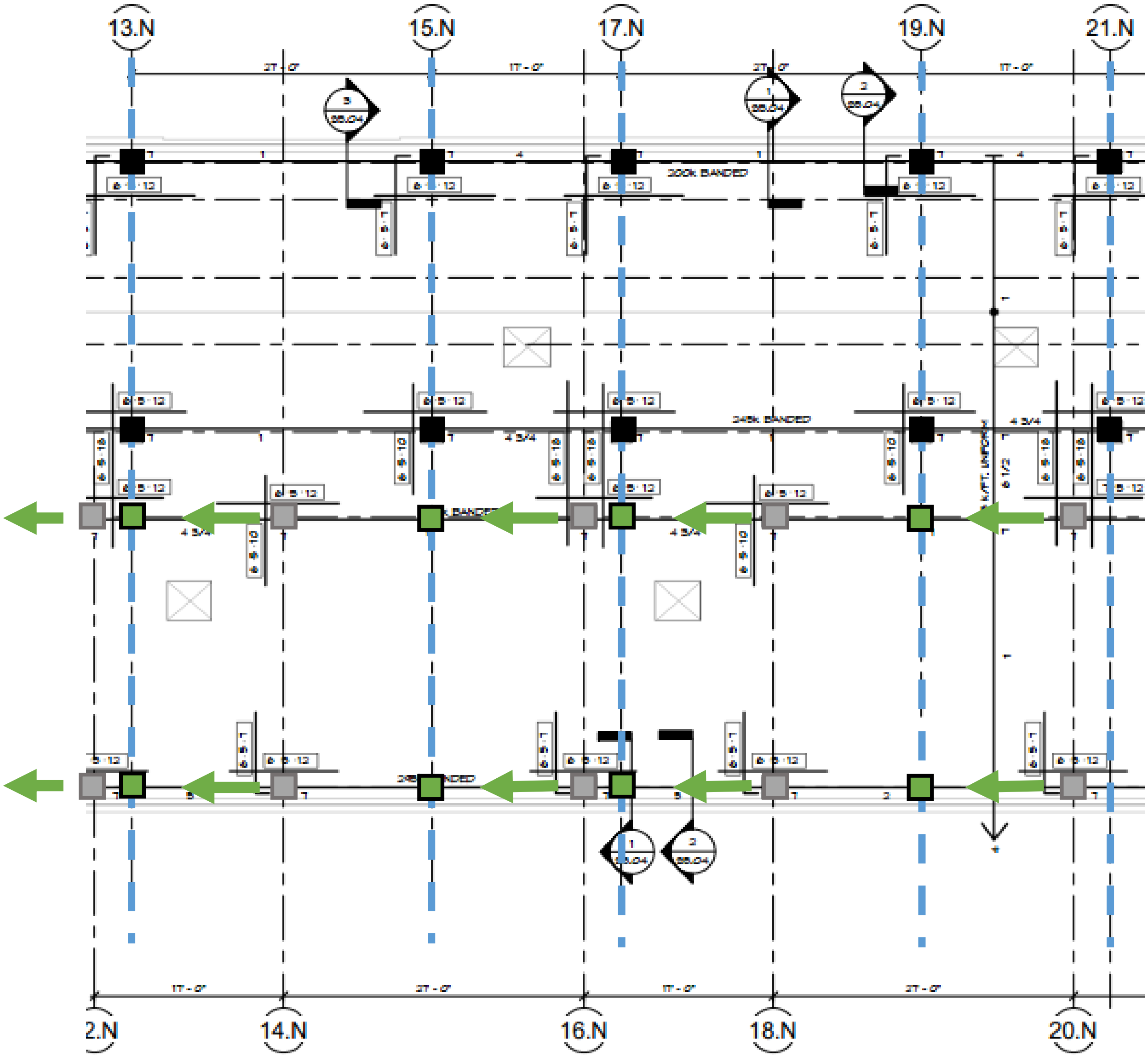


Figure 21: Proposal to shift column lines

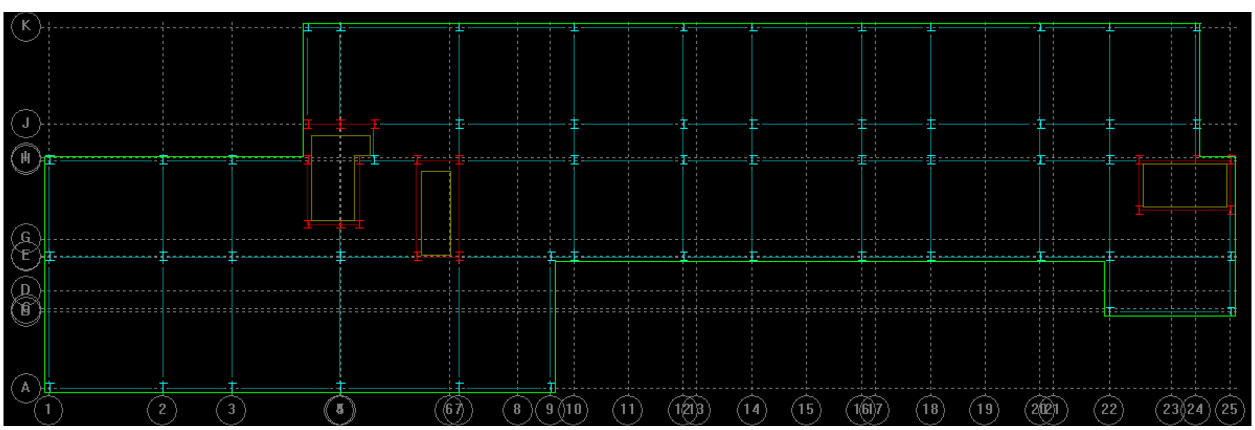


Figure 22: Proposed structural grid

Architecturally, the proposed system has only shifted the apartment units a couple of feet. This allows the layouts of the apartments to be unchanged in respect to the architect's original ideas for the residence hall. Figure 22, represents the apartment shifting by showing the outline of the 4 bedroom units.

Figure 23: Apartment layouts with shifted columns

The beam and girder sizes for the typical redesigned bay are W14's. Fortunately, due to the use of precast hollow core plank, infill beams are not required. The planks are a sufficient way to direct the loads to the girders. On the contrary, these beam sizes will increase the structural depth of the floors. Figure 23, shows the changes of floor to floor heights from the original to the proposed system. For this redesign, it was assumed that increasing the height of the building would not be of concern for the owner. As to be expected, the structural engineers on the project designed an efficient system that has a minimal floor depth.

Existing Floor Heights **Proposed Floor Heights**

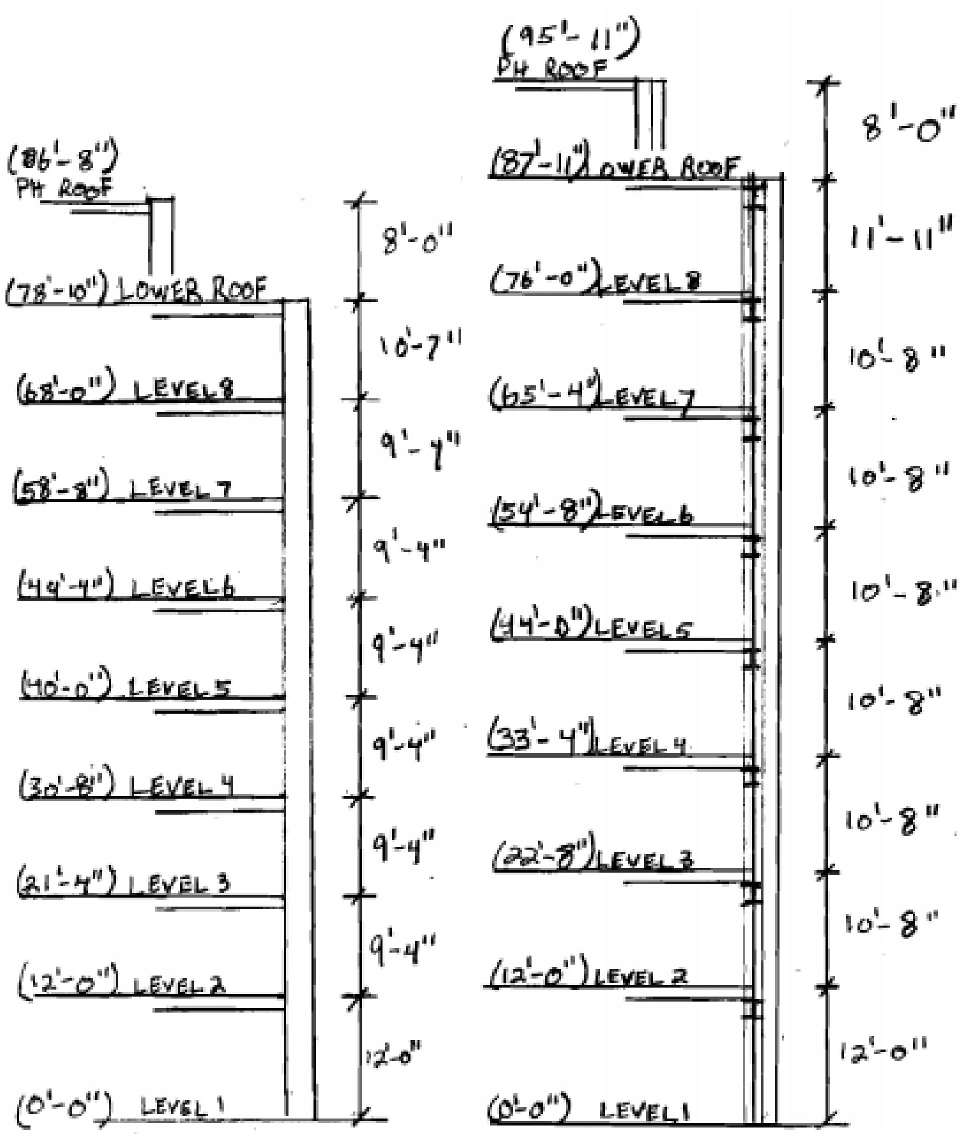


*Figure 24: Floor height differences between existing and proposed systems *This difference is the comparison of a concrete slab in the existing system VS. the required steel beam sizes for the proposed project.*

The more repetitive the steel members are from bay to bay, the more efficient the installation of the gravity system is resulting in a quicker and cheaper project. The final designs for the floor plans are below.

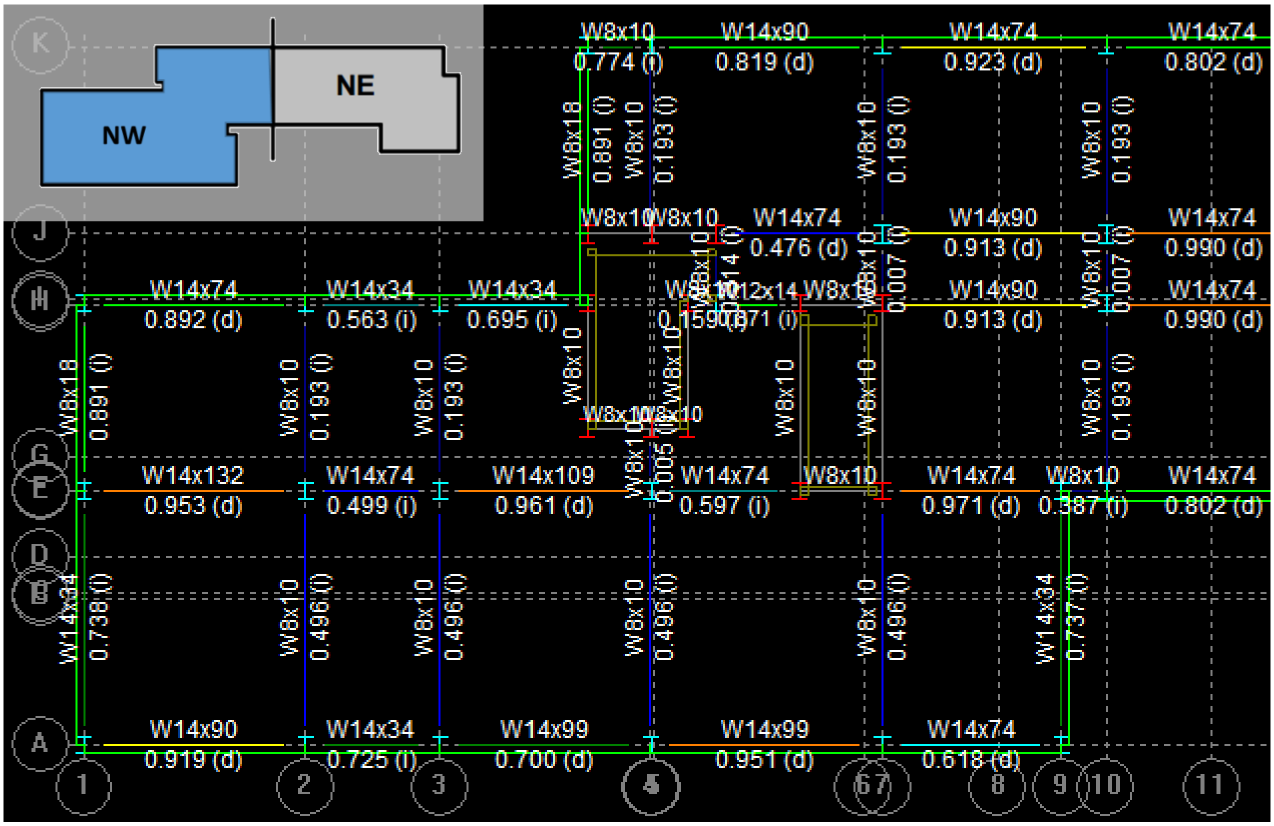


Figure 25: Proposed steel beam sizes for West half of building

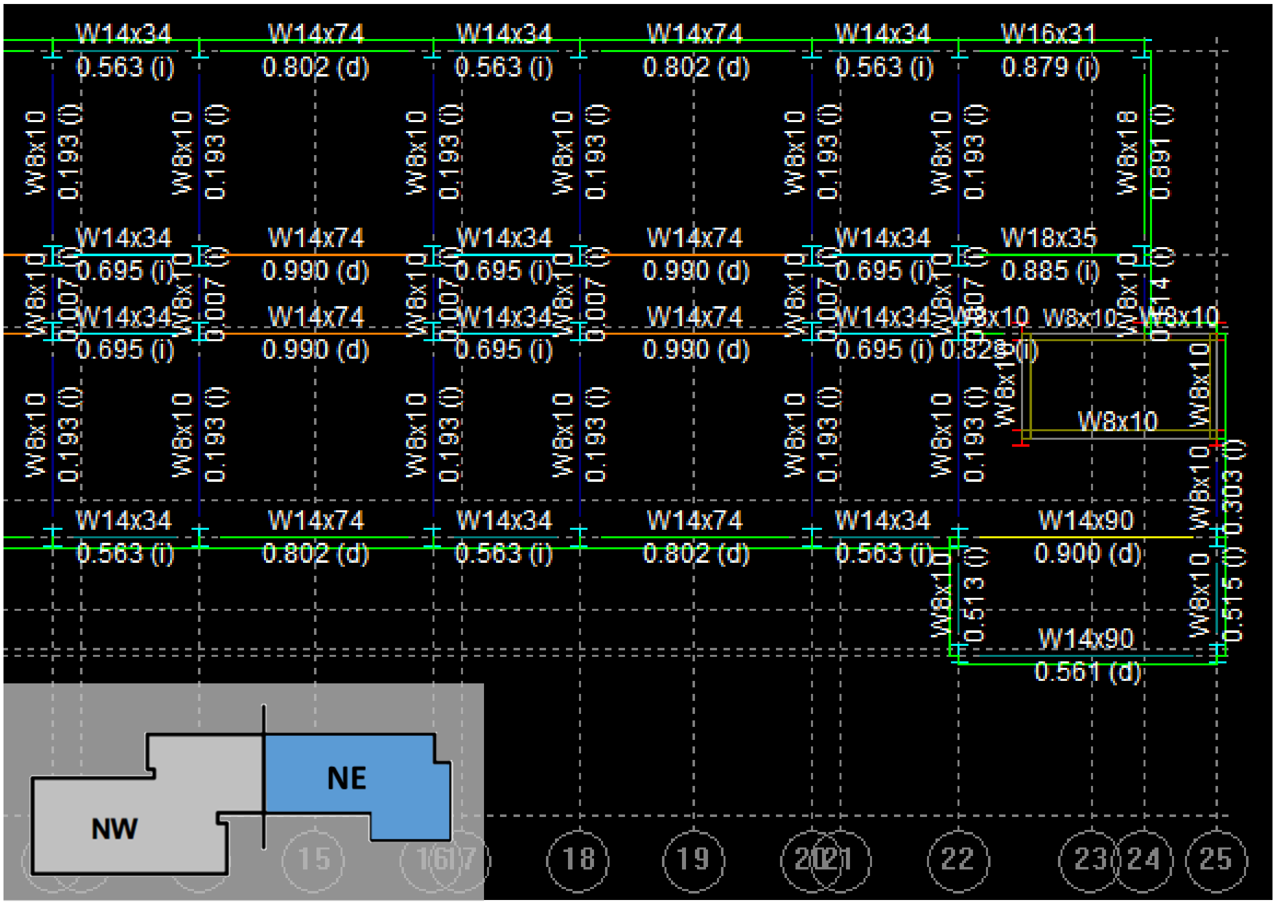


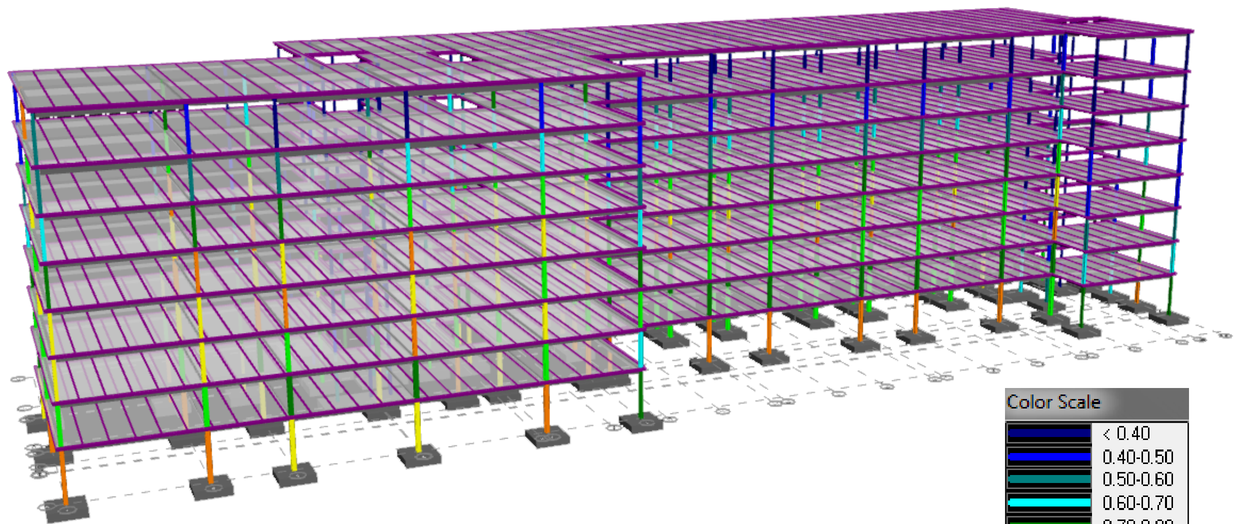
Figure 26: Proposed steel beam sizes for East half of building

The colors in Figure 24 and Figure 25 represent the steel utilization and whether strength interaction or deflection is the controlling factor when designing that member. This is also called the demand interaction relation or DIR. (i) shows that the interaction value is controlling and (d) shows that deflection is controlling. In addition, the warmer the color the closer the member is to failing in one of these factors. Values over 1.00, mean that the beam may fail under the given loads and that the member should be redesigned. Additional hand verification and minimum required size calculations can be found in Appendix C

4.1.3 Gravity Columns

Gravity columns were also designed using RAM Structural System. The columns are designed to be spliced every two floors through the full height of the building and connected to the gravity system via shear connections. Both interior and exterior columns are W12 sizes. The column designs for each of these sizes are compared based on total steel weight. Through engineering judgement and proper hand calculations found in Appendix C, the column designs were chosen.

Column splices in multi-story buildings are usually provided every 2 or 3 stories. This results in convenient lengths for fabrication, transport and erection. Column splices hold the connected members in line. The column splicing for West Village Housings north building is located every two floors. Therefore, the member sizes to change throughout the height of the building. This allows for the design to avoid using larger W14 or greater shapes. Thanks to splicing, the selected column sizes are able to maximize the utilization of steel. Figure 26 shows the column interaction colors for the building.



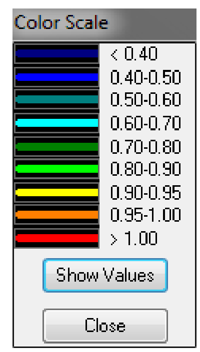


Figure 27: Column interaction colors

The orientation of the columns was generally decided based on the shape and geometry of the building. Its rectangular shape lends its columns to be oriented in the y direction. A previous report (technical report 4) proved that the deflection was greater in the y direction than in the x direction. This helped determine that the columns should be arranged with there strong axis in the y direction. This adds stiffness in that direction reducing the deflection. Column orientation in the y direction is displayed in Figure 27.

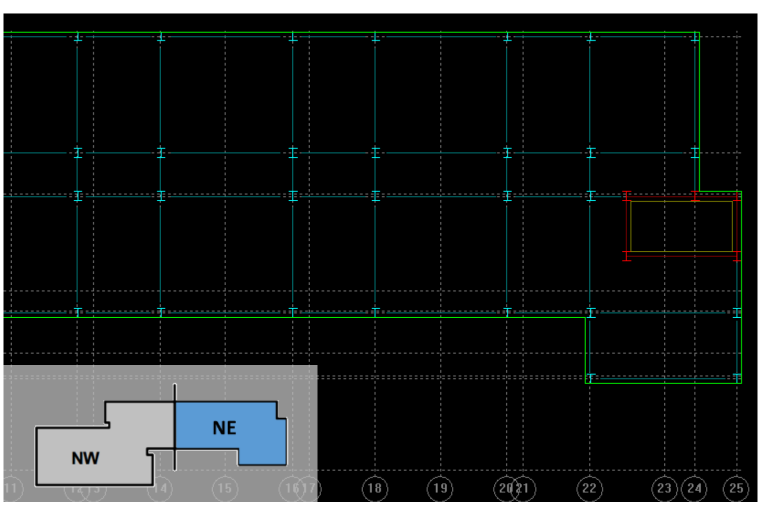


Figure 28: Column orientation

4.1.4 Impact on Foundations

Due to the proposal of switching from a concrete structural system to a steel system, the overall weight of West Village Housing's north building will decrease. As we will see in the next section of the report this weight reduction results in smaller seismic loads. The decrease in load may equate to a decrease in footing sizes. A full analysis of new foundation sizing is outside the scope of this thesis. However, one can predict that the rammed aggregate piers seen in Figure 28, would not have to be drilled as deep as they would be for a heavy concrete system.

