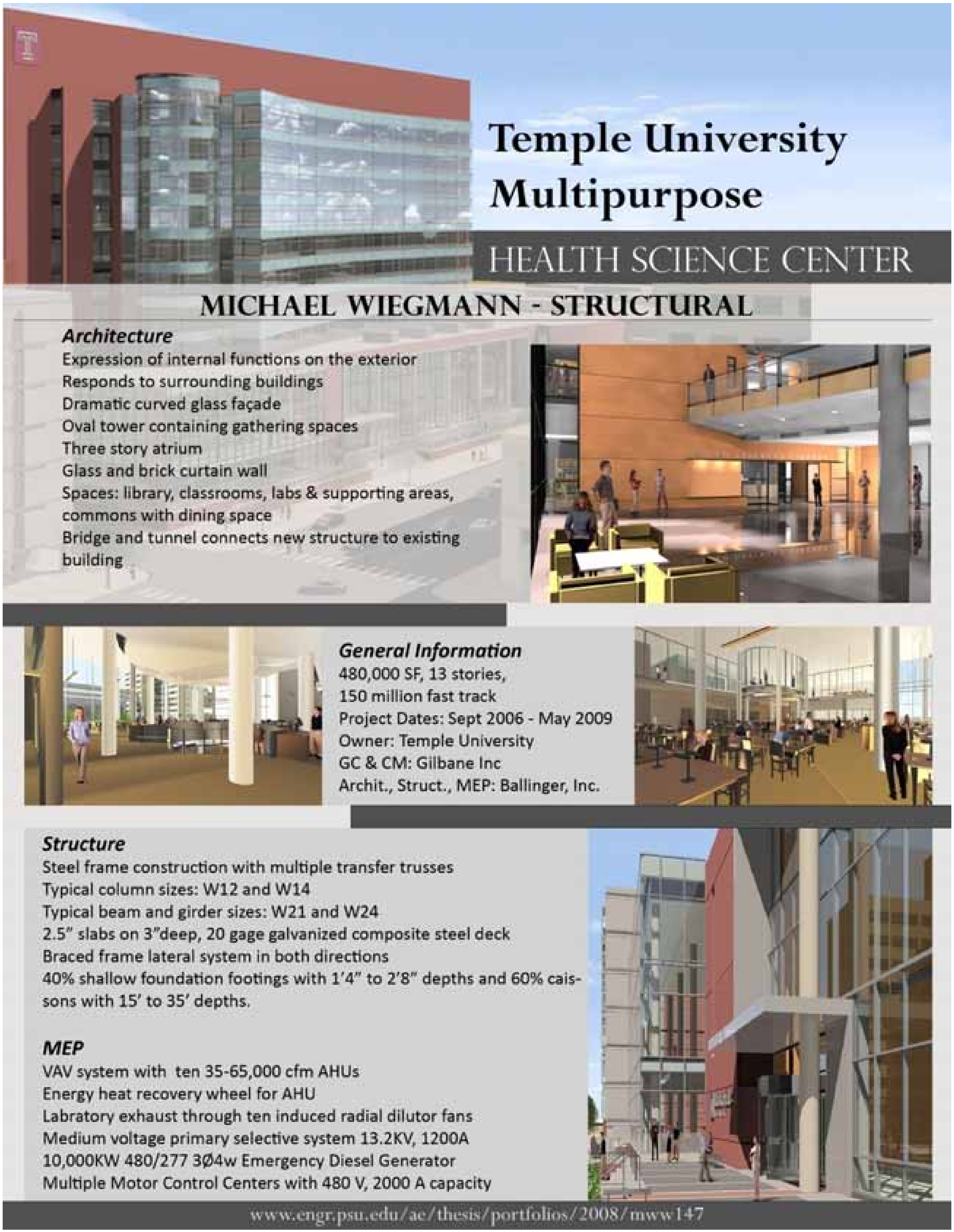
Temple University

3500 N. Broad St. Philadelphia,
Pennsylvania 19140
Michael Wiegmann

Multipurpose Health Science Center

[FINAL REPORT]

Michael Wiegmann – Structural Option – Faculty Advisor Professor Hanagan – Spring 2008