Figure 29: Example of RAP installation

With the decrease in depth of the RAP's, foundation sizes that rest on top of the piers will be able to be reduced in size. The average column load from the RAM model was approximately 500 kips. Taken from the geotechnical report is the allowable bearing pressure of 5 ksf. With an f'c of 3000 psi and fy of 60,000 psi, the CRSI design handbook can be used to size the footing. The square footing with the parameters listed above would be a 8' x 8' footing that is 24 inches thick. The bars each way would be (6) # 8's with a max spacing of 18 inches. Though not included in the construction breadth of this report, these sizes would also help reduce the overall cost of the project.

4.2 Lateral System Redesign

The lateral system redesign of West Village Housings north building takes advantage of buckling restrained steel braced frames to reduce the construction period of the project and to help relieve the building of seismic loads. The analysis and design of these elements including center of mass, center of rigidity, wind forces, seismic forces and the design of the buckling restrained braced frames are noted in the following sections.

The nature of a lateral load depicts where the lateral loads will be applied on a building. For example, wind is a pressure force whereas seismic force is a function of mass. Therefore, these forces will act at different locations on the building. These locations need to be determined first to confirm that all lateral elements are placed in an efficient area in the building to maximize its lateral force resisting nature. The tables in the next sections locate those different points at which the forces act on and the exact values of those loads in order to design an adequate lateral system. It was determined from the following calculations that the proposed locations of the buckling controlled braced frames was sufficient to resist all lateral loads. In addition the stiffness of each element was adequate. Figure 30 shows the diaphragms and braced frames in the RAM model that was used to design the lateral system.

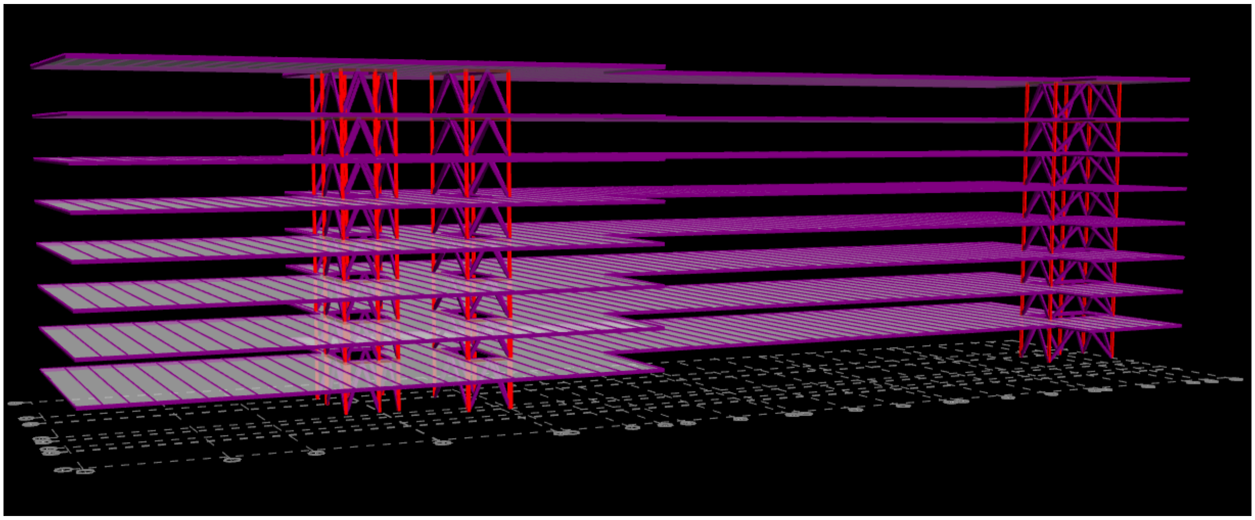


Figure 30: RAM model showing diaphragms and braced frames

4.2.1 Center of Rigidity

The center of rigidity or COR of a building is the centroid of the stiffness for that building. The stiffness elements that are being considered for the steel proposal include all 12 braced frames as they are the only main lateral force resisting elements. Any forces that are applied on a point of the building other than the COR will cause torsion or twist on the building. This is due to the eccentricity of the load applied to the centroid of stiffness. Braced frames are applied at either end of the building to help reduce the difference between the center of rigidity and the center of mass. This leads to a minimal torsion on the building. Table 1, shows the methodology through a spreadsheet for calculating the center of Rigidity. The element labels can be seen in Figure 31.

Table 1: Center of Rigidity

ELEMENT LABEL	ELEMENT DIRECTION	DIST. FROM REF. DATUM X (FT)	Y (FT)	Rx	Ry	Rx*Y	Ry*X
BF1	Y	259	0	0	6687.7	0	1732114
BF2	X	0	44	5091.9	0	224043.6	0
BF3	Y	292	0	0	6687.7	0	1952808
BF4	X	0	56	10165.7	0	569279.2	0
BF5	X	0	64.5	14605	0	942022.5	0
BF6	Y	62	0	0	6106.4	0	378596.8
BF7	X	0	39.67	7351.4	0	291630	0
BF8	Y	77.67	0	0	6106.4	0	474284.1
BF9	Y	86.67	0	0	4944.8	0	428565.8
BF10	X	0	56	6994.6	0	391697.6	0
BF11	Y	95.67	0	0	4951.9	0	473748.3
BF12	X	0	31.83	7001.7	0	222864.1	0
			Sum	51210.3	35484.9	2641537	5440118

Xr = Sum(Ry*X)/ Sum(Ry)	153.307961
Yr = Sum(Rx*Y)/ Sum(Rx)	51.5821436

Checking these values with the blue numbers in Figure 30, one will notice that the percent difference in the X direction is about 10% and in the Y direction about 2%. This difference is probably due to the fact that the RAM model is considering the column near gridline 5 as part of the lateral system when it does not add any rigidity to the system. Eliminating that from the RAM model will move the COR closer to the hand calculations performed in Table 1.

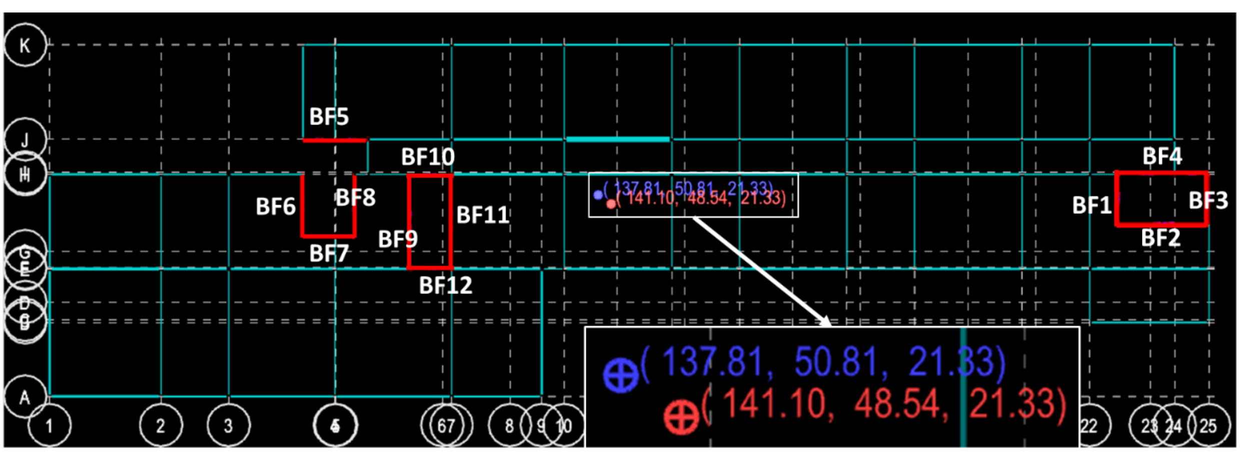


Figure 31: RAM COR and COM

4.2.2 Center of Mass

The center of mass or COM for the building is the average of each floor masses centroid. External loads as moments on the building act through the COM of the building. These loads include the seismic forces that have been calculated in the following section as well as the story weights which can be found in Appendix E.

Table 2: Center of Mass

ELEMENT LABEL	AREA (ft2)	HEIGHT (ft)	UNIT W (kcf)	W (k)	DIST. FROM REF. DATUM		W*X	W*Y
					X (FT)	Y (FT)		
Concrete FLOOR Level 1-6	12954	0.75	0.15	1457.33	141	48	205482.825	69951.6
Concrete FLOOR Level 7-8	5885	0.75	0.15	1324.13	141	48	186701.625	63558
Lower Roof - PH Roof	5885	0.75	0.15	662.063	141	48	93350.8125	31779
BF1				0.97028	259	41.42	251.30252	40.1889976
BF2				1.13518	276.75	44	314.161065	49.94792
BF3				0.97028	292	41.42	283.32176	40.1889976
BF4				1.58867	276.75	56	439.664423	88.96552
BF5				1.55898	72	64.5	112.24656	100.55421
BF6				1.0841	62	43.42	67.2142	47.071622
BF7				0.8988	67	39.67	60.2196	35.655396
BF8				1.08409	77.67	43.42	84.2012703	47.0711878
BF9				0.9481	86.67	41	82.171827	38.8721
BF10				0.9987	96	56	95.8752	55.9272
BF11				1.2874	95.67	41	123.165558	52.7834
BF12				1.11067	96	31.83	106.62432	35.3526261
			SUM (W)	1470.96		SUM	207502.993	70584.1792

Xcom = Sum(W*X)/ Sum(W)	**141.066**
Ycom = Sum(W*Y)/ Sum(W)	**47.9851**

These values vary slightly from the RAM model's coordinates seen in Figure 30 by approximately 1-2%. Fortunately the center of mas and center of rigidity are not too far apart, reducing a large amount of incidental torsion created by the loads.

The following tables, 3 and 4, are preliminary steps to determine the torsional shear and total shear that exists in each of the twelve braced frames.

Table 3: Torsional Rigidity

ELEMENT LABEL	Rx	Ry	dx	dy	Rx*dy	Ry*dx	Rx*dy^2	Ry*dx^2
BF1	0	6687.7	105.69	0	0	706823	0	74704124
BF2	5091.9	0	0	-7.58	-38596.6	0	292562.2	0
BF3	0	6687.7	138.69	0	0	927517.1	0	1.29E+08
BF4	10165.7	0	0	4.42	44932.39	0	198601.2	0
BF5	14605	0	0	12.92	188696.6	0	2437960	0
BF6	0	6106.4	-91.31	0	0	-557575	0	50912208
BF7	7351.4	0	0	-11.91	-87555.2	0	1042782	0
BF8	0	6106.4	-75.64	0	0	-461888	0	34937216
BF9	0	4944.8	-66.64	0	0	-329521	0	21959311
BF10	6994.6	0	0	4.42	30916.13	0	136649.3	0
BF11	0	4951.9	-57.64	0	0	-285428	0	16452042
BF12	7001.7	0	0	-19.75	-138284	0	2731101	0
						SUM	4108555	3.11E+08
							315258762.4	

J = SUM (Ridi^2)	315258762.4	(k/in)ft^2

Table 4: Direct Shear into Frames

V = applied shear (kips)	737.9	116.2	Vd= (Ri/SUM(Ri))*V		Applied Direction
ELEMENT LABEL	**Rx**	**Ry**	**DIRECT SHEAR (Vd)**		
BF1	0	6687.7	0.0	21.9	Y
BF2	5091.9	0	73.4	0.0	X
BF3	0	6687.7	0.0	21.9	Y
BF4	10165.7	0	146.5	0.0	X
BF5	14605	0	210.4	0.0	X
BF6	0	6106.4	0.0	20.0	Y
BF7	7351.4	0	105.9	0.0	X
BF8	0	6106.4	0.0	20.0	Y
BF9	0	4944.8	0.0	16.2	Y
BF10	6994.6	0	100.8	0.0	X
BF11	0	4951.9	0.0	16.2	Y
BF12	7001.7	0	100.9	0.0	X
Sum	51210.3	35484.9			

Table 5: Torsional Shear into Frames

ELEMENT LABEL	Direction	Ridi	Vt
BF1	Y	706823.0	0.9
BF2	X	-38596.6	1.1
BF3	Y	927517.1	1.2
BF4	X	44932.4	1.3
BF5	X	188696.6	5.4
BF6	Y	-557575.4	0.7
BF7	X	-87555.2	0.1
BF8	Y	-461888.1	13.2
BF9	Y	-329521.5	0.4
BF10	X	30916.1	0.0
BF11	Y	-285427.5	0.4
BF12	X	-138283.6	0.2

Table 5 provides the total shear due to torsion that is present in each of the listed braced frames. Load follows stiffness, so one will notice the stiffer elements receive more torsional shear.

Table 6: Total Shear

ELEMENT LABEL	Vd	Vt	Vi	Direction
BF1	21.90	0.94	22.84	Y
BF2	73.37	1.11	74.48	X
BF3	21.90	1.23	23.13	Y
BF4	146.48	1.29	147.77	X
BF5	210.45	5.40	215.85	X
BF6	20.00	0.74	20.74	Y
BF7	105.93	0.12	106.04	X
BF8	20.00	13.23	33.22	Y
BF9	16.19	0.44	16.63	Y
BF10	100.79	0.04	100.83	X
BF11	16.22	0.38	16.59	Y
BF12	100.89	0.18	101.07	X

Table 6 provides the total shear in each of the braced frames. This is a combination of the direct shear and torsional shear calculated in the previous tables.

4.2.3 Wind Loads

This section provides an overview of the wind loads considered for the proposed design. The loads were calculated using ASCE 7-10 section 26. The following tables show the total windward and leeward pressures experienced on the building in both the E-W and N-S directions. Hand calculations in Appendix A, show the procedural steps taken to determine the lateral loads.

Table 7: North-South Wind Forces

Wind Force Determination N-S						
Building Level	Height above ground level z (ft)	K_Z	q_z	$p_{z(W)}$	$p_{h(L)}$	Total (psf)
Level 1	0.0	0.575	16.54	10.66	-11.18	21.84
Level 2	12.0	0.575	16.54	10.66	-11.18	21.84
Level 3	22.7	0.647	18.61	11.99	-11.18	23.17
Level 4	33.3	0.722	20.78	13.39	-11.18	24.57
Level 5	44.0	0.782	22.49	14.49	-11.18	25.67
Level 6	54.7	0.832	23.93	15.42	-11.18	26.60
Level 7	65.3	0.875	25.18	16.23	-11.18	27.41
Level 8	76.0	0.914	26.29	16.94	-11.18	28.12
Lower Roof	87.9	0.953	27.41	17.66	-11.18	28.84
PH Roof	95.9	0.977	28.10	18.11	-11.18	29.29

Table 8: East-West Wind Forces

Wind Force Determination E-W						
Building Level	Height above ground level z (ft)	K_Z	q_z	$p_{z(W)}$	$p_{h(L)}$	Total (psf)
Level 1	0.0	0.575	16.54	11.18	-4.69	15.87
Level 2	12.0	0.575	16.54	11.18	-4.69	15.87
Level 3	22.7	0.647	18.61	12.58	-4.69	17.27
Level 4	33.3	0.722	20.78	14.04	-4.69	18.73
Level 5	44.0	0.782	22.49	15.20	-4.69	19.89
Level 6	54.7	0.832	23.93	16.18	-4.69	20.87
Level 7	65.3	0.875	25.18	17.02	-4.69	21.71
Level 8	76.0	0.914	26.29	17.77	-4.69	22.46
Lower Roof	87.9	0.953	27.41	18.53	-4.69	23.22
PH Roof	95.9	0.977	28.10	19.00	-4.69	23.69

Table 9: North-South Base Shear

Base Shear Determination N-S				
Building Level	**Height above ground level z (ft)**	Tributary Height (ft)	Total Pressure (psf)	Total Lateral Story Force (kip)
Level 1	0.0	6.00	21.84	39.31
Level 2	12.0	11.34	21.84	74.26
Level 3	22.7	10.67	23.17	74.14
Level 4	33.3	10.67	24.57	78.60
Level 5	44.0	10.67	25.67	82.18
Level 6	54.7	10.67	26.60	85.11
Level 7	65.3	10.67	27.41	87.68
Level 8	76.0	11.30	28.12	95.29
Lower Roof	87.9	9.96	28.84	86.18
PH Roof	95.9	4.00	29.29	35.15
			Total Base Shear (kips) =	**737.90**

Table 10: East West Base Shear

Base Shear Determination E-W				
Building Level	**Height above ground level z (ft)**	Tributary Height (ft)	Total Pressure (psf)	Total Lateral Story Force (kip)
Level 1	0.0	6.00	15.87	5.81
Level 2	12.0	11.34	15.87	10.97
Level 3	22.7	10.67	17.27	11.24
Level 4	33.3	10.67	18.73	12.19
Level 5	44.0	10.67	19.89	12.95
Level 6	54.7	10.67	20.87	13.58
Level 7	65.3	10.67	21.71	14.13
Level 8	76.0	11.30	22.46	15.48
Lower Roof	87.9	9.96	23.22	14.11
PH Roof	95.9	4.00	23.69	5.78
			Total Base Shear (kips) =	**116.22**

The largest base shear is found in the North-South direction of the building. This makes sense due to the large surface area that the wind loads will be acting on in that direction. It is important to design a buildings lateral elements to resist all directions because wind is multi directional.

The story forces and building loads that were displayed in the above tables were applied to the building. Results of displacement and stresses were compared to serviceability and strength criteria from ASCE 7-10. The floor diaphragms are rigid and distribute the lateral loads based on location of the lateral force resisting elements.

Using RAM frame analysis software, story drifts were calculated for both wind and seismic loading. It was determined that wind is the controlling factor for drift. Wind drift was found to be within the drift limits of h/400, as set forth in ASCE 7-10. The wind drift limit of the main roof was as follows;

$$\Delta_{max} = \frac{96' \times 12"/1'}{400} = 2.88"$$

This is the maximum allowable drift at the top of the building. 2.4 inches is the worst case scenario displacement from the RAM output found in Figure 32. The braced frames are a sufficient design for the applied loads.

Level: Roof, Diaph: 1

Center of Mass (ft): (141.09, 48.50)

LdC	Disp X in	Disp Y in
D	0.02980	-0.02435
Lp	0.01083	-0.00667
W1	0.84420	0.02105
W2	0.03478	$\boxed{2.47560}$
W3	0.63352	-0.00380
W4	0.63279	0.03538
W5	0.02137	2.11153
W6	0.03080	1.60187

Figure 32: RAM drift output

4.2.4 Seismic Loads

Some of the advantages of buckling restrained braced frames directly relate to seismic loading on West Village Housing's north building. The seismic story forces are given by taking a factor of the total story weight. This factor, C_{vx}, is based on multiple factors such as the geographical location of the site, the structural type (steel), period of vibration and the importance factor. Buckling Restrained Braced frames has a very high response modification coefficient, R, of 8. This helps reduce the overall story forces due to seismic loading. All of the required values that are used in seismic force calculations can be found in Table 7.

Table 11: Seismic Force Factors

Name	Value	Reference
site class	C	per geotechnical report
S_s	0.175	USGS app
S_1	0.051	USGS app
F_a	1.2	table 11.4.1
F_v	1.7	table 11.4.2
S_{MS}	0.21	
S_{M1}	0.0867	
S_{DS}	0.1400	
S_{D1}	0.0578	
Occupancy Cat.	II	table 1-1
Importance Factor	1.00	table 11.5-1
Design Cat. Based on S_{DS}	A	table 11.6-1
Design Cat. Based on S_{D1}	B	table 11.6-2
design method	equivalent lateral force	section 12.8
R	8	table 12.2-1
C_s	0.018	eq. 12.8-2
W=	15139.6	kips
Ta=	0.000	eq 12.8-7
k=	0.750	12.8.3
Seismic Base Shear	**264.943**	**kips**

Table 8 displays the calculation process that these factors are applied to. The total seismic base shear is drastically decreased in comparison to the base shear of the existing system. The new lateral system decreased the seismic forces by almost half. This is due to the extreme stiffness of the buckling restrained braced frames which will be talked about in the next section. The overturning moment also decreased with the proposed system. This will help for the reduction of foundations which has already been addressed in this report.

Table 12: Seismic Story Forces

LEVEL	h_x (ft)	W_x (k)	$W_x h_x^k$	C_{vx}	F_x (k)	V_x (k)	OVERTURNING MOMENT (ft-k)
Level 1	0.00	**1767**	0	0.000	0.00	**264.9**	0.00
Level 2	12.00	**1936**	25287	0.040	10.66	**264.9**	127.87
Level 3	21.3	**1888**	44638	0.071	18.81	**254.3**	400.67
Level 4	30.67	**1889**	65097	0.104	27.43	**235.5**	841.34
Level 5	40	**1888**	85614	0.136	36.08	**208.0**	1443.11
Level 6	49.3	**1888**	106314	0.169	44.80	**172.0**	2208.68
Level 7	58.67	**1460**	98379	0.156	41.46	**127.2**	2432.28
Level 8	68	**1481**	116279	0.185	49.00	**85.7**	3332.01
Lower Roof	78.83	**844**	77167	0.123	32.52	**36.7**	2563.41
PH Roof	86.67	**99**	9944	0.016	4.19	**4.2**	363.19
TOTAL		15140	628717	1.0000	264.94		**13712.55**

Though wind loads are the controlling factor for this design, the seismic drift limit of the main roof was still checked and can be seen below. Per ASCE 7-10, story drift is limited to two percent of the total building height, which limits the total drift of the main roof level to the equation shown below;

$$\Delta_{max} = (96' \times 12"/1') \times 0.02 = 23.04"$$

This is the maximum allowable drift at the top of the building. This is well above the RAM outputs for seismic drifts. Therefore, the braced frames are a sufficient design for the applied loads.

4.2.5 Buckling Restrained Braced Frames

There are few manufacturers of buckling restrained braced frames, BRB frames, due to its unique design as well as the recent interest for there use in the industry. CoreBrace is one of the leading manufacturers for BRB's and is where the proposed frames have been designed from.

In general, the frames provide lateral resistance to buckling and are highly effective for energy dissipation or seismic disturbance. They consist of a steel core surrounded by a steel section that is filled with a specialized mortar. The mortar helps the steel from buckling when in compression, while the casing prevents strength loss from axial loads. The maximum tension and compression forces in a buckling restrained brace are much closer in value than in a standard lateral brace. Due to the smaller amount of imbalance of force in a chevron brace, smaller beam sizes can be selected compared to a standard braced frame.

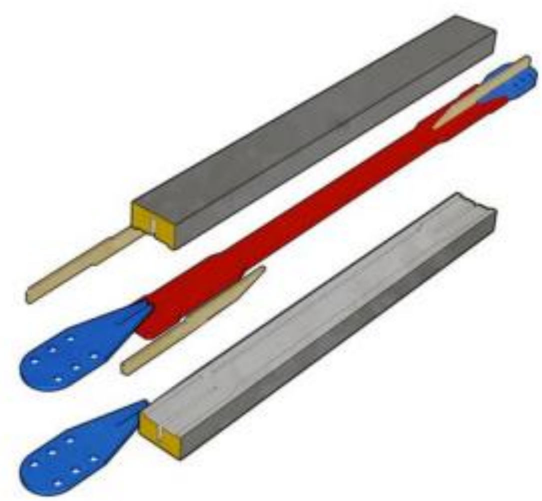


Figure 34: Deconstructed view of bolted lug connection. Courtesy of CoreBrace.com

A design guide from CoreBrace website for bolt-lug connection braces is provided in Appendix F. This type of connection shown in Figure 34 is the standard BRB connection. It has plenty of field tolerance allowing for a fast and efficient frame erection. The guide accounts for the brace force, elongation and stiffness, frame deflection and alternate elastic forms.

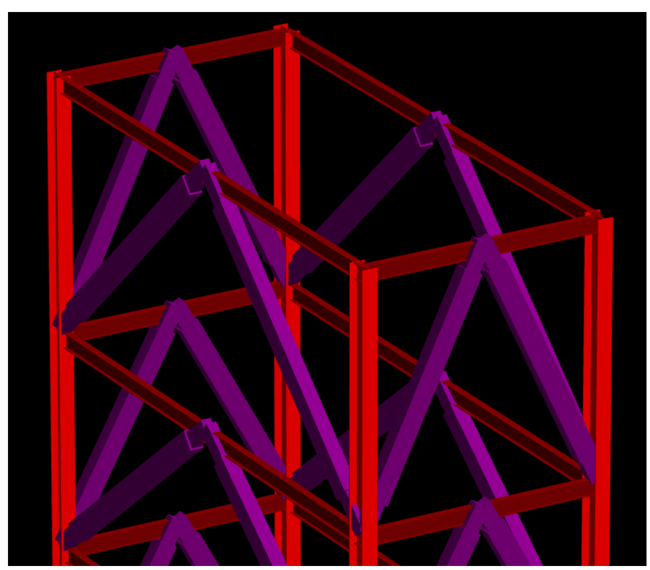


Figure 33: CoreBrace as applied in RAM Structural

RAM Structural designs buckling restrained braced frames. In RAM Frame, one can input a BRBF by selecting StarSeismic or CoreBrace and specifying any stiffness factors that may be applied in the structural model.

5 Construction Management Breadth

5.1 Overview and Design Objectives

West Village Housing Phases III & IV is a project that is driven by both cost and schedule which is typical for any University construction project. One of the main reasons for changing the structural system of the building from concrete to steel was to shorten the overall construction time in hopes that this would also save the project money. A detailed schedule analysis will help determine the aspects of both existing and proposed systems that help fast track the project schedule. A cost analysis will be performed to provide a basis for investigating the economic feasibility of the new system.

5.2 Critical Path Schedule Analysis

The schedule analysis includes the determination of the critical path for both the existing and redesigned systems as well as a comparison between the two. Only the main structural elements were included in the analysis because theoretically those are the only aspects that will directly affect the schedule if the structural system is changed such as the gravity system and lateral system structures. They span from beginning of the lower level (LL) to the completion of the mechanical penthouse. The schedule analysis is based on the total time required for the items within the scope of the analysis as well as an approximate schedule using *RS Means Building Construction Cost Data 2014.* This was also used to help determine the total labor hours for each task within the schedule. The labor hours were used to calculate exactly how long each task would take and then applied to the schedule.

A critical path schedule is one that identifies the sequence of activities for the construction project. It is also referred to as the longest path schedule because it is the longest duration path through the network of activities. This means that the activities located on the path cannot be delayed without delaying the project. For example, in the existing concrete system the columns and walls on level 1 need to be prepped and poured before the slab on level 2 can be placed. Due to the impact of these activities on the entire project, the critical path analysis is a crucial aspect of project planning.

Both the existing and proposed longest path also known as critical path schedules can be both in Appendix G for comparison. It was assumed for both schedules that the lateral system, shear walls and braced frames, will be constructed within the task for each floor. The existing schedule used crew sizes and labor time values from RS Means to determine the time required to construct the building. The total time for construction of the concrete structural system from the lower level to the roof is known to be approximately 7 months from February 24, 2015 to August 27, 2015. The project management team allocated approximately 8 work days for each floor. Two days for the prep and pour of the slab on deck, two for the columns and walls and four for the erection of the framing.

The proposed structural system was partially selected in order to directly shorten the project schedule. Precast elements help advance tasks on a project schedules because they are manufactured off site and simply need to be installed once they arrive on site. In addition, theoretically a steel system should be a quicker system to construct because unlike concrete, formwork is not required and one does not need to account for the time of concrete pouring and setting. In addition, multiple construction management strategies were utilized for the goal of shortening the project schedule.

Essential tasks for multiple areas on a project can be completed simultaneously to accelerate a projects schedule. This is called sequencing and will be used for the proposed structural steel system. The strategy of sequencing is to expedite the time it takes for each level to be constructed. The current concrete system has each floor being constructed after one another. Each floor takes 8 work days to be completed.

The proposed system can take advantage sequencing by dividing the floor plan in half. Beams and floor planks would be constructed on one half of the building as columns are being set on the other half for the floor above. Figure 33 shows how the building could be divided for construction management. Planks can be installed on the East half of the building for Level 1 while simultaneously the columns for level 2 are being installed on the West half. This strategy will be effective in eliminate cost and time from the existing schedule. This approach assumes that there are two cranes on site during construction. This is required for the lifting and installation of the steel members on the East and West sides of the building at the same time. Zoning is a geometrical way to divide the building up into areas making it easy for workers to understand creating an efficient work environment. Figure 33 also shows possible zones that can be used during the construction sequencing.

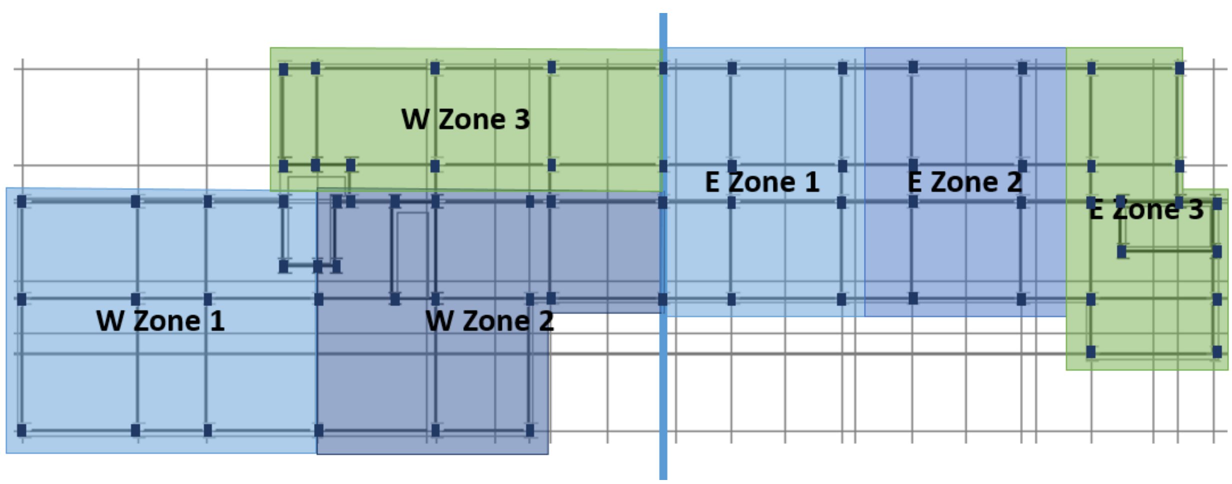


Figure 35

The proposed system will use these strategies to accelerate the project and shorten the schedule. The proposed schedule in Appendix G shows that the tasks for each floor were reduced from 8 work days to 7. This is due to the smaller number of required labor hours to install steel beams and columns. If sequencing and zoning plans are used then the schedule can be reduced even more. Each floor would take approximately half the time than originally schedule since half of one floor is being constructed the same time as half of another. This would result in a schedule reduction of more than 50% for the construction schedule. The proposed system would take about 3 to 4 months as compared to the 7 months for the concrete system.

5.3 Cost Analysis

The cost analysis includes a unit cost estimate for both the existing and redesigned systems as well as a comparison between the two. The gravity system and lateral system structures were included. The scope of the breadth will focus on the structural costs and will also account for differences in general conditions costs due to schedule changes. Due to the request of the owner, cost figures will not be given. Instead, a general square footage will be analyzed as both the existing concrete system and proposed steel system. This will give a fair representation of both systems which can then draw conclusions towards the cheaper system.

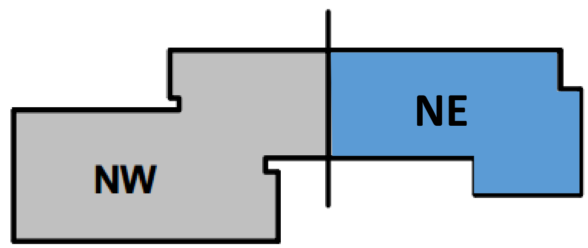


Figure 36

Unit cost values for the elements of the existing gravity and lateral system were located in *RSMeans Concrete and Masonry Cost Data 2012, 30th Edition.* The area being analyzed in the gravity system analysis is the North East half of the building shown in figure 34. This was to simplify the calculation without compromising the main materials that are included in both the concrete system and steel system. The following tables include the approximate cost estimates for the systems.

Though the total cost numbers do not reflect the estimate of the entire building, it can be applied through a ratio to prove which system is more cost efficient. The gravity system will generally cost more than the lateral system because it covers much more square footage of the building than the lateral system. Through a simply cost estimate of the gravity systems which was demonstrated in Table 1 and Table 2, one will find that the proposed steel gravity system will be less than the existing concrete system. Figure 35, is a visual representation of the system costs adjacent to each other. The steel gravity system is proposing a 23.5 % price savings in relation to the existing system.

Table 3

Existing Gravity System				
Building Element	Quantity	Unit	Unit Cost	Total Cost
Prestressing Steel - 200' span, 300 kip	232.05	Lb	4.12	956.05
Reinforcing in Place - Elevated slabs, #4 - #7	10.974938	Ton	2025	22224.25
Normal Weight Concrete - 5000 psi	202.94267	C.Y.	120	24353.12
Placing Concrete - 6"-10" thick, pumped	202.94267	C.Y.	28.5	5783.87
Concrete in Place - Columns, square (4000psi), 24"x24" average reinforcing	1.3836048	C.Y.	1275	1764.10
			Total $	**55081.38**

Table 4

Proposed Gravity System				
Building Element	Quantity	Unit	Unit Cost	Total Cost
Precast Slab Planks - Hollow, 8" thick	1056	S.F.	10.35	10929.6
Finishing Floors - Integral topping and finish, 2" thick	1056	S.F.	6.55	6916.8
Structural Steel Members - Beam, W 14 x 82	88	L.F.	201	17688
Structural Steel Members - Beams, W 8 x 10	72	L.F.	27	1944
Columns Structural - W Shape, 12 x 40	9.33	L.F.	86.5	807.045
			Total $	**42113.99**

The lateral system cost was a more unique analysis due to the material and manufacturing. One lateral element in both systems was priced out for this analysis. The existing concrete shear wall was fairly simply. The calculation included the cost of concrete, reinforcing, labor and formwork. The buckling restrained braced frame costs are specific to the manufacturer. A representative from StarSeismic was contacted for the estimate of their braced frames. Through a simply cost estimate of the lateral systems which was demonstrated in Table 1 and Table 2, one will find that the proposed steel lateral system will be slightly more than the existing shear walls. This is due to the personalization from the manufacturer of the braces for the project. The added cost for these braces accounts for the convenience of their availability and production. The braces are assembled off site and then transported to the site which was accounted for in the estimate. Figure 35 shows the lateral costs comparison with a visual representation. The steel lateral system is proposing a 21.5 % price increase in relation to the existing system.

Table 5

Existing Lateral System				
Building Element	Quantity	Unit	Unit Cost	Total Cost
Placing Concrete - walls, 8" thick	129.30859	C.Y.	96.5	12478.28
Reinforcing in Place - Columns, #3 - #7	1.37676	Ton	2750	3786.09
Forms In Place, Walls - job-build plywood, over 8' 3 use	210.0219	SFCA	13.95	2929.81
				0.00
			Total $	**19194.17**

Table 6

Proposed Lateral System				
Building Element	Quantity	Unit	Unit Cost	Total Cost
Columns Structural - W Shape, 12 x 53	77.3	L.F.	146	11285.8
Structural Steel Members - Beams, W 8 x 10	180.664	L.F.	27	4877.928
Braces - Star Seismic BRBF bolted lug connection				11000
				0
			Total $	24447.36

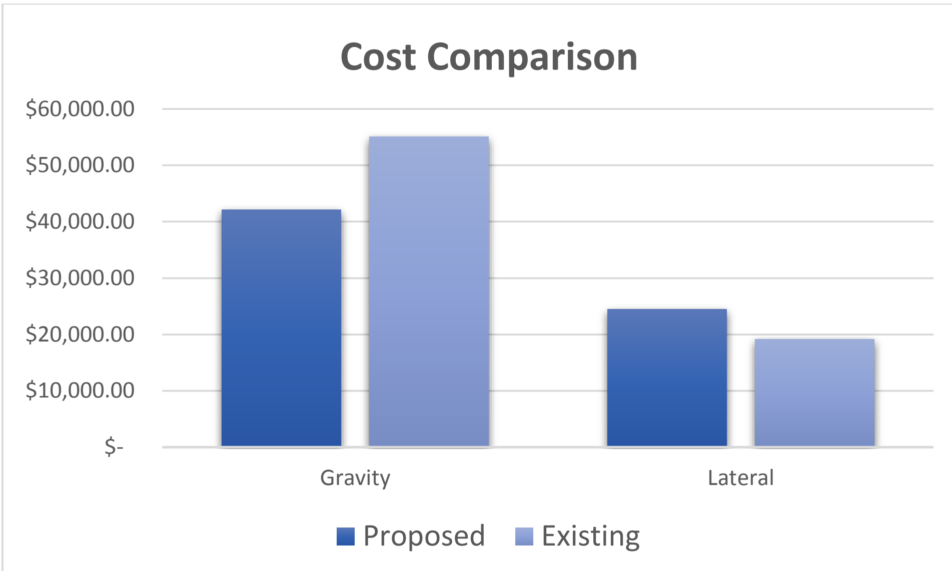


Figure 37

5.4 Conclusion

The structural redesign in steel and hollow core planks for the gravity system is expected to save roughly 23.5% while the redesigned lateral system is expected to increase by about 21.5% in cost as seen in Figure 36. In general, the structural gravity system for this project and scenario is a larger portion of the system. Since specific cost values of the entire building are not to be released upon request of the owner, it could holistically be assumed that the savings of the proposed gravity system outweigh the price increase of the lateral system.

	Gravity	Lateral
Proposed	$42,113.99	$24,447.36
Existing	$55,081.38	$19,194.17

% Savings	**23.5**	**-**
% Increase	**-**	**21.5**

Figure 38

The proposed schedule will speed up the production of the building. The structure of a building is the first main critical path element to be constructed. Mechanical and Electrical contractors usually will not install there work until the structure is built hence why it is important. The existing schedule has the concrete system for the entirety of the building lasting about 7 months. The proposed steel system as well as the strategy of sequencing and zoning will reduce the schedule by approximately 3 to 4 months. This reduction in the construction time of the structural system is significant when considering the total two year timeline for the project.

6 Acoustics Breadth

6.1 Overview and Design Objectives

The acoustics breadth analyzes the acoustical performance of the typical shared apartment space and improves the condition with the proposed design. The concept of using concrete hollow core planks supported by steel beams led to a change in both floor to floor heights as well as the floor cross section details. By changing the systems and layout in the space, the acoustics of the space differed from its original performance. Since this space is going to be used for upper class living, acoustical performance is important to the occupants and should be considered during design.

6.2 Background of Analysis

Sound transmission loss (TL) is the energy that does not get transmitted into an adjacent space. The TL of a single material could be found by mass, stiffness, and damping. The transmission loss value depends on the surface area of each wall element as well as the transmission coefficient of each element. Sound Transmission Class or STC rating is a single number rating that represents the overall sound transmission of a common partition type, e.g. walls, ceilings, etc. STC is obtained based on transmission loss measurements and is not just an average of all the TL values. Finding the STC is an effective way to compare the acoustical performance of different partitions, and helps in finding appropriate partition for the apartments in West Village Housings north building. All the information needed to test the partitions were taking from the wall section information on the partition detail drawings.

This analysis determines whether the existing partitions and floor sections in the apartment areas control the noise properly or not. If they can be improved, different materials that work effectively with the space will be proposed. The following list of criteria was set at the beginning of this project in the project narrative. These values will be checked and ways to improve them will be investigated.

F. ACOUSTICAL CRITERIA

1. Systems will be designed to meet the following noise criteria:

Area	NC Level
Apartments	30
Bedrooms	30
Lobbies and common areas	40
Multipurpose rooms	35
Offices	35

2. Sound attenuators and acoustically lined ductwork will be used to achieve these levels.
3. Vibration transmission from equipment will be minimized with the use of vibration isolation devices.
4. Floor/ Ceiling Assembly: STC 55, IIC-55
5. Apartment demising walls: STC 55
6. Bedroom walls: STC 50
7. Corridor walls: STC 50

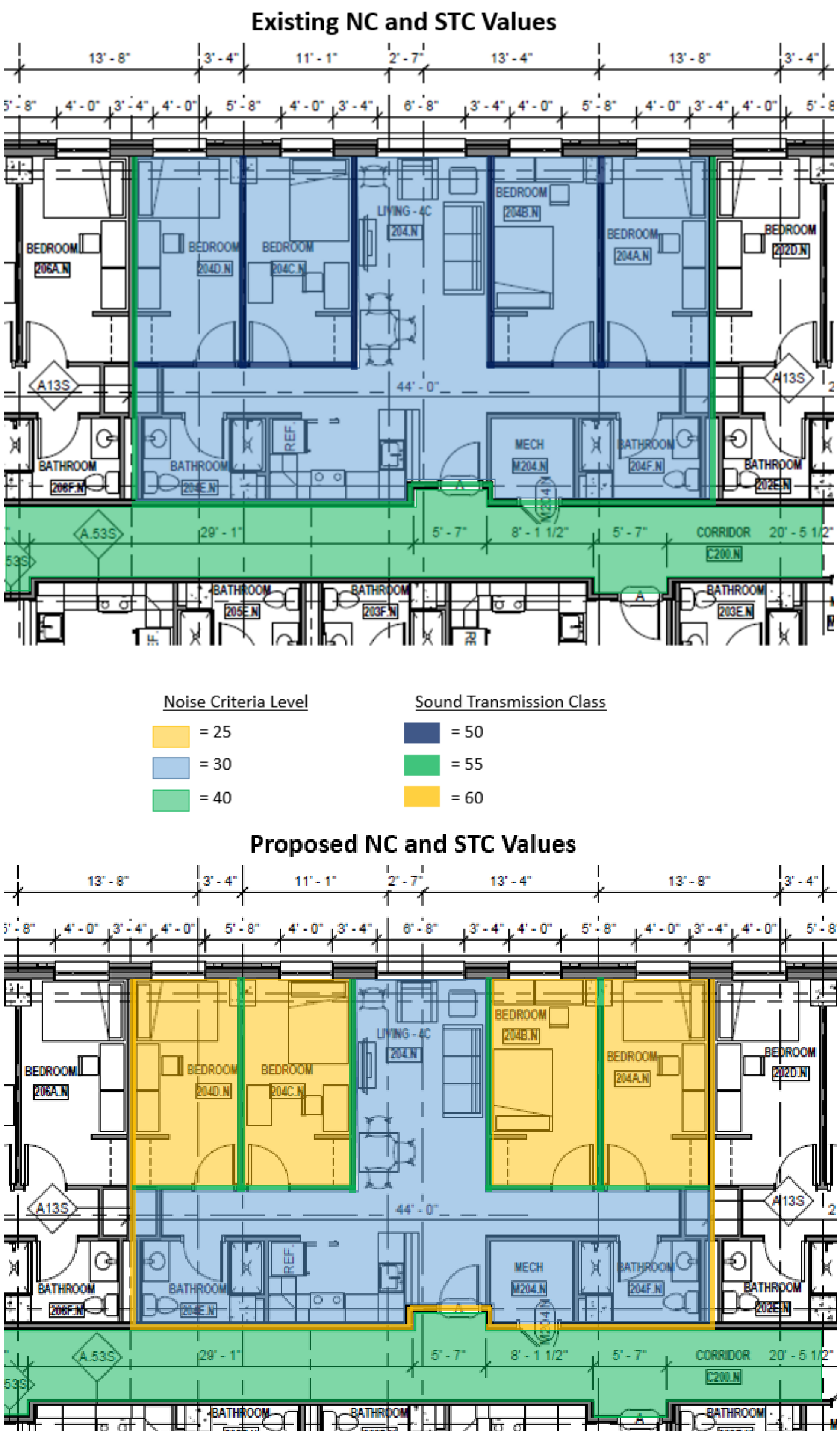


Figure 39: NC and STC values for both systems

6.3 STC Analysis

Noise criterion, NC, define the limits of octave band spectra that must not be exceeded to meet the occupants acceptance in certain spaces. Spaces with higher NC require less acoustical performance design such as a sports venue where the recommended NC level is from 45-55. Rooms with lower noise criteria levels limit the sound pressures within the space resulting in a quieter room such as a sound broadcasting room with NC level of about 15-20. From Figure 37 above, one can tell that the proposed system is trying to decrease the amount of noise that is heard in the bedrooms by increasing the STC of the bedroom walls and demising walls as well as decreasing the NC level of the individual bedrooms.