Temple University Multipurpose

HEALTH SCIENCE CENTER

MICHAEL WIEGMANN - STRUCTURAL

Architecture

Expression of internal functions on the exterior
Responds to surrounding buildings
Dramatic curved glass façade
Oval tower containing gathering spaces
Three story atrium
Glass and brick curtain wall
Spaces: library, classrooms, labs & supporting areas, commons with dining space
Bridge and tunnel connects new structure to existing building

General Information

480,000 SF, 13 stories,
150 million fast track
Project Dates: Sept 2006 - May 2009
Owner: Temple University
GC & CM: Gilbane Inc
Archit., Struct., MEP: Ballinger, Inc.

Structure

Steel frame construction with multiple transfer trusses
Typical column sizes: W12 and W14
Typical beam and girder sizes: W21 and W24
2.5" slabs on 3"deep, 20 gage galvanized composite steel deck
Braced frame lateral system in both directions
40% shallow foundation footings with 1'4" to 2'8" depths and 60% caissons with 15' to 35' depths.

MEP

VAV system with ten 35-65,000 cfm AHUs
Energy heat recovery wheel for AHU
Labratory exhaust through ten induced radial dilutor fans
Medium voltage primary selective system 13.2KV, 1200A
10,000KW 480/277 3Ø4w Emergency Diesel Generator
Multiple Motor Control Centers with 480 V, 2000 A capacity

www.engr.psu.edu/ae/thesis/portfolios/2008/mww147

Acknowledgements

The Author of this thesis study wishes to thank the following individuals and organizations for their support, encouragement, and guidance in completing this thesis.

Ballinger, Inc.

Edward Zinski
Scott Holsinger
Joyce Raybuck

Temple University

The Pennsylvania State University

Dr. Linda M. Hanagan
M. Kevin Parfitt
Robert J. Holland
The entire AE faculty,
staff, and students

Table of contents

[INTRODUCTORY MATERIALS]

Executive Summary – Introduction – Design Impetus – Building Information

Executive Summary

The Multipurpose Health Science Center (MHSC) is a new, state of the art medical facility within the heart of Philadelphia. The great urban location comes with a primary shortcoming: lack of space, which has prompted the owner to express need for a significant program expansion including the addition of green space; thereby requesting an in-depth architectural, green roof, and structural investigation.

Architectural Breath

By analyzing the existing architectural conditions, a building mass, typical floor plans and framing layouts were created which successfully met the 126,000 sf program requirement at 83% efficiency. Finally, an egress checked insured that the adequate size and number of exits were present to insure the safety of the building's 6,000 potential occupants.

Green Roof Breath

The green roof will be a dual intensive and extensive system with a 1400sf accessible garden above the library, with an extensive green roof system covering the remaining 4800 sf of roof. This breadth will show that the $49,871 cost for the intensive roof is only 0.03% of the total $150 million budget. It becomes clear that the green roof is a very feasible addition to the budget, especially when long term cost reductions in storm water treatment and energy consumption are considered.

Structural Depth

The structural system had to not only meet the strength and serviceability requirements of the new design, but to do so as efficiently as possible. Extensive computer modeling of the lateral and gravity system was used to meet these design criteria, which was verified by even more extensive manual checks.

One element of the lateral system design was an attempt at creating a more efficient system by eliminating existing moment frames and replacing them with braces frames. After dozens of design iterations in RAM, a check using STAAD, and multiple hand calculations, a finalized design was made which not only met the 8.8" drift limit and resisted the 2300kip wind load, but also decreased the frame cost relative to the square footage by 6%.

In-depth hand checks of the gravity system verified the computer modeling data, provided further evidence of the design's effectiveness in addressing the program requirements.

Lastly, a vibration analysis for sensitive equipment was made, which demonstrated that the floor system is adequate for less sensitive equipment and procedures, such as microscopes with 100x magnification and surgery. This may meet the university's needs; however, a redesign would be required to accommodate more sensitive equipment.