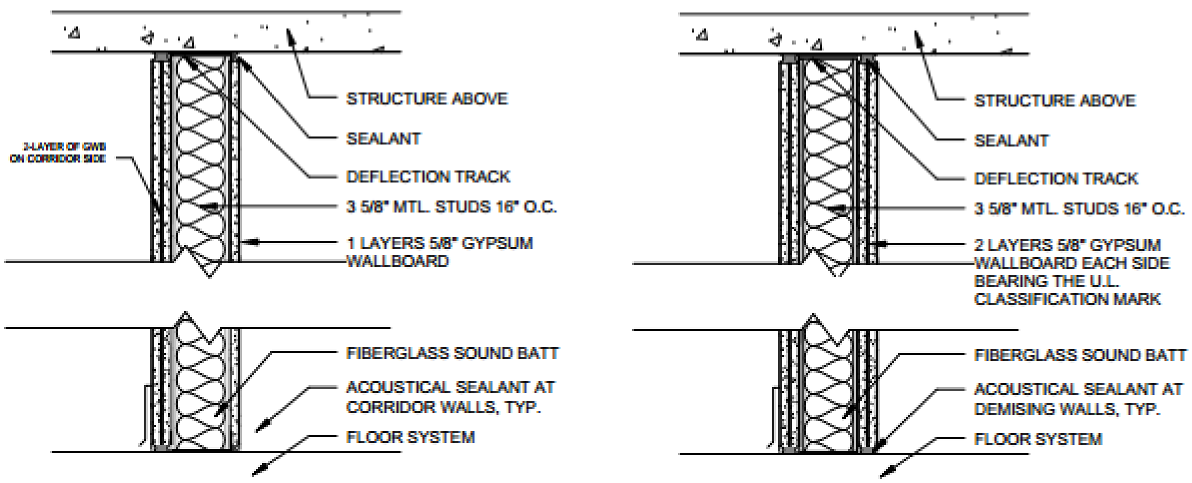


Figure 40: A13S, A23S, A.53S

The above partitions are both details from the existing architectural drawings on the project. The sound transmission loss STC values were provided and then checked with *Architectural Acoustics, Principles and Design.* The double layers of 5/8" gypsum wallboard was able to provide enough acoustical insulation for the partition to help meet the projects goal of an STC value of 55 between walls that separate tenant apartments also known as demising walls. For the proposed goal of increasing the values on the bedroom walls and demising walls by at least 5, some more layers had to be added to the assemblies in order to meet the target STC values.

Table 7

Existing Apartment Partitions				
Partition	**Label**	**Room #'s**	**Description**	**STC Value**
Apartment demising wall	A12/A23	Apartment-Apartment	Two layers 1/2" GB each side of 3-5/8" studs 16" O.C. plus 1-1/2" FG	**55**
Corridor wall	A12/A23	Apartment-Apartment	Two layers 1/2" GB each side of 3-5/8" studs 16" O.C. plus 1-1/2" FG	**55**
Bedroom wall	A.53	Bedroom-Bedroom-Living Area	One layer 5/8" GB 3-5/8" studs 16" O.C. plus 1-1/2" FG	**50**
Exterior wall	Type 1	Apartment-Outside	4" masonry, 2 1/2" stone wool/acoustical insulation, 5/8" GB	**60**
Floor/Ceiling Assembly	S5.00	Apartment-Apartment	8" PT slab, ceiling tiles, carpeting	**58**

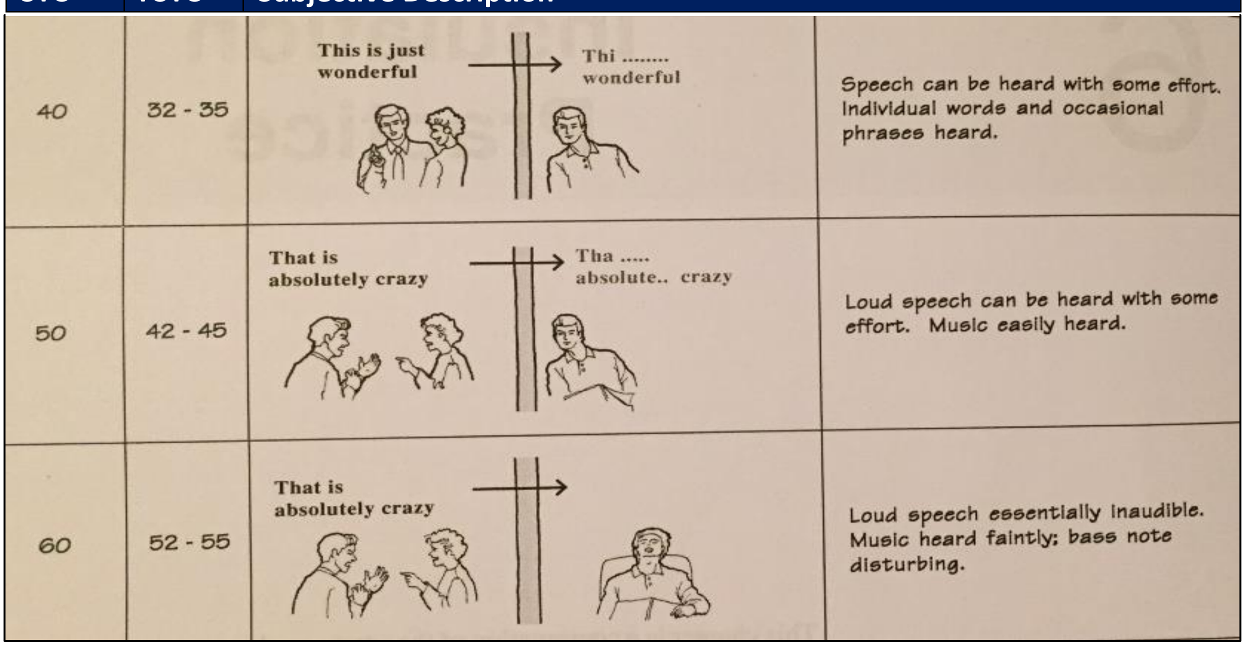
Table 8

Proposed Apartment Partitions				
Partition	**Label**	**Room #'s**	**Description**	**STC Value**
Apartment demising wall	NA	Apartment-Apartment	Three layers 1/2" GB each side of 3-5/8" studs 16" O.C. plus 3" FG	**61**
Corridor wall	NA	Apartment-Apartment	Three layers 1/2" GB each side of 3-5/8" studs 16" O.C. plus 3" FG	**61**
Bedroom wall	NA	Bedroom-Bedroom-Living Area	One layer 5/8" GB 3-5/8" studs 16" O.C. plus 1-1/2" FG	**50**
Exterior wall	Type 1	Apartment-Outside	4" masonry, 2 1/2" stone wool/acoustical insulation, 5/8" GB	**60**
Floor/Ceiling Assembly	NA	Apartment-Apartment	8" hollow core plank 2" topping, acousitcal ceiling tiles	**59**

Table 14 shows that the apartment demising walls added two more layers of gypsum board to add acoustical absorption to the wall, in turn, increasing the STC value of the assembly. A benefit about using hollow core planks in the proposed system design is that they provide assemblies with good acoustical barriers. Air gaps are known to be good insulators not only for thermal insulation but for acoustical reasons as well. Acoustical ceiling tiles are known to absorb a lot of sound and therefore helped the floor/ceiling assembly increase its STC to 59. Table ## represents the subjective description that is associated with different STC values, proving that our partitions will be good to block out loud speech and music.

Table 9

STC	FSTC	Subjective Description



STC	FSTC	Subjective Description
40	32 - 35	Speech can be heard with some effort. Individual words and occasional phrases heard.
50	42 - 45	Loud speech can be heard with some effort. Music easily heard.
60	52 - 55	Loud speech essentially inaudible. Music heard faintly; bass note disturbing.

6.4 STC Calculation

The given STC values that were taken from *Architectural Acoustics Principles and Design*, while giving a basic understanding it does not give the reader what frequencies are best and worst for the use of the analyzed space.

In order to eliminate the common complaint of loud apartment dwellers and hearing music from neighbors, it is important to make sure there are higher TL values in the mid frequency ranges. This is because fewer discrepancies in the middle ranges allows for the sound to be less irritating to the occupants. A wall with minor variances in the 500-1000 range of frequencies must be selected so to counter the common noises of music and speech from neighboring units because those are the frequencies where they are usually heard at.

In order to confirm that the proper proposed demising wall partition was selected, a graphical procedure STC calculation can be performed. First, the transmission loss values for the partition are graphed in one-third octave bands. Then a standard STC contour line is placed above all of the data points. TL data points that fall below the STC contour are call deficiencies. This line is then adjusted by lowering it along the ordinate until the maximum deficiency between any data point and the contour is less than 8 dB and the sum of all the deficiencies is less than 32 dB.

Table 10

1/3 Octave-Band Frequency (HZ)	Adjustment for Contour Level	Contour Level dB	TL dB	Deficiency dB	Max Deficieny ≤x dB
125	3	45	41	4	ok
160	3	48	45	3	ok
200	3	51	50	1	ok
250	3	54	52	2	ok
315	3	57	57	0	ok
400	1	60	60	0	ok
500	1	61	62	0	ok
630	1	62	62	0	ok
800	1	63	64	0	ok
1000	1	64	64	0	ok
1250	1	65	66	0	ok
1600	0	65	66	0	ok
2000	0	65	62	3	ok
2500	0	65	59	6	ok
3150	0	65	63	2	ok
4000	0	65	68	0	ok
			Total	**21**	**Passes**

Wall is STC:	61

The below graphical representation of the proposed demising wall shows the TL values of the partition at the varied frequencies. The wall meets the goal of having high TL values and few discrepancies from 500 HZ to 1000 HZ. This means that it will sufficiently block the unwanted sounds of speech and loud music from neighboring apartments.

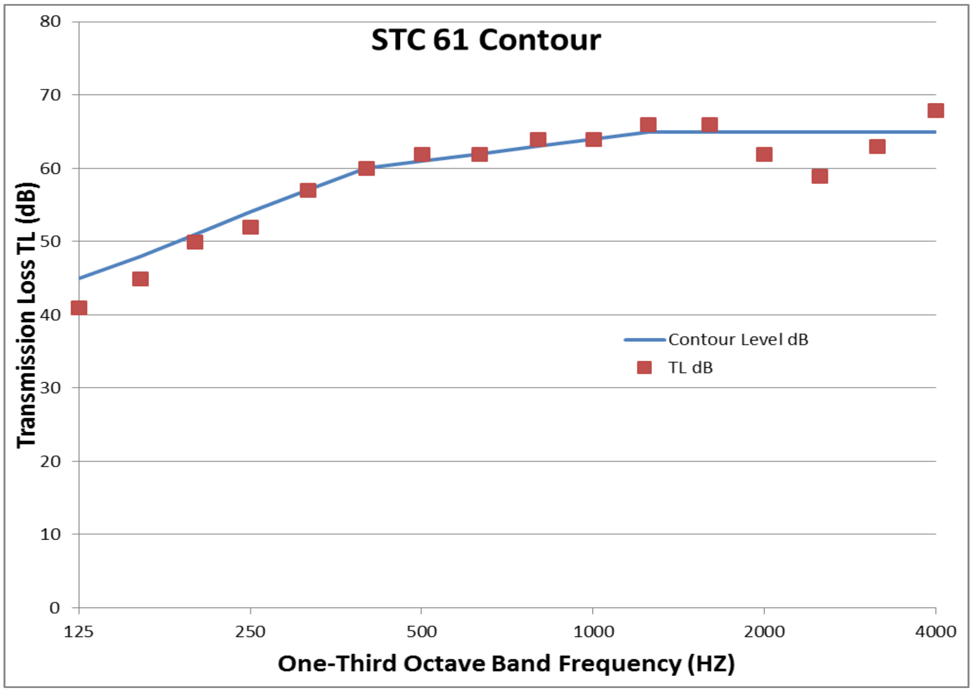


Figure 41

6.5 Conclusion

This acoustical breadth determined that the acoustical performance of the residence halls apartment units meets the original requirements. In addition, it also set new goals to increase the STC of the assemblies for the apartments. The values increased by 5 and 6 by adding more layers of gypsum board to the partitions. Hollow core planks also provided good acoustical absorption. Acoustical ceiling tiles were added to the floor/ceiling assembly to further the performance of the space helping increase the overall satisfaction of the occupants.

Appendix A Design Loads

1) RISK CATEGORY: (TABLE 1.5-1)
CATEGORY II

2) BASIC WIND SPEED, V: (FIGURE 26.5-1A)
$V = 115mph$

3) WIND LOAD PARAMETERS:
- WIND DIRECTIONALITY FACTOR, Kd (TABLE 26.6-1)
  $K_d = 0.85$
- EXPOSURE CATEGORY (SECTION 26.7)
  CATEGORY B
  * USED BY ENGINEER
- TOPOGRAPHIC FACTOR (TABLE 26.8-1)
  $K_{zt} = 1.0$
  * BUILDING IS NOT LOCATED ON A RIDGE, ESCARPMENT, HILL
- GUST EFFECT FACTOR (SECTION 26.9)
  * MAY VARY
  $G = 0.85$ → ENCLOSED BUILDING

INTERNAL PRESSURE COEFFICIENTS
$GC_{pi} = \pm 0.18$

4) VELOCITY PRESSURE EXPOSURE COEFFICIENT, Kz:
- Kz VARIES W/HEIGHT (TABLE 27.3-1)
  * SEE SPREADSHEET

5) VELOCITY PRESSURE, qz:
$q_z = 0.00256\, K_z\, K_{zt}\, K_d\, V^2$
K_z = VARIES W/ HEIGHT
$K_{zt} = 1.0$
$K_d = 0.85$
$V = 115mph$

$q_z = 0.00256\, K_z (1.0)(0.85)(115)^2 = 28.78 K_z$

* SEE SPREADSHEET FOR VALUES OF q_z

6) EXTERNAL PRESSURE COEFFICIENT, Cp:

NORTH - SOUTH: $\frac{L}{B} = \frac{300'}{61'} = 4.9$

EAST - WEST: $\frac{B}{L} = \frac{61'}{300'} = 0.69$

WALLS: - WINDWARD Cp=0.8

- LEEWARD Cp= -0.2 in N/S
  Cp= -0.5 in E/W
- SIDE WALL Cp= -0.7

ROOFS: - FLAT ROOF, $\theta = 0$

$h/L < 0.5$ - HORIZONTAL DISTANCE FROM WINDWARD EDGE = 300' > 2h

Cp= -0.3, -0.18

7) WIND PRESSURE

$P = q\,GC_p - q_i(GC_{pi})$ (27.4-1)

* SEE SPREADSHEET FOR P VALUES

SEISMIC LOAD CALCULATIONS

1) FIND MAPPED ACCELERATION PARAMETERS:

$S_s = 0.175 \quad S_1 = 0.051$

2) SITE CLASSIFICATION: (VERIFIED BY GEOTECHNICAL REPORT)

SITE CLASSIFICATION C

3) MAX CONSIDERED SPECTRAL RESPONSE ACCELERATION PARAMETERS:

$F_a = 1.2 \qquad F_v = 1.7$

$S_{ms} = F_a S_s = 1.2(0.175) = 0.21$

$S_{m1} = F_v S_1 = 1.7(0.051) = 0.087$

4) DESIGN SPECTRAL PARAMETERS (11.4.4)

$S_{DS} = \frac{2}{3} S_{ms} = 0.14$

$S_{D1} = \frac{2}{3} S_{m1} = 0.058$

5) IMPORTANCE FACTOR: $I_e = 1.0$ (1.5-2)

6) RISK CATEGORY: II (SECTION 11.6)

7) SEISMIC DESIGN CATEGORY: A (TABLE 11.6-1)

BASIC SEISMIC FORCE RESISTING SYSTEM

—

$R = 8$

$\Omega = 2\frac{1}{2}$

$C_d = 5$

8) ANALYSIS PROCEDURE SELECTION:

- BUILDINGS WITH SEISMIC DESIGN CATEGORY A ARE EXEMPT FROM SEISMIC DESIGN CRITERIA AND MUST ONLY COMPLY WITH SECTION 1.4 (SECTION 11.7)

9) LATERAL FORCES (SECTION 1.4.3)

$F_x = 0.01 W_x$

W_x = TOTAL DEAD LOAD PER STORY

* SEE SPREADSHEET FOR FLOOR WEIGHTS AND STORY FORCES.

Appendix B Hollow Core Slab Calculations

HOLLOW CORE SLABS

NEAREST MANUFACTURER → OLDCASTLE PRECAST, EDGEWOOD MD
↳ USES ELEMATIC PLANKS

SECTION E 8" x 48" WITH 2" TOPPING
- SPAN: 24'-3"

LOADING: S.I. DL = 10 PSF
LL ROOMS = 40 PSF
LL Corridors = 15 PSF
65 PSF < - 25' span, 139 psf ✓

- (6) 7/16" Lo Lax STRANDS
- 4' WIDE PLANKS

- PROPERTIES:
  - PLANK + TOPPING = 54 + 25 = 79 PSF

$f'_c = 5000\,psi$ $f'_{ci} = 3000\,psi$ $A = 207\,in^2$ $E_s = 29{,}000\,Ksi$
$f_{pu} = 270{,}000\,psi$ $I_c = 3{,}072\,in^4$ $b_w = 10\,in$

* CALCULATIONS REFERENCED FROM PCI MANUAL

LOSS OF PRESTRESS

- ELASTIC SHORTENING
  - $A_{ps} f_{pu} = 0.115(270) = 31.05\ ^K/_{STRAND}$
  - $P_i = 0.6(6)(31.05) = 111.78K$
  - $M_g = \frac{(24.25)^2}{8}(0.054)(4)' = 190.5\,in\text{-}K$
  - $F_{cir} = K_{cir}\left(\frac{P_i}{A} + \frac{P_i e^2}{I}\right) - \frac{M_g e}{I}$

$$= 0.9\left(\frac{111.78}{207} + \frac{111.78\,(3)^2}{3{,}072}\right) - \frac{190.5(3)}{3072} = 0.59\,Ksi$$

  - $ES = K_{es}\frac{E_s}{E_{ci}} f_{cir}$

$$= (1.0)\frac{29000}{3250}(0.59)$$

$$= \underline{5.26\,Ksi}$$

- CONCRETE CREEP

$- M_{sd} = \frac{(24.25)^2}{8}(0.025+0.01)(4)^{\times 12} = 123.5\ \text{K-in}$

$- f_{cds} = \frac{M_{sd}\, e}{I} = \frac{123.5(3)}{3072} = 0.12\ \text{Ksi}$

$- CR = K_{cr}\frac{E_s}{E_c}(f_{cir} - f_{cds})$

$= (2.0)\left(\frac{29000}{4300}\right)(0.59-0.12) = \underline{6.34\ \text{Ksi}}$

- SHRINKAGE OF CONCRETE

$- \frac{V}{S} = \frac{\text{AREA}}{\text{PERIMETER}} = \frac{207\ in^2}{2(48+8)} = 1.85$

- USE RH = 70% (FIG 2.2.3.1)

$- SH = 8.2\times10^{-6} K_{sh} E_s \left(1 - 0.06\frac{V}{S}\right) \times (100 - RH)$

$= 8.2\times10^{-6}(1.0)(29000)(1-0.06(1.85))(100-70) = \underline{6.34\ \text{Ksi}}$

- STEEL RELAXATION

$- K_{re} = 5000 \quad J = 0.04$ (TABLE 2.2.3.1)

$- f_{si}/f_{pu} \rightarrow C = 0.53$

$- RE = [K_{re} - J(SH + CR + ES)]\, C$

$= \left[\frac{5000}{1000} - 0.04(6.34 + 6.34 + 5.26)\right]0.53 = 2.27\ \text{Ksi}$

- TOTAL LOSS AT MIDSPAN

$= 5.26 + 6.34 + 6.34 + 2.27 = 20.21\ \text{Ksi}$

$\% = \frac{20.21}{(0.6)(270)}(100) = \boxed{12.5\%}$

COMET 3-0235 — 50 SHEETS — 5 SQUARES 3-0236 — 100 SHEETS — 5 SQUARES 3-0237 — 200 SHEETS — 5 SQUARES 3-0137 — 200 SHEETS — FILLER

SERVICE LOAD STRESS

ACCOUNT FOR LOSSES

- $A_{ps} f_{se} = 0.6(6)(31.05)(1-0.125) = 97.8K$
- $M_{non\ comp} = \frac{(24.25)^2}{8}(0.079)12 = 69.7 \text{ in-}^{K}/_{ft}$

$M_{comp} = \frac{(24.25)^2}{8}(0.065)12 = 57.34 \text{ in-}^{K}/_{ft}$

- Top of Topping

$$f_{top} = \frac{57.34(4)(13-5.41)}{3024}\left(\frac{3605}{4696}\right) = 0.44 \text{ksi}$$

- Top of Plank

$$f_{top} = \frac{97.8}{207} - \frac{97.8(3)(3.97)}{1580} + \frac{69.7(4)(3.97)}{1580} + \frac{57.34(4)(10-5.41)}{3024}$$

$$= 0.781 \text{ Ksi}$$

- Bottom of Plank

$$f_{bot} = 0.47 + 0.737 - 0.7 - \frac{57.34(4)(5.41)}{3024} = 0.09 \text{Ksi}$$

PERMISSIBLE COMPRESSION

$0.45f'c = 0.45(5000) = 2.25 \text{Ksi} > 0.44 \text{Ksi}$ ✓

$0.6 f'c = 0.6(5000) = 3.0 \text{Ksi} > 0.781 \text{Ksi}$ ✓

PERMISSIBLE TENSION

$7.5\sqrt{f'c} = 7.5\sqrt{5000} = .53 \text{ Ksi} > 0.09 \text{Ksi}$ ✓

FLEXURAL STRENGTH

$w_u = 1.2(0.079 + 0.01) + 1.6(0.055) = 0.195 \text{KSF}$

$M_u = \frac{24.25^2}{8}(0.195) = 14.3 \text{ ft-}^{K}/_{ft} = 57.3 \text{ ft}^{K}/_{slab}$

- $\beta_1 = 0.85 - \left(\frac{5000-3500}{1000}\right)0.05 = 0.775$
- $\rho_p = \frac{A_{ps}}{b d_p} = \frac{6(0.115)}{48(7)} = 0.0021$
- $\gamma_p = 0.28$ (Low lax)
- $f_{ps} = 270\left[1 - \frac{0.28}{0.775}\left(0.0021\frac{270}{5}\right)\right] = 258.9 \text{ Ksi}$

$- \omega_p = \frac{\rho_p f_{ps}}{f'_c} = \frac{0.0021(258.9)}{5} = 0.109 < 0.36\beta_1$ ✓

$- a = \frac{A_{ps} f_{ps}}{0.85 f'_c b} = \frac{6(0.115)(258.9)}{0.85(5)(48)} = 0.88''$ ✓

$- \phi M_n = 0.9(6)(0.115)(258.9)\left(7 - \frac{0.88}{2}\right) = 101.31\ ^{ft\text{-}k}/_{slab}$

$M_u = 57.3 < 101.3$ ✓

$- \phi M_n \geq 1.2 M_{cr}$

$f_{bot} = \frac{97.8}{196} + \frac{97.8(3)(5)}{1580} = 1.42 ksi$

$M_{cr} = \frac{3024}{5.41}\left(1.42 + \frac{7.5\sqrt{5000}}{1000}\right) = 90.85\ ftK$

SHEAR

use ≤ 1

$V_c = \min \text{ of } \begin{cases} \left[0.6\sqrt{f'_c} + 700\frac{V_u d_p}{M_u}\right] b_w d_p = \left[0.6\sqrt{5000} + 700(1)\right]\frac{(10)(7)}{12} \\ \left[0.6\sqrt{f'_c} + 700\right] b_w d_p = 4330.8\ Kip \\ 5\sqrt{f'_c}\, b_w d_p = 5\sqrt{5000}\frac{(10)(7)}{12} = 2062.4\ Kip \end{cases}$

$V_u = w\left(\frac{l}{2} - x\right)$

$\omega = [1.2(54+10) + 1.6(55)]\,4' = 659.2\ ^{lb}/_{ft}$

$V_u = \frac{\omega l}{2} = \frac{0.6592(24)}{2} = 7.9\ Kip$

$V_u < V_c$ ✓

Initial camber

$$\frac{Pe\,\ell^2}{8E_sI} - \frac{5w\,\ell^4}{384EI}$$

$$\frac{(1-0.125)(111.78)(3)(24.25\times12)^2}{8(3250)(3024)} - \frac{5(3.5)(0.079)(24.25)^4(1728)}{384(3250)(3024)}$$

$$= 0.316'' - 0.218'' = 0.098$$

LONG TERM CAMBER (table 2.4.1)

$$2.2(0.316) - 2.4(0.218) = 0.172'' \therefore \tfrac{1}{4}'' \text{ camber}$$

DEFLECTION

$P = A_{topping}$ (strain) modulus)

$= 48''(2'')(0.00025)(3605)$

$= 86.52K$

$$P = \frac{86.52}{2.3} = 37.6K \quad \text{(TABLE 2.4.1)}$$

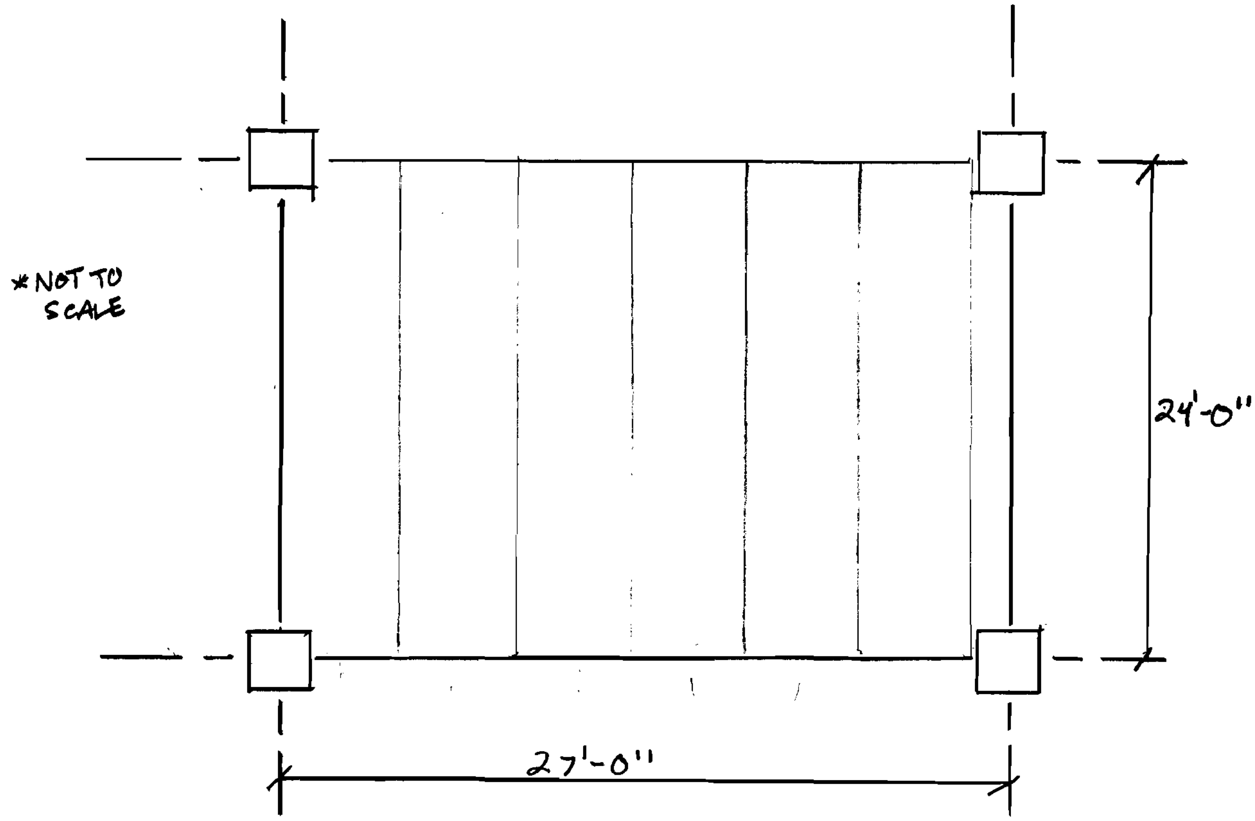
$e = 9'' - 4 = 5''$

$M = Pe = (37.6)(5) = 188.1 \text{ Kin}$

$$\downarrow\Delta = \frac{M\ell^2}{8EI} = \frac{188.1(24.25\times12)^2}{8(4696)(3024)} = 0.14''$$

$$\frac{L}{360} = \frac{24.25\times12}{360} = 0.81'' > 0.14'' \checkmark$$

Appendix C Steel Beam/Column Checks



HOLLOW CORE SLAB

E8" × 48" W/ 2" TOPPING

DL = 54 (PLANK) + 25 (TOPPING) + 10 (SI) = 89 PSF

$W_U = 1.2(89) + 1.6(55) = 194.8$ PSF × 4 ft SPACING

$W_U = 0.779$ K/F

$$M_U = \frac{(0.779)(24)^2}{8} = 56.1 \text{ FtK}$$

STRENGTH

TRY W10×22

$\phi M_n = 97.5$ FtK > 56.1 FtK

Pg WGW 866

CHECK DEFLECTION

FIND REQUIRED I_x TO LIMIT DEFLECTIONS TO $L/240$

$$I_{req} = \frac{5(240)(89+55)(4)(24\times12)^3}{384(29000)\ 12^{in}/_{ft}} = 123.6\,in^4$$

FIND REQUIRED I_x TO LIMIT LIVE LOAD Δ to $L/360$

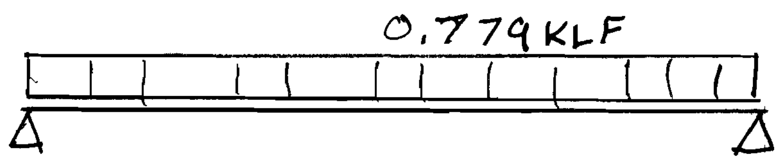
$$I_{req} = \frac{5(360)(55)(4)(24\times12)^3}{384(29000)\ 12^{in}/_{ft}} = 70.8\,in^4$$

USE W10x26

$$\phi M_n = 117\,ftK > 56.1$$

$$I_x = 144\,in^4 > 123.6\,in^4$$

GIRDER DESIGN



$$M_u = \frac{(0.779)(27)^2}{8} = 71.0\,ftK$$

CHECK DEFLECTION

↳ REFERENCE STEEL MANUAL TABLE 3-23 #1

$$\frac{L}{240} = \frac{5 w l^4}{384 E I}$$

$$I_{req} = \frac{240(5) w l^4}{384 E L}$$

$$I_{req} = \frac{240(5)(0.779)(27\times12)^3}{384(29000)}$$

$$= 2855.1 \text{ in}^4$$

FIND REQUIRED I_x TO LIMIT LIVELOADS TO $L/360$

$$I_{req} = \frac{360(5)(0.352)(27\times12)^3}{384(29000)}$$

$$= 1935.2 \text{ in}^4$$

USE W14 × 233

$\phi M_n = 1640 \text{ ft·K} > 71.0 \text{ ft·K}$

$I_x = 3010 \text{ in}^4 > 2855.1 \text{ in}^4$

INTERIOR COLUMN J17

TRIB AREA = $\left(\frac{17'+27'}{2}\right)\left(\frac{24'+8'}{2}\right)$

$= 352\ ft^2$

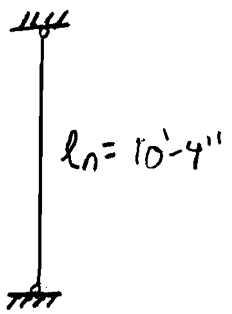
$P_d = 89\ psf\ (352\ ft^2)(8\ FLOORS) = 250K$

$P_L = 55\ psf\ (352\ ft^2)(8\ FLOORS) = 155K$

$P_{LR} = 20\ psf\ (352\ ft^2) = 8K$

$P_u = 1.2D + 1.6L + 0.5LR$

$= 1.2(250) + 1.6(155) + 0.5(8) = 552K$



PINNED-PINNED (TABLE C-A-7.1)

$K = 1.0$

$KL_y \approx 11'$

TABLE 4-1

MINIMUM REQ. SIZE

W12x58

$\phi P_n = 625K > P_u$ ✓

$\frac{P_u}{\phi P_n} = \frac{552K}{625K} = 0.88 < 1.0$

EXTERIOR COLUMN K17

TRIB AREA $= \left(\frac{17'+27'}{2}\right)\left(\frac{24'-2''}{2}\right)$

$= 265.8\,ft^2$

TYPICAL EXTERIOR WALL LOAD $= 52\,PSF \times 10.3' = 535.6\,plf$

$P_d = \left[89\,psf(265.8) + 535.6\,plf\left(\frac{17'+27'}{2}\right)\right](8\,floors) = 283.5K$

$P_L = 55\,psf(265.8\,ft^2)(8\,floors) = 116.9\,K$

$P_{LR} = 20\,psf(265.8\,ft^2) = 5.3K$

$P_U = 1.2D + 1.6L + 0.5LR$

$= 1.2(283.5) + 1.6(116.9) + 0.5(5.3)$

$= 530K$

TABLE 4-1 $\quad KL_y \approx 11' \quad K = 1.0$

MINIMUM REQ SIZE

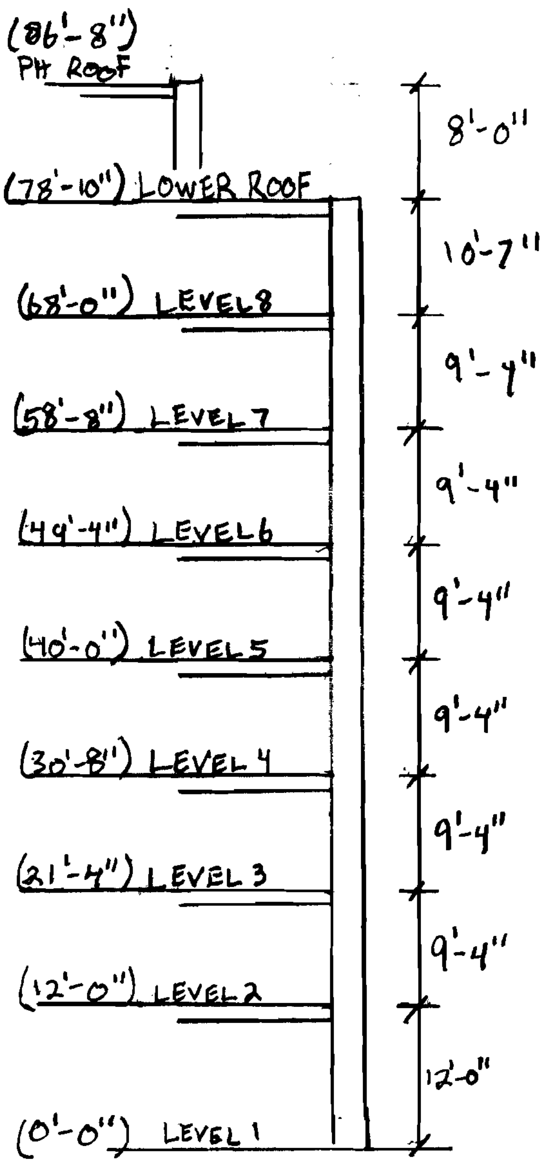
W12×53

$\phi P_n = 571K > P_U$

$\frac{P_U}{\phi P_n} = \frac{530}{571} = 0.928 < 1.0$

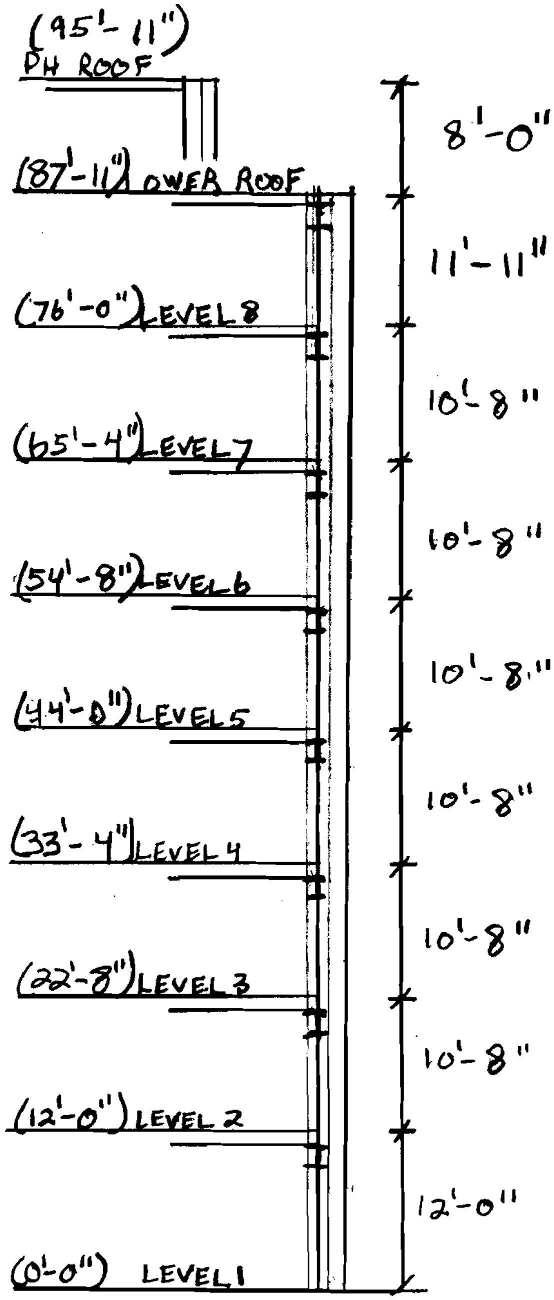
- 24"x24" CONC COLUMNS
- 8" PT SLAB

EXISTING FLOOR HEIGHTS



- W12 COLUMNS
- VARYING W12, W14 BEAMS
- 8" HC SLAB, 2" TOPPING

PROPOSED FLOOR HEIGHTS



Appendix E Center of Rigidity/Center of Mass Tables

BRACED FRAME STIFFNESSES

		A (in²)	I_x (in⁴)
COLUMNS:	W10x33	9.71	171
	W12x40	11.7	307
	W12x58	17.0	475
BRACES:	W8x10	2.96	30.8

$$K_{col} = \frac{3E(I_1+I_2)}{h^3} \qquad K_{brace} = \frac{AE}{L}$$

BF1:

$$K_{col} = \frac{3(29000)(171+171)}{(9.85\times12)^3} = 18.02\,\text{k/in}$$

$$K_{brace} = \frac{30.8(29000)}{11.16\times12} = 6669.7\,\text{k/in}$$

$K_{brace} = 6687.7\,\text{k/in}$

BF2:

$$K_{col} = \frac{3(29000)(171+171)}{(9.85\times12)^3} = 18.02\,\text{k/in}$$

$$K_{brace} = \frac{(30.8)(29000)}{(14.67)\times12} = 5073.8\,\text{k/in}$$

$K_{BRACE} = 5091.9\,\text{k/in}$

BF3:

$$K_{col} = \frac{3(29000)(171+171)}{(9.85\times12)^3} = 18.02\,\text{k/in}$$

$$K_{brace} = \frac{(30.8)(29000)}{(11.16)\times12} = 6669.7\,\text{k/in}$$

$K_{brace} = 6687.7\,\text{k/in}$

BF4:

$$K_{col} = \frac{3(29000)(171+171)}{(9.85\times12)^3} = 18.02\,\text{k/in}$$

$$K_{brace} = 2\frac{(30.8)(29000)}{(14.67)\times12} = 10147.7\,\text{k/in}$$

$K_{brace} = 10165.7\,\text{k/in}$

BF5

$$K_{col} = \frac{3(29000)(307+171)}{(9.85 \times 12)^3} = 25.2 \, ^{k}/_{in}$$

$$K_{brace} = \frac{2\,30.8\,(29000)}{10.21 \times 12} = 14580.5 \, ^{k}/_{in}$$

$K_{brace} = 14605 \, ^{k}/_{in}$

BF6

$$K_{col} = \frac{3(29000)(307+171)}{(9.85 \times 12)^3} = 25.2 \, ^{k}/_{in}$$

$$K_{brace} = \frac{30.8\,(29000)}{12.24 \times 12} = 6081.2 \, ^{k}/_{in}$$

$K_{brace} = 6106.4 \, ^{k}/_{in}$

BF7

$$K_{col} = \frac{3(29000)(171+171)}{(9.85 \times 12)^3} = 18.02 \, ^{k}/_{in}$$

$$K_{brace} = \frac{30.8(29000)}{10.15 \times 12} = 7333.3 \, ^{k}/_{in}$$

$K_{brace} = 7351.4 \, ^{k}/_{in}$

BF8

$$K_{col} = \frac{3(29000)(307+171)}{(9.85 \times 12)^3} = 25.2 \, ^{k}/_{in}$$

$$K_{brace} = \frac{30.8(29000)}{12.24 \times 12} = 6081.2 \, ^{k}/_{in}$$

$K_{brace} = 6106.4 \, ^{k}/_{in}$

BF9

$$K_{col} = \frac{3(29000)(307+171)}{(9.85 \times 12)^3} = 25.2 \, ^{k}/_{in}$$

$$K_{brace} = \frac{30.8\,(29000)}{15.13 \times 12} = 4919.6 \, ^{k}/_{in}$$

$K_{brace} = 4944.8 \, ^{k}/_{in}$

BF10

$$K_{col} = \frac{3(29000)(307+171)}{(9.85 \times 12)^3} = 25.2 \, ^{k}/_{in}$$

$$K_{brace} = \frac{30.8\,(29000)}{10.68 \times 12} = 6969.4 \, ^{k}/_{in}$$

$K_{brace} = 6994.6 \, ^{k}/_{in}$

BF11

$$K_{col} = \frac{3(29000)(307+307)}{(9.85 \times 12)^3} = 32.3 \text{ k/in}$$

$$K_{brace} = \frac{30.8(29000)}{15.13 \times 12} = 4919.6 \text{ k/in}$$

$$K_{brace} = 4951.9 \text{ k/in}$$

BF12

$$K_{col} = \frac{3(29000)(307+307)}{(9.85 \times 12)^3} = 32.3 \text{ k/in}$$

$$K_{brac} = \frac{30.8(29000)}{10.68 \times 12} = 6969.4 \text{ k/in}$$

$$K_{brace} = 7001.7 \text{ k/in}$$

BRACED FRAME WEIGHTS (STORY 5)

BF1 = (2)(33 lb/ft × 9.33 ft) + (10 lb/ft × 12.25 ft) + (2)(10 lb/ft × 11.61 ft)
= 970.28 lbs

BF2 = (2)(33 × 9.33) + (10 × 22.6) + (2)(10 × 14.67) = 1135.18 lbs

BF3 = BF1 = 970.28 lbs

BF4 = (3)(33 × 9.33) + (10 × 22.6) + (10 × 43.9) = 1588.67 lbs

BF5 = (2)(33 × 9.33) + (40 × 9.33) + (10 × 16.3) + (10 × 40.7) = 1558.98 lbs

BF6 = (33 × 9.33) + (40 × 9.33) + (10 × 15.83) + (2)(10 × 12.235) = 1084.1 lbs

BF7 = (2)(33 × 9.33) + (10 × 8) + 2(10 × 10.151) = 898.8

BF8 = BF6 = 1084.09 lbs

BF9 = (33 × 9.33) + (40 × 9.33) + (10 × 23.83) + (2)(10 × 15.133) = 948.1 lbs

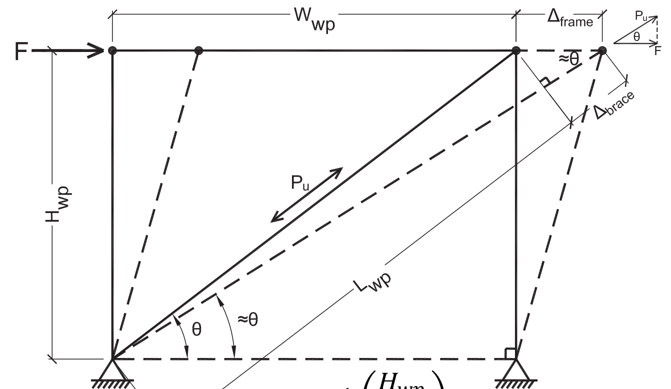
BF10 = (33 × 9.33) + (40 × 9.33) + (10 × 10.4) + (2)(10 × 10.681) = 998.7 lbs

BF11 = 2(40 × 9.33) + (10 × 23.83) + (2)(10 × 15.183) = 1287.4 lbs

BF12 = (40 × 9.33) + (45.933) + (10 × 10.4) + (2)(10 × 10.681) = 1110.67 lbs

Appendix F Buckling Restrained Brace Design Tables

Brace Stiffness Calculations



$$\theta = \tan^{-1}\left(\frac{H_{wp}}{W_{wp}}\right)$$

EQ# **Brace Force**

❶ $$P_u = F/\cos\theta = F\left(\frac{L_{wp}}{W_{wp}}\right)$$

Brace Elongation & Stiffness

❷ $$\Delta_{brace} = \frac{P_u\, L_{wp}}{KF\, A_{sc}\, E} = P_u/K_{eff} \Rightarrow$$

❸ $$K_{eff} = P_u/\Delta_{brace} \Rightarrow K_{eff} = \frac{KF\, A_{sc}\, E}{L_{wp}}$$

Frame Deflection (Elastic)

$$\Delta_{frame} = \frac{\Delta_{brace}}{\cos\theta} \xRightarrow{\text{sub from ❷}} \Delta_{frame} = \frac{P_u}{K_{eff}\cos\theta} \xRightarrow{\text{sub from ❶}}$$

❹ $$\Delta_{frame} = \frac{F}{K_{eff}\cos^2\theta} \xRightarrow{\text{sub from ❸}} \Delta_{frame} = \frac{F\, L_{wp}}{KF\, A_{sc}\, E\cos^2\theta}$$

Alternate Forms (Elastic)

$$\Delta_{frame} = \frac{2\Phi\left(F_y/E\right) H_{wp}}{KF\,\sin 2\theta}$$

$$\text{Story Drift} = \frac{\Delta_{frame}}{H_{wp}} = \frac{2\Phi\left(F_y/E\right)}{KF\,\sin 2\theta}$$

For LRFD design, use Φ = 0.9 in addition to any factor of safety resulting from the ratio of the required A_{sc} to the provided A_{sc}.