Introduction

The report culminates the Architectural Engineering Senior Thesis Project. Based within the structure option, the purpose of the thesis is to investigate and analyze an existing building, and then to provide a scenario which requires the redesign of the building. This report focuses on the redesign scenario, but integrates some of the initial analysis results, which are available in their full at the website listed in the building abstract under Tech1, 2 and 3.

The addition of five levels to the original design is the basis for the redesign scenario – the structural breadth – and is the core of the architectural and green roof breadths. Two goals guided the completion of this thesis:

The creation of a design which meets the addition's requirements.

To make this design as efficient as possible.

The Design Impetus section further details this design problem and outlines the process for solving it. The following Building Information section introduces general information relating to the original and new designs.

The rest of the report is separated into five sections, each containing a set of defined goals, a design process, and conclusions. The first is **Architectural Breadth**, which details the architectural planning and design of the building addition. This is followed by the **Green Roof Breadth** which describes the green roof selection and design process. These two sections provide the design impetus that defines the requirements of the **Structural Breadth** section, which is broken into: Introduction, Building Loads & Design Codes, Overall Addition, Lateral System Design, Gravity System Design, Vibration Check, and Conclusions. The remaining sections are **Conclusions**, which summarizes all of the design information, and the **Appendix**.

Design Impetus

Problem Statement

The Multipurpose Health Science Center (MHSC) is a new, state of the art facility within the heart of Philadelphia. The great urban location comes with a primary shortcoming: lack of space. Therefore, the owner has expressed the need for significant expansion, requesting the addition of five levels to the existing design, as well as a feasibility study into the addition of a green roof, with an accessible garden above the library. This will alleviate the constriction of urban green space while giving the medical campus room to grow (in red on map below).

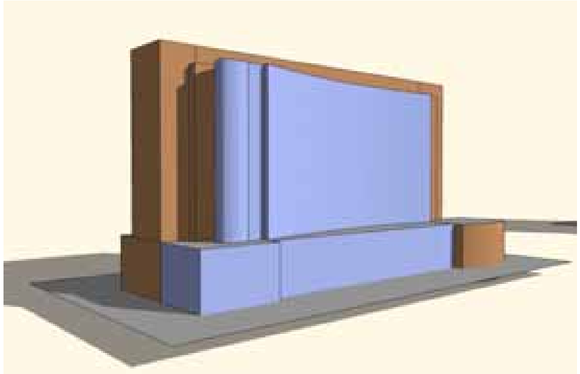
The addition of five levels will include several laboratories with especially sensitive laboratory equipment, requiring a vibration analysis. This analysis will need to be done in addition to a redesign of the gravity columns and foundations, as well as the lateral force resisting system. Architecturally, this addition will need to be designed to merge with the existing in terms of aesthetics, programming, and egress capacities.

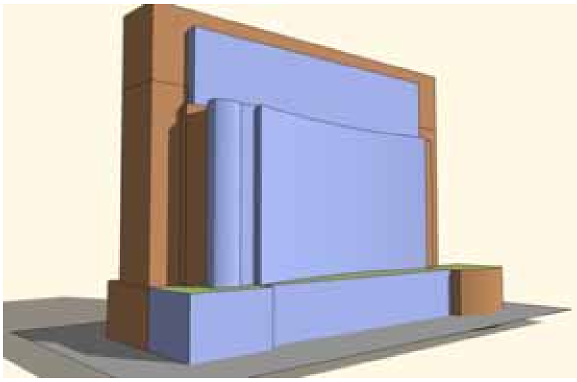
The green roof feasibility study will need to take into account the additional loading on the structure, especially in the green roof garden area, as well as a cost and schedule analysis. Both short-term and long-term benefits will need to be taken into account in the study to insure accuracy. Additionally, the garden area will need to be designed and incorporated into the existing building

Proposed Solution

The Addition

The addition of five levels of occupied space will be accomplished by cutting the existing building -seen below left- at the 12th floor penthouse level. Five additional levels of usable space will be inserted here creating floors 12-16. The penthouse, including its mezzanine level will be moved to the 17th floor, creating the new building form seen below, right.





Architecturally, the programming will remain similar to the existing tower floors. The large administrative offices will be eliminated to keep space for the laboratories and their support areas. A check will need to be made using the IBC 2006 to ensure that egress requirements are met.