Frame Stiffness

The effective horizontal stiffness can be summarized by the following statement:

$$\Delta_{frame} = F/K_{frame} \Rightarrow$$

$$K_{frame} = F/\Delta_{frame} \xRightarrow{\text{sub from ❹}}$$

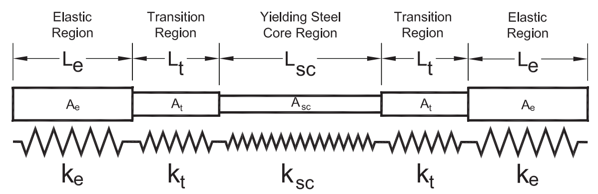
$$K_{frame} = K_{eff}\cos^2\theta$$

For design assistance please contact CoreBrace:

5789 West Wells Park Road
West Jordan, UT 84081
801.280.0701
www.corebrace.com

Rev 3 9/2014

Stiffness Determination of a Composite Element



$$K_{sc\_wp} = \frac{A_{sc}\,E}{L_{wp}}$$ Axial stiffness of yielding core projected from workpoint to workpoint

$$K_{eff} = \frac{1}{\sum_{i=0}^{n}\frac{1}{k_i}}$$ Effective (Actual) stiffness of composite element

$$KF = \frac{K_{eff}}{K_{sc\_wp}}$$ Axial Stiffness Adjustment Factor supplied by CoreBrace

$$K_{eff} = \frac{KF\,A_{sc}\,E}{L_{wp}}$$ Effective stiffness of BRB for use in deflection analysis

Rigid Assumption

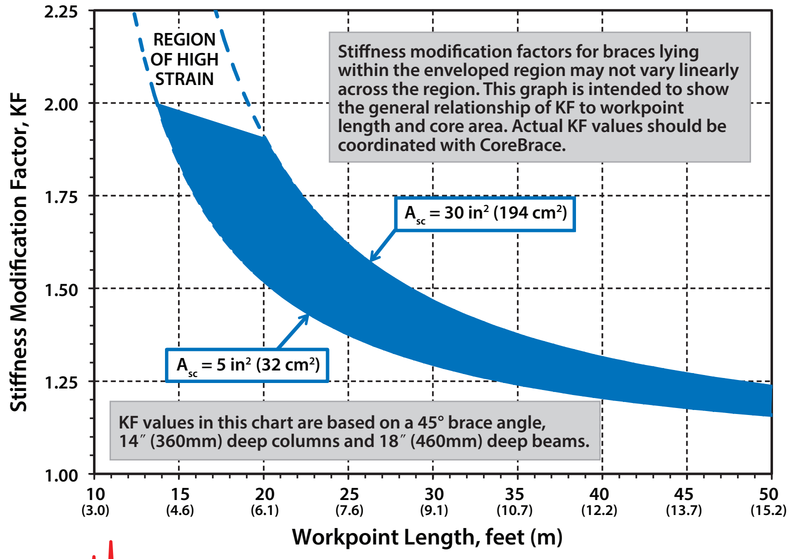
If k_e and k_t are assumed rigid, the above equation for KF simplifies to:

$$KF \cong \frac{L_{wp}}{L_{sc}} = \frac{1}{0.6} = 1.67 \Rightarrow$$

$$K_{eff} \cong 1.67 K_{sc\_wp}$$

This assumption can result in significant error as the ratio of L_{wp}/L_{sc} increases and can underestimate elastic deformations. However, since the elastic deformations of non-yielding regions calculated with non-rigid assumptions are amplified by Cd to calculate their inelastic deformations (which over-predicts these deformations , the rigid assumption may more accurately predict the total inelastic deformations of the element.

Approximate Stiffness Modification Factor

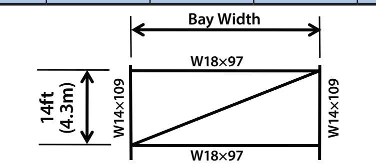


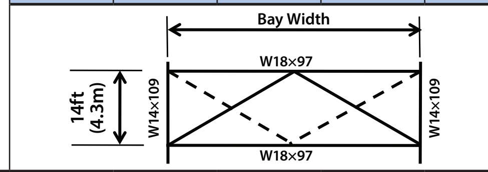
Approximate Stiffness Modification Factors, KF[1,2,7]

Sizes shown are representative of typical BRB sizes. Information on intermediate and larger sizes is available upon request.

STORY HEIGHT: 14ft (4.3m)

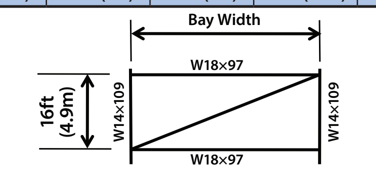
F_{ysc} = 38 ksi (262 MPa)		Bay Width, ft (m)									
A_{sc}[3] in² (cm²)	P_{y_axial}[4] kip (kN)	15 (4.6)	20 (6.1)	25 (7.6)	30 (9.1)	35 (10.7)	30 (9.1)	35 (10.7)	40 (12.2)	45 (13.7)	50 (15.2)
		SINGLE DIAGONAL					CHEVRON/V				
2.0 (13)	68 (306)	1.50	1.44	1.40	1.36	1.34	1.50	1.47	1.44	1.42	1.40
3.0 (19)	103 (448)	1.48	1.42	1.38	1.35	1.33	1.48	1.45	1.42	1.40	1.38
4.0 (26)	137 (613)	1.46	1.41	1.37	1.34	1.32	1.46	1.43	1.41	1.38	1.37
5.0 (32)	171 (754)	1.52	1.46	1.41	1.37	1.35	1.52	1.49	1.46	1.43	1.41
6.0 (39)	205 (919)	1.60	1.52	1.46	1.41	1.38	1.60	1.56	1.52	1.49	1.46
7.0 (45)	239 (1060)	1.66	1.56	1.50	1.45	1.41	1.66	1.61	1.56	1.53	1.50
8.0 (52)	274 (1225)	1.67	1.57	1.50	1.45	1.41	1.67	1.61	1.57	1.53	1.50
9.0 (58)	308 (1367)	1.64	1.55	1.48	1.44	1.40	1.64	1.59	1.55	1.51	1.48
10.0 (65)	342 (1532)	1.79	1.66	1.57	1.50	1.46	1.79	1.71	1.66	1.61	1.57
11.0 (71)	376 (1673)	1.80	1.66	1.57	1.51	1.46	1.80	1.72	1.66	1.61	1.57
12.0 (77)	410 (1814)	1.81	1.67	1.58	1.51	1.47	1.81	1.73	1.67	1.62	1.58
14.0 (90)	479 (2121)	1.65	1.56	1.49	1.45	1.41	1.65	1.60	1.56	1.52	1.49
16.0 (103)	547 (2427)	1.62	1.54	1.48	1.43	1.40	1.62	1.58	1.54	1.50	1.48
18.0 (116)	616 (2733)	1.66	1.57	1.50	1.45	1.42	1.66	1.61	1.57	1.53	1.50
20.0 (129)	684 (3040)	1.63	1.55	1.49	1.44	1.41	1.63	1.59	1.55	1.51	1.49
22.0 (142)	752 (3346)	1.83	1.69	1.60	1.54	1.49	1.83	1.75	1.69	1.64	1.60
24.0 (155)	821 (3652)	1.80	1.67	1.58	1.52	1.48	1.80	1.73	1.67	1.62	1.58
26.0 (168)	889 (3959)	1.77	1.65	1.57	1.51	1.47	1.77	1.70	1.65	1.61	1.57
28.0 (181)	958 (4265)	1.98	1.80	1.69	1.61	1.55	1.98	1.88	1.80	1.74	1.69
30.0 (194)	1026 (4571)	1.95	1.78	1.67	1.60	1.54	1.95	1.86	1.78	1.72	1.67
Workpoint Length, ft (m)		20.5 (6.3)	24.4 (7.4)	28.7 (8.7)	33.1 (10.0)	37.7 (11.5)	20.5 (6.3)	22.4 (6.8)	24.4 (7.4)	26.5 (8.1)	28.7 (8.7)

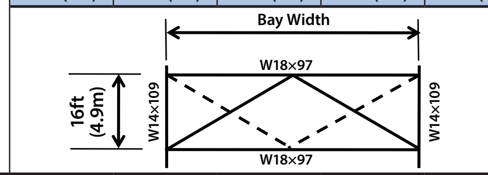




STORY HEIGHT: 16ft (4.9m)

F_{ysc} = 38 ksi (262 MPa)		Bay Width, ft (m)									
A_{sc}[3] in² (cm²)	P_{y_axial}[4] kip (kN)	15 (4.6)	20 (6.1)	25 (7.6)	30 (9.1)	35 (10.7)	30 (9.1)	35 (10.7)	40 (12.2)	45 (13.7)	50 (15.2)
		SINGLE DIAGONAL					CHEVRON/V				
2.0 (13)	68 (306)	1.43	1.39	1.35	1.32	1.30	1.43	1.41	1.39	1.37	1.35
3.0 (19)	103 (448)	1.41	1.37	1.34	1.31	1.29	1.41	1.39	1.37	1.35	1.34
4.0 (26)	137 (613)	1.40	1.36	1.32	1.30	1.28	1.40	1.38	1.36	1.34	1.32
5.0 (32)	171 (754)	1.45	1.40	1.36	1.33	1.31	1.45	1.43	1.40	1.38	1.36
6.0 (39)	205 (919)	1.52	1.46	1.41	1.37	1.34	1.52	1.49	1.46	1.43	1.41
7.0 (45)	239 (1060)	1.57	1.49	1.44	1.40	1.37	1.57	1.53	1.49	1.46	1.44
8.0 (52)	274 (1225)	1.57	1.50	1.44	1.40	1.37	1.57	1.53	1.50	1.47	1.44
9.0 (58)	308 (1367)	1.55	1.48	1.43	1.39	1.36	1.55	1.51	1.48	1.45	1.43
10.0 (65)	342 (1532)	1.67	1.57	1.50	1.45	1.41	1.67	1.62	1.57	1.54	1.50
11.0 (71)	376 (1673)	1.68	1.58	1.51	1.45	1.41	1.68	1.63	1.58	1.54	1.51
12.0 (77)	410 (1814)	1.69	1.59	1.51	1.46	1.42	1.69	1.63	1.59	1.55	1.51
14.0 (90)	479 (2121)	1.56	1.49	1.44	1.40	1.37	1.56	1.52	1.49	1.46	1.44
16.0 (103)	547 (2427)	1.53	1.47	1.42	1.38	1.35	1.53	1.50	1.47	1.44	1.42
18.0 (116)	616 (2733)	1.57	1.49	1.44	1.40	1.37	1.57	1.53	1.49	1.47	1.44
20.0 (129)	684 (3040)	1.55	1.48	1.43	1.39	1.36	1.55	1.51	1.48	1.45	1.43
22.0 (142)	752 (3346)	1.70	1.60	1.53	1.48	1.44	1.70	1.65	1.60	1.56	1.53
24.0 (155)	821 (3652)	1.68	1.58	1.51	1.46	1.43	1.68	1.63	1.58	1.55	1.51
26.0 (168)	889 (3959)	1.65	1.57	1.50	1.45	1.42	1.65	1.61	1.57	1.53	1.50
28.0 (181)	958 (4265)	1.82	1.69	1.60	1.54	1.49	1.82	1.75	1.69	1.65	1.60
30.0 (194)	1026 (4571)	1.80	1.68	1.59	1.53	1.48	1.80	1.73	1.68	1.63	1.59
Workpoint Length, ft (m)		21.9 (6.7)	25.6 (7.8)	29.7 (9.0)	34.0 (10.4)	38.5 (11.7)	21.9 (6.7)	23.7 (7.2)	25.6 (7.8)	27.6 (8.4)	29.7 (9.0)





NOTES:

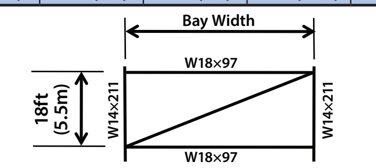
1. KF = K_{eff} / K_{sc_wp} where K_{eff} is the actual WP to WP stiffness of the installed brace and K_{sc_wp} is the stiffness of the indicated core area if it occurred from WP to WP. The indicated KF factors are expected "base value" (minimums). These can be adjusted upwards somewhat, but doing so can affect overstrength factors and may have undesirable effects. Coordinate with CoreBrace.
2. Indicated KF factors are approx values for the indicated frame geometry and beam/column sizes. Different beam and/or column sizes will result in different KF values.
3. Indicated core area is minimum cross sectional area of yielding portion of BRB core.
4. P_{y_axial} is the design yield strength of the BRB equal to: $\phi F_y A_{sc}$ based on the lower-bound of the yield stress range. Typ yield stress range = 42 ksi ± 4 ksi (290MPa ± 28MPa)
5. Where a KF is not given in the table, the brace orientation may result in an unacceptable level of strain. Contact CoreBrace for project specific details.
6. The area of the "Connection Region" (WP to Tip of gusset) is taken to be on average 5× stiffer than the "End Region" of the brace, which results in a semi-rigid offset.
7. Values given are intended for schematic design only. Contact CoreBrace for project specific information.
8. Brace sizes and Stiffness Modification Factors in addition to those shown are available upon request.
9. This table was created by considering core strain at 2% story drift minimum as required by AISC 341-10. For other story drifts, values may be different.

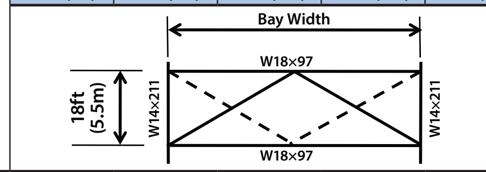
Approximate Stiffness Modification Factors, KF[1,2,7] (CONT'D)

Sizes shown are representative of typical BRB sizes. Information on intermediate and larger sizes is available upon request.

STORY HEIGHT: 18ft (5.5m)

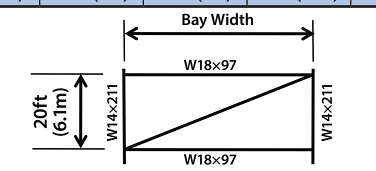
F_{ysc} = 38 ksi (262 MPa)		Bay Width, ft (m)									
A_{sc}[3] in² (cm²)	P_{y_axial}[4] kip (kN)	15 (4.6)	20 (6.1)	25 (7.6)	30 (9.1)	35 (10.7)	30 (9.1)	35 (10.7)	40 (12.2)	45 (13.7)	50 (15.2)
		SINGLE DIAGONAL					CHEVRON/V				
2.0 (13)	68 (306)	1.40	1.35	1.32	1.29	1.27	1.40	1.36	1.35	1.33	1.32
3.0 (19)	103 (448)	1.38	1.33	1.30	1.28	1.26	1.38	1.35	1.33	1.32	1.30
4.0 (26)	137 (613)	1.37	1.32	1.29	1.27	1.25	1.37	1.33	1.32	1.30	1.29
5.0 (32)	171 (754)	1.42	1.36	1.33	1.30	1.28	1.42	1.38	1.36	1.34	1.33
6.0 (39)	205 (919)	1.48	1.41	1.37	1.33	1.31	1.48	1.43	1.41	1.39	1.37
7.0 (45)	239 (1060)	1.52	1.44	1.40	1.36	1.33	1.52	1.47	1.44	1.42	1.40
8.0 (52)	274 (1225)	1.53	1.44	1.40	1.36	1.33	1.53	1.47	1.44	1.42	1.40
9.0 (58)	308 (1367)	1.51	1.43	1.38	1.35	1.32	1.51	1.45	1.43	1.41	1.38
10.0 (65)	342 (1532)	1.61	1.51	1.45	1.41	1.37	1.61	1.55	1.51	1.48	1.45
11.0 (71)	376 (1673)	1.62	1.52	1.46	1.41	1.38	1.62	1.55	1.52	1.48	1.46
12.0 (77)	410 (1814)	1.63	1.52	1.46	1.41	1.38	1.63	1.56	1.52	1.49	1.46
14.0 (90)	479 (2121)	1.52	1.43	1.39	1.36	1.33	1.52	1.46	1.43	1.41	1.39
16.0 (103)	547 (2427)	1.49	1.42	1.38	1.35	1.32	1.49	1.44	1.42	1.40	1.38
18.0 (116)	616 (2733)	1.52	1.44	1.40	1.36	1.34	1.52	1.47	1.44	1.42	1.40
20.0 (129)	684 (3040)	1.50	1.43	1.38	1.35	1.33	1.50	1.45	1.43	1.40	1.38
22.0 (142)	752 (3346)	1.64	1.53	1.47	1.43	1.39	1.64	1.57	1.53	1.50	1.47
24.0 (155)	821 (3652)	1.62	1.52	1.46	1.42	1.38	1.62	1.55	1.52	1.49	1.46
26.0 (168)	889 (3959)	1.60	1.50	1.45	1.41	1.38	1.60	1.53	1.50	1.47	1.45
28.0 (181)	958 (4265)	1.75	1.61	1.54	1.48	1.44	1.75	1.66	1.61	1.57	1.54
30.0 (194)	1026 (4571)	1.73	1.60	1.53	1.47	1.43	1.73	1.64	1.60	1.56	1.53
Workpoint Length, ft (m)		23.4 (7.1)	26.9 (8.2)	30.8 (9.4)	35.0 (10.7)	39.4 (12.0)	23.4 (7.1)	25.1 (7.7)	26.9 (8.2)	28.8 (8.8)	30.8 (9.4)

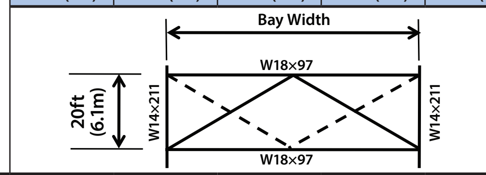




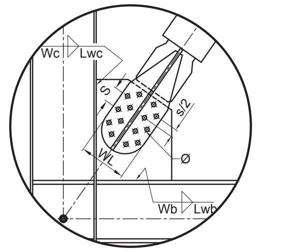
STORY HEIGHT: 20ft (6.1m)

F_{ysc} = 38 ksi (262 MPa)		Bay Width, ft (m)									
A_{sc}[3] in² (cm²)	P_{y_axial}[4] kip (kN)	15 (4.6)	20 (6.1)	25 (7.6)	30 (9.1)	35 (10.7)	30 (9.1)	35 (10.7)	40 (12.2)	45 (13.7)	50 (15.2)
		SINGLE DIAGONAL					CHEVRON/V				
2.0 (13)	68 (306)	1.38	1.31	1.29	1.27	1.25	1.38	1.33	1.31	1.30	1.29
3.0 (19)	103 (448)	1.36	1.30	1.28	1.26	1.24	1.36	1.32	1.30	1.29	1.28
4.0 (26)	137 (613)	1.35	1.29	1.27	1.25	1.23	1.35	1.31	1.29	1.28	1.27
5.0 (32)	171 (754)	1.40	1.33	1.30	1.28	1.26	1.40	1.35	1.33	1.31	1.30
6.0 (39)	205 (919)	1.45	1.37	1.33	1.31	1.28	1.45	1.39	1.37	1.35	1.33
7.0 (45)	239 (1060)	1.49	1.40	1.36	1.33	1.30	1.49	1.43	1.40	1.38	1.36
8.0 (52)	274 (1225)	1.50	1.40	1.36	1.33	1.31	1.50	1.43	1.40	1.38	1.36
9.0 (58)	308 (1367)	1.48	1.39	1.35	1.32	1.30	1.48	1.41	1.39	1.37	1.35
10.0 (65)	342 (1532)	1.57	1.46	1.41	1.37	1.34	1.57	1.50	1.46	1.43	1.41
11.0 (71)	376 (1673)	1.58	1.46	1.42	1.38	1.35	1.58	1.50	1.46	1.44	1.42
12.0 (77)	410 (1814)	1.59	1.47	1.42	1.38	1.35	1.59	1.51	1.47	1.44	1.42
14.0 (90)	479 (2121)	1.49	1.39	1.36	1.33	1.30	1.49	1.42	1.39	1.37	1.36
16.0 (103)	547 (2427)	1.47	1.38	1.34	1.32	1.29	1.47	1.41	1.38	1.36	1.34
18.0 (116)	616 (2733)	1.49	1.40	1.36	1.33	1.31	1.49	1.43	1.40	1.38	1.36
20.0 (129)	684 (3040)	1.48	1.38	1.35	1.32	1.30	1.48	1.41	1.38	1.37	1.35
22.0 (142)	752 (3346)	1.61	1.48	1.43	1.39	1.36	1.61	1.52	1.48	1.45	1.43
24.0 (155)	821 (3652)	1.59	1.46	1.42	1.38	1.35	1.59	1.50	1.46	1.44	1.42
26.0 (168)	889 (3959)	1.57	1.45	1.41	1.37	1.34	1.57	1.49	1.45	1.43	1.41
28.0 (181)	958 (4265)	1.70	1.55	1.49	1.44	1.40	1.70	1.60	1.55	1.52	1.49
30.0 (194)	1026 (4571)	1.68	1.53	1.48	1.43	1.40	1.68	1.58	1.53	1.50	1.48
Workpoint Length, ft (m)		25.0 (7.6)	28.3 (8.6)	32.0 (9.8)	36.1 (11.0)	40.3 (12.3)	25.0 (7.6)	26.6 (8.1)	28.3 (8.6)	30.1 (9.2)	32.0 (9.8)





BOLTED COREBRACE BRB TABLES



BOLTED COREBRACE BRB TABLES

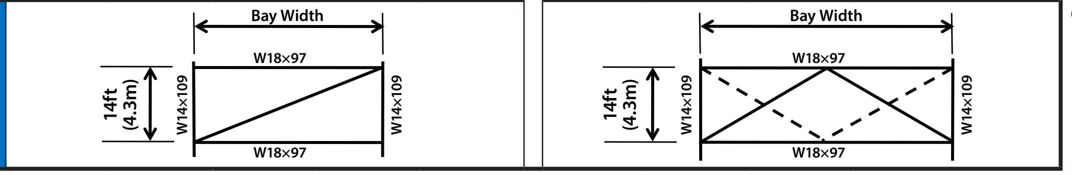
Approximate Casing Sizes[1,8] in (mm)

Sizes shown are representative of typical BRB sizes. Information on intermediate and larger sizes is available upon request.

STORY HEIGHT: 14ft (4.3m)

F_{ysc} = 38 ksi (262 MPa)		Bay Width, ft (m)									
A_{sc}[3] in^2 (cm^2)	P_{y_axial}[4] kip (kN)	15 (4.6)	20 (6.1)	25 (7.6)	30 (9.1)	35 (10.7)	30 (9.1)	35 (10.7)	40 (12.2)	45 (13.7)	50 (15.2)
		SINGLE DIAGONAL					CHEVRON/V				
2.0 (13)	68 (306)	t6 (t152.4)	t6 (t152.4)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)	t6 (t152.4)	t6 (t152.4)	t6 (t152.4)	t8 (t203.2)	t8 (t203.2)
3.0 (19)	103 (448)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)
4.0 (26)	137 (613)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)
5.0 (32)	171 (754)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)	t10 (t254.0)	t10 (t254.0)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)
6.0 (39)	205 (919)	t10 (t254.0)	t10 (t254.0)	t10 (t254.0)	t10 (t254.0)	t12 (t304.8)	t10 (t254.0)	t10 (t254.0)	t10 (t254.0)	t10 (t254.0)	t10 (t254.0)
7.0 (45)	239 (1060)	t10 (t254.0)	t10 (t254.0)	t10 (t254.0)	t10 (t254.0)	t10 (t254.0)	t10 (t254.0)	t10 (t254.0)	t10 (t254.0)	t10 (t254.0)	t10 (t254.0)
8.0 (52)	274 (1225)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)
9.0 (58)	308 (1367)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)
10.0 (65)	342 (1532)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)
11.0 (71)	376 (1673)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)
12.0 (77)	410 (1814)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t14 (t355.6)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)
14.0 (90)	479 (2121)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)
16.0 (103)	547 (2427)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)
18.0 (116)	616 (2733)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t14 (t355.6)	t14 (t355.6)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)
20.0 (129)	684 (3040)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t14 (t355.6)	t14 (t355.6)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)
22.0 (142)	752 (3346)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)
24.0 (155)	821 (3652)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)
26.0 (168)	889 (3959)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)
28.0 (181)	958 (4265)	t16 (t406.4)	t16 (t406.4)	t16 (t406.4)	t16 (t406.4)	t16 (t406.4)	t16 (t406.4)	t16 (t406.4)	t16 (t406.4)	t16 (t406.4)	t16 (t406.4)
30.0 (194)	1026 (4571)	t16 (t406.4)	t16 (t406.4)	t16 (t406.4)	t16 (t406.4)	t16 (t406.4)	t16 (t406.4)	t16 (t406.4)	t16 (t406.4)	t16 (t406.4)	t16 (t406.4)
Workpoint Length, ft (m)		20.5 (6.3)	24.4 (7.4)	28.7 (8.7)	33.1 (10.1)	37.7 (11.5)	20.5 (6.3)	22.4 (6.8)	24.4 (7.4)	26.5 (8.1)	28.7 (8.7)

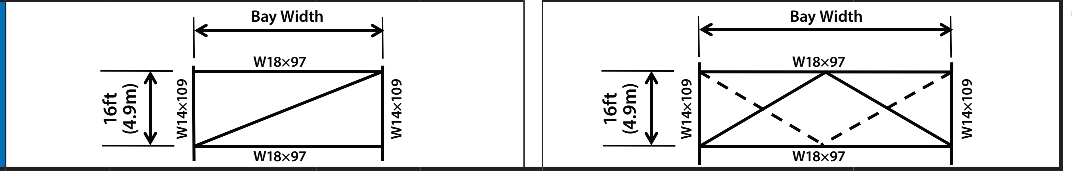
t = square tube (round pipe casing also available)



STORY HEIGHT: 16ft (4.9m)

F_{ysc} = 38 ksi (262 MPa)		Bay Width, ft (m)									
A_{sc}[3] in^2 (cm^2)	P_{y_axial}[4] kip (kN)	15 (4.6)	20 (6.1)	25 (7.6)	30 (9.1)	35 (10.7)	30 (9.1)	35 (10.7)	40 (12.2)	45 (13.7)	50 (15.2)
		SINGLE DIAGONAL					CHEVRON/V				
2.0 (13)	68 (306)	t6 (t152.4)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)	t6 (t152.4)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)
3.0 (19)	103 (448)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)
4.0 (26)	137 (613)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)	t10 (t254.0)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)
5.0 (32)	171 (754)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)	t10 (t254.0)	t10 (t254.0)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)
6.0 (39)	205 (919)	t10 (t254.0)	t10 (t254.0)	t10 (t254.0)	t10 (t254.0)	t12(t304.8)	t10 (t254.0)	t10 (t254.0)	t10 (t254.0)	t10 (t254.0)	t10 (t254.0)
7.0 (45)	239 (1060)	t10 (t254.0)	t10 (t254.0)	t10 (t254.0)	t10 (t254.0)	t10 (t254.0)	t10 (t254.0)	t10 (t254.0)	t10 (t254.0)	t10 (t254.0)	t10 (t254.0)
8.0 (52)	274 (1225)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)
9.0 (58)	308 (1367)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)
10.0 (65)	342 (1532)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)
11.0 (71)	376 (1673)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)
12.0 (77)	410 (1814)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t355.6)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)
14.0 (90)	479 (2121)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)
16.0 (103)	547 (2427)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)
18.0 (116)	616 (2733)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t14 (t355.6)	t14 (t355.6)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)
20.0 (129)	684 (3040)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t14 (t355.6)	t14 (t355.6)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)
22.0 (142)	752 (3346)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)
24.0 (155)	821 (3652)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)
26.0 (168)	889 (3959)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)
28.0 (181)	958 (4265)	t16 (t406.4)	t16 (t406.4)	t16 (t406.4)	t16 (t406.4)	t16 (t406.4)	t16 (t406.4)	t16 (t406.4)	t16 (t406.4)	t16 (t406.4)	t16 (t406.4)
30.0 (194)	1026 (4571)	t16 (t406.4)	t16 (t406.4)	t16 (t406.4)	t16 (t406.4)	t16 (t406.4)	t16 (t406.4)	t16 (t406.4)	t16 (t406.4)	t16 (t406.4)	t16 (t406.4)
Workpoint Length, ft (m)		21.9 (6.7)	25.6 (7.8)	29.7 (9.0)	34.0 (10.4)	38.5 (11.7)	21.9 (6.7)	23.7 (7.2)	25.6 (7.8)	27.6 (8.4)	29.7 (9.0)

t = square tube (round pipe casing also available)



NOTES:

1. CoreBrace BRB Casing Sizes are approx square minimums for the indicated frame geometry and beam/column sizes. Different beam/column sizes will affect brace length and possibly casing size. More economical sizes may be used unless specifically required otherwise. Round casings of similar size are also available.
2. Indicated core area is minimum cross sectional area of yielding portion of BRB core.
3. P_{y_axial} is the calculated yield force of the BRB equal to: $øF_yA_{sc}$ based on the lower-bound of the yield stress range. Typ yield stress range = 42 ksi ±4 ksi (290 MPa ±28MPa).
4. Where casing size is controlled by fit up, alternate core configurations (width × thickness) may allow for reduced casing sizes, on a project specific basis. Contact CoreBrace for details.
5. Casing buckling checks include a FS of 1.3 accounting for code prescribed phi factors and casing initial out-of-straightness. Where casing size is controlled by buckling, alternate casing configurations must maintain the same min moment of inertia about the core critical axis. Contact CoreBrace for assistance with alternate configurations.
6. Brace and casing sizes other than those shown are available upon request.
7. This table was created by considering overstrength at 2% story drift as required by AISC 341-10. For other story drifts, values may be different.
8. Casing sizes in table are intended for schematic design only. Contact CoreBrace for project specific sizes.

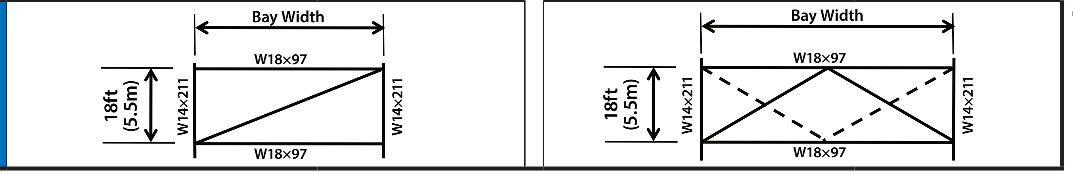
Approximate Casing Sizes[1,8] in (mm) (cont'd)

Sizes shown are representative of typical BRB sizes. Information on intermediate and larger sizes is available upon request.

STORY HEIGHT: 18ft (5.5m)

F_{ysc} = 38 ksi (262 MPa)		Bay Width, ft (m)									
A_{sc}[3] in^2 (cm^2)	P_{y_axial}[4] kip (kN)	15 (4.6)	20 (6.1)	25 (7.6)	30 (9.1)	35 (10.7)	30 (9.1)	35 (10.7)	40 (12.2)	45 (13.7)	50 (15.2)
		SINGLE DIAGONAL					CHEVRON/V				
2.0 (13)	68 (306)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)
3.0 (19)	103 (448)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)
4.0 (26)	137 (613)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)	t10 (t254.0)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)
5.0 (32)	171 (754)	t8 (t203.2)	t8 (t203.2)	t10 (t254.0)	t10 (t254.0)	t10 (t254.0)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)	t10 (t254.0)
6.0 (39)	205 (919)	t10 (t254.0)	t10 (t254.0)	t10 (t254.0)	t10 (t254.0)	t12 (t304.8)	t10 (t254.0)	t10 (t254.0)	t10 (t254.0)	t10 (t254.0)	t10 (t254.0)
7.0 (45)	239 (1060)	t10 (t254.0)	t10 (t254.0)	t10 (t254.0)	t10 (t254.0)	t10 (t254.0)	t10 (t254.0)	t10 (t254.0)	t10 (t254.0)	t10 (t254.0)	t10 (t254.0)
8.0 (52)	274 (1225)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t10 (t304.8)	t10 (t304.8)	t10 (t304.8)	t10 (t304.8)	t10 (t304.8)
9.0 (58)	308 (1367)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t10 (t304.8)	t10 (t304.8)	t10 (t304.8)	t10 (t304.8)	t10 (t304.8)
10.0 (65)	342 (1532)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t10 (t304.8)	t10 (t304.8)	t10 (t304.8)	t10 (t304.8)	t10 (t304.8)
11.0 (71)	376 (1673)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t10 (t304.8)	t10 (t304.8)	t10 (t304.8)	t10 (t304.8)	t10 (t304.8)
12.0 (77)	410 (1814)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t14 (t355.6)	t14 (t355.6)	t10 (t304.8)	t10 (t304.8)	t10 (t304.8)	t10 (t304.8)	t10 (t304.8)
14.0 (90)	479 (2121)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t10 (t304.8)	t10 (t304.8)	t10 (t304.8)	t10 (t304.8)	t10 (t304.8)
16.0 (103)	547 (2427)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t14 (t355.6)	t10 (t304.8)	t10 (t304.8)	t10 (t304.8)	t10 (t304.8)	t10 (t304.8)
18.0 (116)	616 (2733)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t14 (t355.6)	t14 (t355.6)	t10 (t304.8)	t10 (t304.8)	t10 (t304.8)	t10 (t304.8)	t10 (t304.8)
20.0 (129)	684 (3040)	t12 (t304.8)	t12 (t304.8)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)	t10 (t304.8)	t10 (t304.8)	t10 (t304.8)	t10 (t304.8)	t14 (t355.6)
22.0 (142)	752 (3346)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)
24.0 (155)	821 (3652)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)
26.0 (168)	889 (3959)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)	t16 (t406.4)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)
28.0 (181)	958 (4265)	t16 (t406.4)	t16 (t406.4)	t16 (t406.4)	t16 (t406.4)	t16 (t406.4)	t16 (t406.4)	t16 (t406.4)	t16 (t406.4)	t16 (t406.4)	t16 (t406.4)
30.0 (194)	1026 (4571)	t16 (t406.4)	t16 (t406.4)	t16 (t406.4)	t16 (t406.4)	t16 (t406.4)	t16 (t406.4)	t16 (t406.4)	t16 (t406.4)	t16 (t406.4)	t16 (t406.4)
Workpoint Length, ft (m)		23.4 (7.1)	26.9 (8.2)	30.8 (9.4)	35.0 (10.7)	39.4 (12.0)	23.4 (7.1)	25.1 (7.7)	26.9 (8.2)	28.8 (8.8)	30.8 (9.4)

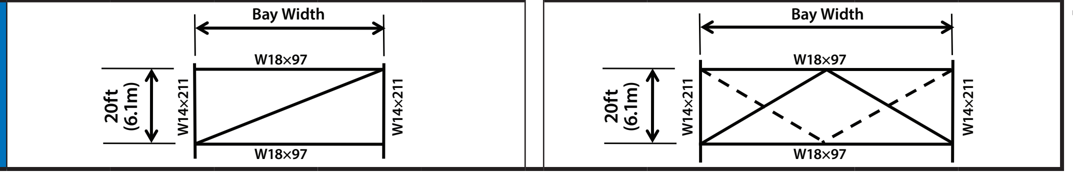
t = square tube (round pipe casing also available)



STORY HEIGHT: 20ft (6.1m)

F_{ysc} = 38 ksi (262 MPa)		Bay Width, ft (m)									
A_{sc}[3] in^2 (cm^2)	P_{y_axial}[4] kip (kN)	15 (4.6)	20 (6.1)	25 (7.6)	30 (9.1)	35 (10.7)	30 (9.1)	35 (10.7)	40 (12.2)	45 (13.7)	50 (15.2)
		SINGLE DIAGONAL					CHEVRON/V				
2.0 (13)	68 (306)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)
3.0 (19)	103 (448)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)
4.0 (26)	137 (613)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)	t10 (t254.0)	t10 (t254.0)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)
5.0 (32)	171 (754)	t8 (t203.2)	t8 (t203.2)	t10 (t254.0)	t10 (t254.0)	t10 (t254.0)	t8 (t203.2)	t8 (t203.2)	t8 (t203.2)	t10 (t254.0)	t10 (t254.0)
6.0 (39)	205 (919)	t10 (t254.0)	t10 (t254.0)	t10 (t254.0)	t12 (t304.8)	t12 (t304.8)	t10 (t254.0)	t10 (t254.0)	t10 (t254.0)	t10 (t254.0)	t10 (t254.0)
7.0 (45)	239 (1060)	t10 (t254.0)	t10 (t254.0)	t10 (t254.0)	t10 (t254.0)	t12 (t304.8)	t10 (t254.0)	t10 (t254.0)	t10 (t254.0)	t10 (t254.0)	t10 (t254.0)
8.0 (52)	274 (1225)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)
9.0 (58)	308 (1367)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)
10.0 (65)	342 (1532)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)
11.0 (71)	376 (1673)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)
12.0 (77)	410 (1814)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t14 (t355.6)	t14 (t355.6)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)
14.0 (90)	479 (2121)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t14 (t355.6)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)
16.0 (103)	547 (2427)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t14 (t355.6)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)
18.0 (116)	616 (2733)	t12 (t304.8)	t12 (t304.8)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t14 (t355.6)
20.0 (129)	684 (3040)	t12 (t304.8)	t12 (t304.8)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t12 (t304.8)	t14 (t355.6)
22.0 (142)	752 (3346)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)
24.0 (155)	821 (3652)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)	t16 (t406.4)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)
26.0 (168)	889 (3959)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)	t16 (t406.4)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)	t14 (t355.6)
28.0 (181)	958 (4265)	t16 (t406.4)	t16 (t406.4)	t16 (t406.4)	t16 (t406.4)	t16 (t406.4)	t16 (t406.4)	t16 (t406.4)	t16 (t406.4)	t16 (t406.4)	t16 (t406.4)
30.0 (194)	1026 (4571)	t16 (t406.4)	t16 (t406.4)	t16 (t406.4)	t16 (t406.4)	t16 (t406.4)	t16 (t406.4)	t16 (t406.4)	t16 (t406.4)	t16 (t406.4)	t16 (t406.4)
Workpoint Length, ft (m)		25.0 (7.6)	28.3 (8.6)	32.0 (9.8)	36.1 (11.0)	40.3 (12.3)	25.0 (7.6)	26.6 (8.1)	28.3 (8.6)	30.1 (9.2)	32.0 (9.8)

t = square tube (round pipe casing also available)



For design assistance please contact CoreBrace:

5789 West Wells Park Road
West Jordan, UT 84081
801.280.0701
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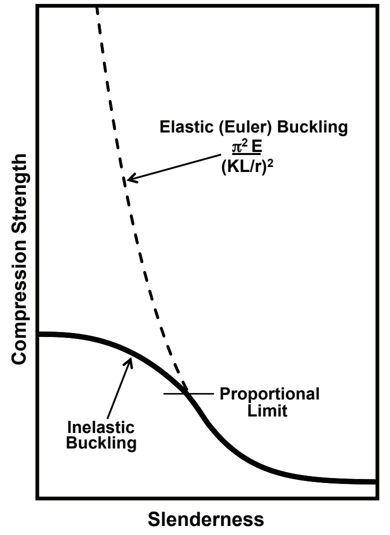
Bolted Lug Brace and Casing Information



1st-Mode Euler Buckling

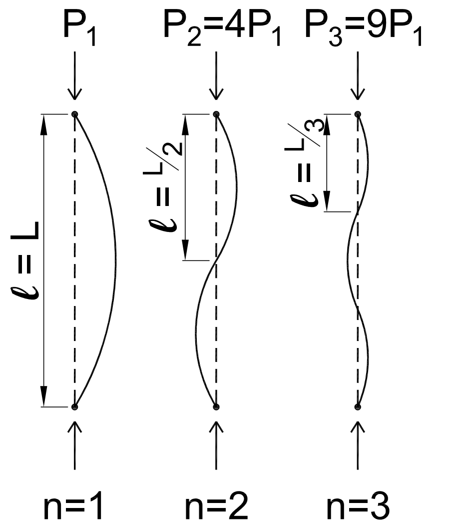
$$P_{cr} = \frac{\pi^2 EI}{(KL)^2} \qquad F_{cr} = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2}$$

nth-Mode Euler Buckling

$$P_{cr\_n} = \frac{\pi^2 EI}{(K\ell)^2} = \frac{\pi^2 EI}{\left(\frac{KL}{n}\right)^2} = n^2\frac{\pi^2 EI}{(KL)^2} = n^2 P_{cr}$$

where $\ell = L/n$

$$P_{cr\_n} = n^2 P_{cr} \qquad F_{cr\_n} = n^2 F_{cr}$$



Casing Demands

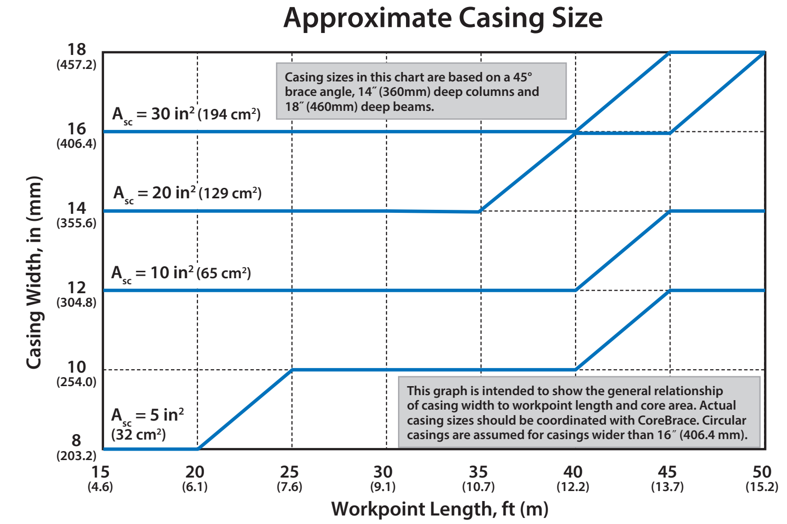
$$I_{req'd} = \frac{FS_B P_u (KL_g)^2}{\pi^2 E}$$

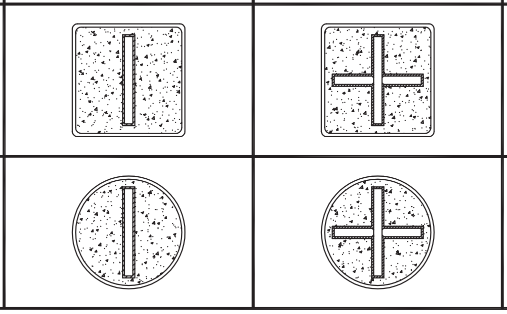
Adjusted Brace Strength Determination

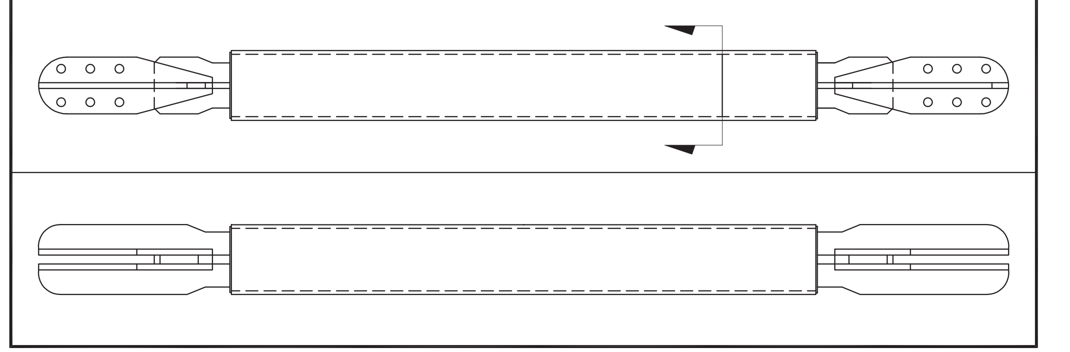
$$P_c = \beta\omega F_y A_{sc} \quad (compression)$$

$$P_t = \omega F_y A_{sc} \quad (tension)$$

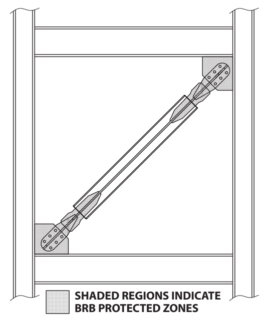
F_y of material used to fabricate brace yielding cores to be established based on coupon testing of individual plates. In such cases, R_y may be taken as equal to 1.0 in the above equations. (See AISC 341)



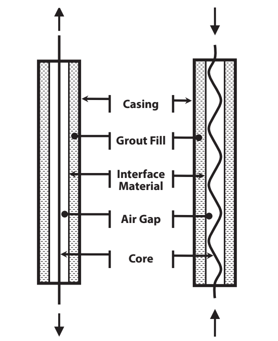
	FLAT	CRUCIFORM	MATERIAL SPECS
SQUARE CASING			CORE PL: A36 CONTROLLED YIELD STIFFENER PL: A36 (F_y = 249 MPa) LUG PL: A572 GR-50 (F_y = 345 MPa) CASING: A500 GR-B (F_y = 318 MPa) GUSSET/REPAD PL: A572 GR-50 (F_y = 345 MPa) BOLTS: A490/F2280 (F_U = 1035 MPa)
ROUND CASING			CORE PL: A36 CONTROLLED YIELD STIFFENER PL: A36 (F_y = 249 MPa) LUG PL: A572 GR-50 (F_y = 345 MPa) CASING: ROUND HSS: A500 GR-B (F_y = 290 MPa) PIPE: A53 GR-B (F_y = 242 MPa) GUSSET/REPAD PL: A572 GR-50 (F_y = 345 MPa) BOLTS: A490/F2280 (F_U = 1035 MPa)



BRB Protected Zones



Schematic BRB Behavior



5789 West Wells Park Road, West Jordan, UT 84081 ph: 801.280.0701 www.corebrace.com

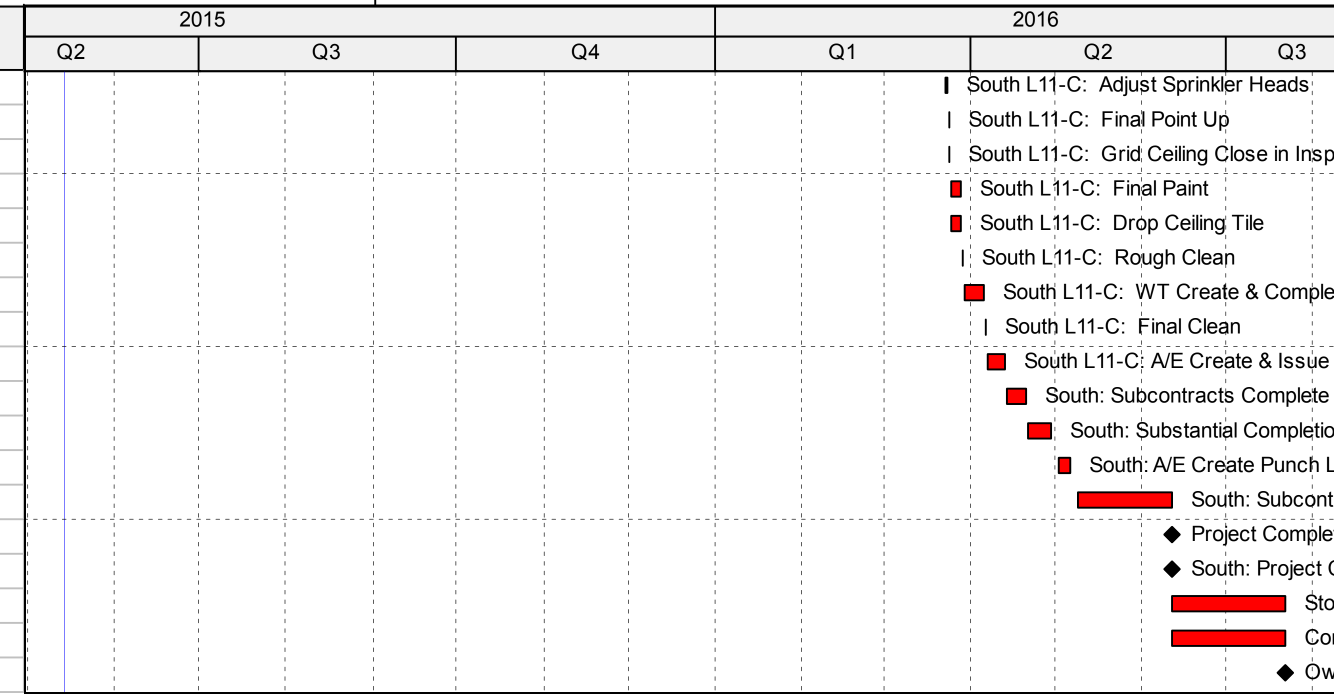
Appendix G Construction Breadth

Towson West Village Housing 3 4 UPDATE 5.14.2015 (CONSTRUCTION BASELINE)		LONGEST PATH			01-Jun-15 14:48
Activity ID	**Activity Name**	**Original Duration**	**Remaining Duration**	**Start**	**Finish**
5200	Procure Scaffold	55	35	27-Apr-15 A	02-Jul-15
11040	South Erect Scaffold	30	30	06-Jul-15	19-Aug-15
12500	South L2: Sheathing	10	10	20-Aug-15	04-Sep-15
12520	South L3: Sheathing	10	10	25-Aug-15	10-Sep-15
18460	South L1-B: Wall & Ceiling Electrical / Fire Alarm Rough In	5	5	11-Sep-15	17-Sep-15
18770	South L1-C: Wall & Ceiling Electrical / Fire Alarm Rough In	5	5	17-Sep-15	23-Sep-15
19080	South L2-A: Wall & Ceiling Electrical / Fire Alarm Rough In	5	5	23-Sep-15	29-Sep-15
19390	South L2-B: Wall & Ceiling Electrical / Fire Alarm Rough In	5	5	29-Sep-15	05-Oct-15
19700	South L2-C: Wall & Ceiling Electrical / Fire Alarm Rough In	5	5	05-Oct-15	09-Oct-15
20010	South L3-A: Wall & Ceiling Electrical / Fire Alarm Rough In	5	5	09-Oct-15	16-Oct-15
20320	South L3-B: Wall & Ceiling Electrical / Fire Alarm Rough In	5	5	16-Oct-15	22-Oct-15
20630	South L3-C: Walls & Ceiling Electrical / Fire Alarm Rough In	5	5	22-Oct-15	28-Oct-15
20940	South L4-A: Walls & Ceiling Electrical / Fire Alarm Rough In	5	5	28-Oct-15	03-Nov-15
21250	South L4-B: Wall & Ceiling Electrical / Fire Alarm Rough In	5	5	03-Nov-15	09-Nov-15
21560	South L4-C: Walls & Ceiling Electrical / Fire Alarm Rough In	5	5	09-Nov-15	16-Nov-15
21870	South L5-A: Walls & Ceiling Electrical / Fire Alarm Rough In	5	5	16-Nov-15	20-Nov-15
22180	South L5-B: Walls & Ceiling Electrical / Fire Alarm Rough In	5	5	20-Nov-15	27-Nov-15
22490	South L5-C: Walls & Ceiling Electrical / Fire Alarm Rough In	5	5	27-Nov-15	03-Dec-15
22140	South L5-B: Wall & Ceiling Close In Inspection	1	1	30-Nov-15	30-Nov-15
22150	South L5-B: Hang & Finish GWB Walls & Ceiling	8	8	01-Dec-15	09-Dec-15
22450	South L5-C: Wall & Ceiling Close In Inspection	1	1	04-Dec-15	04-Dec-15
22460	South L5-C: Hang & Finish GWB Walls & Ceiling	8	8	05-Dec-15	14-Dec-15
28680	South L6-A: Hang & Finish GWB Walls & Ceiling	8	8	10-Dec-15	18-Dec-15
28990	South L6-B: Hang & Finish GWB Walls & Ceiling	8	8	15-Dec-15	23-Dec-15
29300	South L6-C: Hang & Finish GWB Walls & Ceiling	8	8	19-Dec-15	30-Dec-15
29610	South L7-A: Hang & Finish GWB Walls & Ceiling	8	8	24-Dec-15	05-Jan-16
29920	South L7-B: Hang & Finish GWB Walls & Ceilng	8	8	31-Dec-15	09-Jan-16
30230	South L7-C: Hang & Finish GWB Walls & Ceiling	8	8	06-Jan-16	14-Jan-16
30540	South L8-A: Hang & Finish GWB Walls & Ceiling	8	8	11-Jan-16	19-Jan-16
30850	South L8-B: Hang & Finish GWB Walls & Ceiling	8	8	15-Jan-16	23-Jan-16
31160	South L8-C: Hang & Finish GWB Walls & Ceiling	8	8	20-Jan-16	28-Jan-16
22770	South L9-A: Hang & Finish GWB Walls & Ceiling	8	8	25-Jan-16	02-Feb-16
23080	South L9-B: Hang & Finish GWB Walls & Ceiling	8	8	29-Jan-16	06-Feb-16
23390	South L9-C: Hang & Finish GWB Walls & Ceiling	8	8	03-Feb-16	11-Feb-16
23700	South L10-A: Hang & Finish GWB Walls & Ceilings	8	8	08-Feb-16	16-Feb-16
24010	South L10-B: Hang & Finish GWB Walls & Ceiling	8	8	12-Feb-16	20-Feb-16
24320	South L10-C: Hang & Finish GWB Walls & Ceiling	8	8	17-Feb-16	25-Feb-16
24630	South L11-A: Hang & Finish GWB Walls & Ceiling	8	8	22-Feb-16	01-Mar-16
24940	South L11-B: Hang & Finish GWB Walls & Ceiling	8	8	26-Feb-16	05-Mar-16
25250	South L11-C: Hang & Finish GWB Walls & Ceiling	8	8	02-Mar-16	10-Mar-16
25310	South L11-C: Install Door Frames	2	2	11-Mar-16	14-Mar-16
25320	South L11-C: Prime & First Coat Paint	2	2	14-Mar-16	15-Mar-16
25340	South L11-C: Install Kitchen Cabinets	3	3	15-Mar-16	17-Mar-16
39230	South L11-C: Install Ceiling Grid	2	2	16-Mar-16	17-Mar-16
25370	South L11-C: Set Appliances	1	1	18-Mar-16	18-Mar-16
39240	South L11-C: Install Grid Lighting	3	3	18-Mar-16	22-Mar-16
25390	South L11-C: Final Plumbing Trim Out	3	3	21-Mar-16	23-Mar-16

Actual Work
Critical Remaining Work
Remaining Work
Milestone

Towson West Village Housing 3 4 UPDATE 5.14.2015 (CONSTRUCTION BASELINE) | LONGEST PATH | 01-Jun-15 14:48

Activity ID	Activity Name	Original Duration	Remaining Duration	Start	Finish
39250	South L11-C: Adjust Sprinkler Heads	1	1	23-Mar-16	23-Mar-16
25430	South L11-C: Final Point Up	1	1	24-Mar-16	24-Mar-16
39260	South L11-C: Grid Ceiling Close in Inspection	1	1	24-Mar-16	24-Mar-16
25440	South L11-C: Final Paint	2	2	25-Mar-16	28-Mar-16
39270	South L11-C: Drop Ceiling Tile	2	2	25-Mar-16	28-Mar-16
25450	South L11-C: Rough Clean	1	1	29-Mar-16	29-Mar-16
25460	South L11-C: WT Create & Complete Punch List	5	5	30-Mar-16	05-Apr-16
25470	South L11-C: Final Clean	1	1	06-Apr-16	06-Apr-16
25480	South L11-C: A/E Create & Issue Punch List	5	5	07-Apr-16	13-Apr-16
4070	South: Subcontracts Complete WT Work To Complete List	5	5	14-Apr-16	20-Apr-16
4080	South: Substantial Completion / Final Inspections	7	7	21-Apr-16	29-Apr-16
4090	South: A/E Create Punch List	5	5	02-May-16	06-May-16
4100	South: Subcontractors Complete Punch List	25	25	09-May-16	11-Jun-16
2340	Project Completion (6.12.2016)	0	0		11-Jun-16*
4110	South: Project Completion (June 12, 2016)	0	0		11-Jun-16
3640	Stock Furniture	30	30	12-Jun-16	22-Jul-16
4630	Commissioning	30	30	12-Jun-16	22-Jul-16
3650	Owner Move In (8.12.2014)	0	0		22-Jul-16

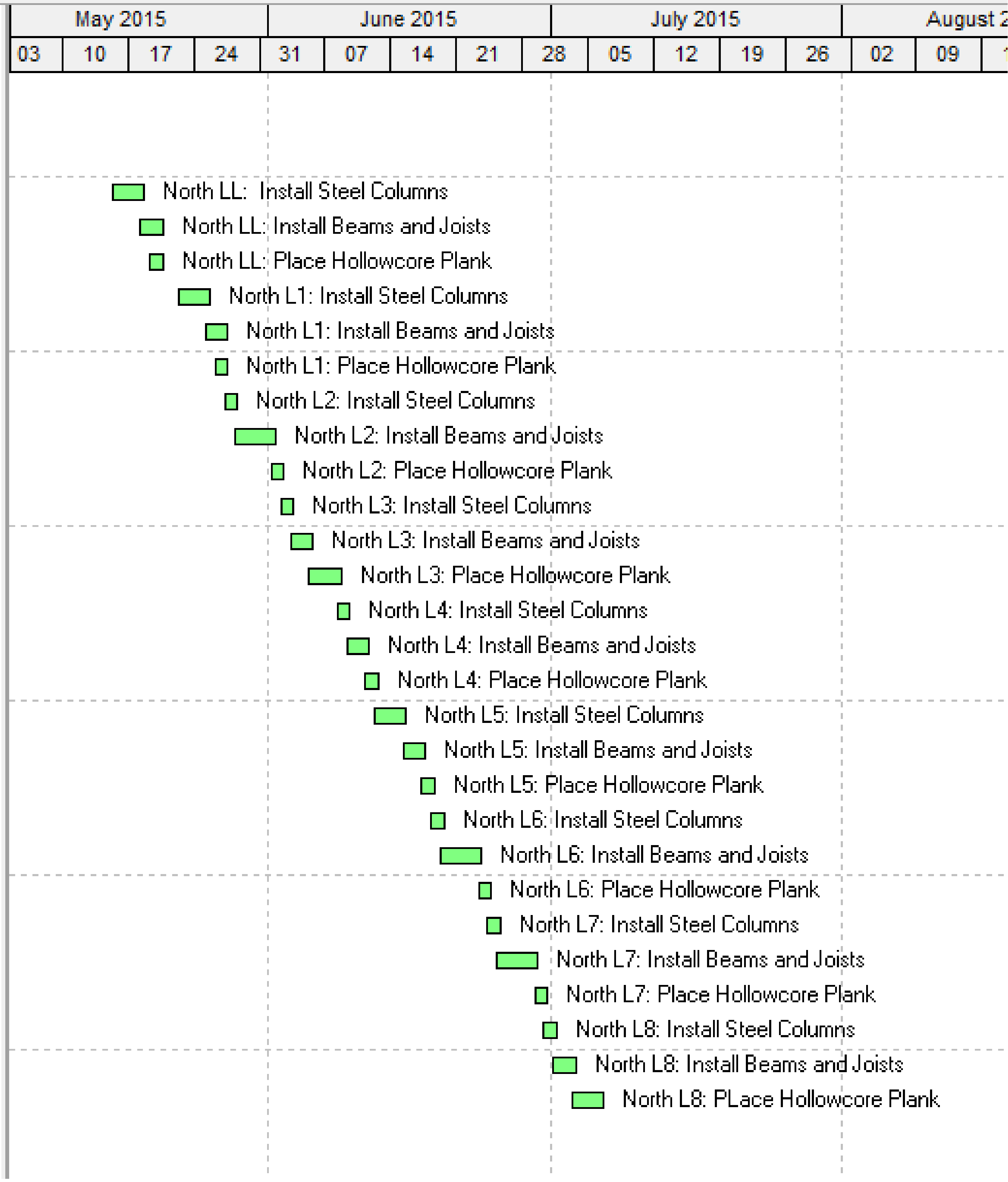


Actual Work
Critical Remaining Work
Remaining Work
Milestone

Activities

Layout: Classic Schedule Layout | Filter: All Activities

Activity ID	Activity Name	Original Duration	Start	Finish
A1090	North: Drill & RAP	13	27-Jan-15	12-Feb-15
A1100	North: Prep & Pour Pile Caps	13	05-Feb-15	23-Feb-15
A1110	North: Prep & Pour Grade Beams	7	13-Feb-15	23-Feb-15
A1120	North LL: Install Steel Columns	2	15-May-15	18-May-15
A1130	North LL: Install Beams and Joists	3	18-May-15	20-May-15
A1140	North LL: Place Hollowcore Plank	2	19-May-15	20-May-15
A1150	North L1: Install Steel Columns	2	22-May-15	25-May-15
A1160	North L1: Install Beams and Joists	3	25-May-15	27-May-15
A1170	North L1: Place Hollowcore Plank	2	26-May-15	27-May-15
A1180	North L2: Install Steel Columns	2	27-May-15	28-May-15
A1190	North L2: Install Beams and Joists	3	28-May-15	01-Jun-15
A1200	North L2: Place Hollowcore Plank	2	01-Jun-15	02-Jun-15
A1210	North L3: Install Steel Columns	2	02-Jun-15	03-Jun-15
A1220	North L3: Install Beams and Joists	3	03-Jun-15	05-Jun-15
A1230	North L3: Place Hollowcore Plank	2	05-Jun-15	08-Jun-15
A1240	North L4: Install Steel Columns	2	08-Jun-15	09-Jun-15
A1250	North L4: Install Beams and Joists	3	09-Jun-15	11-Jun-15
A1260	North L4: Place Hollowcore Plank	2	11-Jun-15	12-Jun-15
A1270	North L5: Install Steel Columns	2	12-Jun-15	15-Jun-15
A1280	North L5: Install Beams and Joists	3	15-Jun-15	17-Jun-15
A1290	North L5: Place Hollowcore Plank	2	17-Jun-15	18-Jun-15
A1300	North L6: Install Steel Columns	2	18-Jun-15	19-Jun-15
A1310	North L6: Install Beams and Joists	3	19-Jun-15	23-Jun-15
A1320	North L6: Place Hollowcore Plank	2	23-Jun-15	24-Jun-15
A1330	North L7: Install Steel Columns	2	24-Jun-15	25-Jun-15
A1340	North L7: Install Beams and Joists	3	25-Jun-15	29-Jun-15
A1350	North L7: Place Hollowcore Plank	2	29-Jun-15	30-Jun-15
A1360	North L8: Install Steel Columns	2	30-Jun-15	01-Jul-15
A1370	North L8: Install Beams and Joists	3	01-Jul-15	03-Jul-15
A1380	North L8: PLace Hollowcore Plank	2	03-Jul-15	06-Jul-15
A1390	North LL: MEP Sleeves	19	24-Feb-15	20-Mar-15



WBS Task	Sub-task Description	Qty.	Units	Production Rate	Crew hours	Crew Cost / hr	# workers on crew	total labor hours
Overhead rough-in	Domestic Water	100	LF	0.25	25	$98.30	2	50
Precast Slab Planks	Hollow, 8" thick	1056	S.F.	0.023	24.288	235.45	1	24.288
Finishing Floors	Integral topping and finish, 2" thick	1056	S.F.	0.08	84.48	189.05	1	84.48
Structural Steel Members	Beam, W 14 x 82	88	L.F.	0.078	6.864	135.5	1	6.864
Structural Steel Members	Beam, W 8 x 10	72	L.F.	0.093	6.696	172	1	6.696
Columns Structural	W shape, 12 x 40	9.33	L.F.	0.054	0.50382	102.5	1	0.50382
Columns Structural	W shape, 12 x 53	77.3	L.F.	0.057	4.4061	135.5	1	4.4061
Structural Steel Members	Beam, W 8 x 10	181	L.F.	0.093	16.8051		1	16.8051

Appendix H Acoustical Breadth

APPENDIX J
Sound Transmission Loss Data

Assembly/Frequency (Hz)	STC	100	125	160	200	250
Metal stud, gypsum board (GB) assemblies with or without mineral wool or fiberglass (FG) insulation						
24 g. studs						
1/2" GB each side of 2-1/2" studs 24" o.c., 1.5" FG	**45**	19	22	26	31	38
Same as above plus an additional layer GB on one side (2+1 layers)	**50**	22	31	31	38	43
Same as above with two layers GB on each side (2+2 layers)	**53**	28	35	35	41	48
5/8" GB on each side of 2-1/2" studs 24" o.c., no insulation	**39**	22	24	24	28	37
Same as above plus 1-1/2" FG	**46**	17	26	28	37	42
Same as above, but with two layers GB on each side (2+2 layers)	**54**	30	37	37	41	46
1/2" GB on each side of 3-5/8" studs 16" o.c., no insulation	**43**	19	26	23	29	36
Same as above but with 3" FG	**49**	20	28	30	37	43
Two layers 1/2" GB each side of 3-5/8" studs 24" o.c. plus 1-1/2" FG	**55**	31	34	36	46	47
5/8" GB each side of 3-5/8" studs 24" o.c. plus 1-1/2" FG	**45**	22	29	31	39	41
Same as above but with 3" FG	**49**	18	32	33	39	44
5/8" GB (2+1 layers) on 3-5/8" studs 24" o.c. and 2" FG	**51**	28	36	37	42	46
Same as above except 3" FG	**53**	25	35	41	46	51
5/8" GB (2+2 layers) on 3-5/8" studs 24" o.c., no insulation	**48**	27	34	30	37	41
5/8" GB (2+2 layers) on 3-5/8" studs 24" o.c., 3" FG	**57**	28	38	44	47	52
Same as above except 3 + 3 GB layers, a total of 6 GB layers	**61**	33	40	47	51	55
20 g. studs						
1/2" GB on each side of 3-5/8" studs, no insulation	**39**	16	26	19	26	36
Same as above plus 2" FG	**41**	19	30	29	34	43