The building form will continue up with a flat façade instead of the curve to accentuate the layering of the building. This will allow most of the existing structural framing to continue upward unchanged, with the exception of the cantilevered area, highlighted in blue.

There are several options which will be analyzed for achieving this cantilever, such as a hanging structure, moment connections, or continuous girders. The best option will be picked by a rough cost estimate of the different systems.

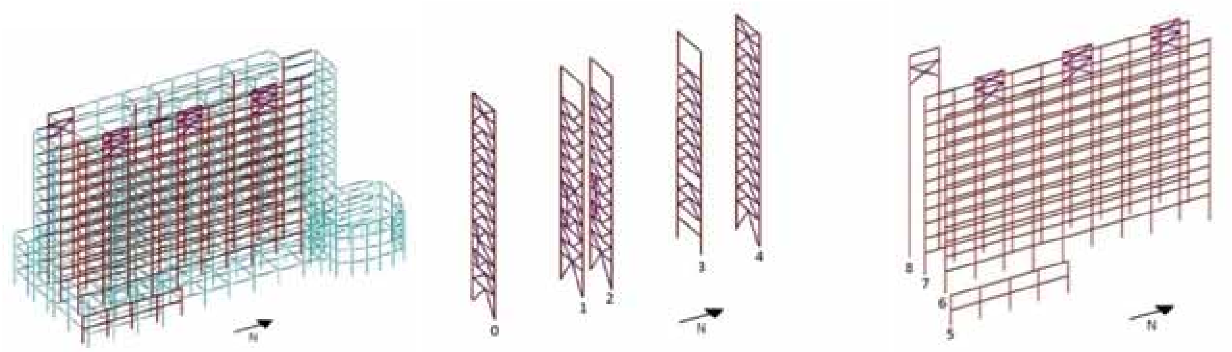
Special Bay

It is assumed that laboratories on each floor will contain especially sensitive lab equipment that needs to be designed for vibration control. Design Guide 11 will be used to perform a hand calculation of the structure's suitability for minimizing walking vibrations affecting sensitive equipment. This calculation will determine the type of equipment can be placed within the MHSC's laboratories.

Lateral System

The increased building height will require a close look at the building's lateral system in both directions. The existing braced frames in the E-W direction will have to be redesigned to meet the higher building loads, while the moment frames in the N-S direction will be redesigned as braced frames with the hope of increasing the design's efficiency, which will be determine by a comparison of takeoff data taken from the original and new design. The following image shows the existing frames and locations, with changes in the design described in the Structural Breadth section.

The design of this system will be accomplished by reanalyzing the already determined wind and seismic loads calculated through the ASCE 2007 design code. Next, the frames will be modeled and analyzed using RAM structural system. Hand checks and a comparison with another modeling program, STAAD, will be performed to verify the accuracy of results.



Gravity System

Although the focus of this report relies on analyzing the lateral system, a basic design of the gravity system will be done as well. RAM will be used to size the structure's columns for both the original and new designs. After verification with hand spot checks, these two designs will be used to obtain takeoff data to compare the systems and determine the efficiency and relative cost of the addition.

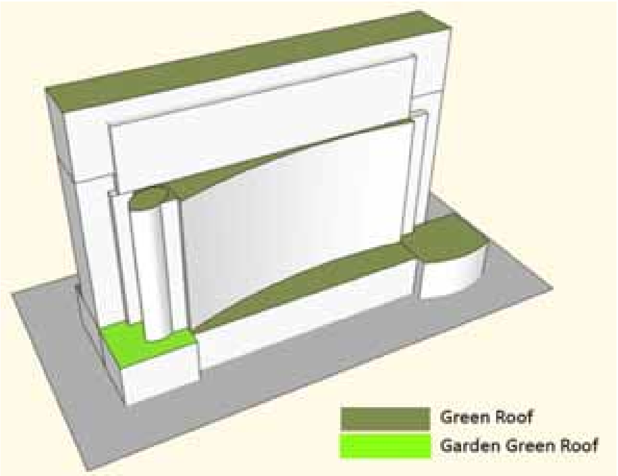
Green Roof

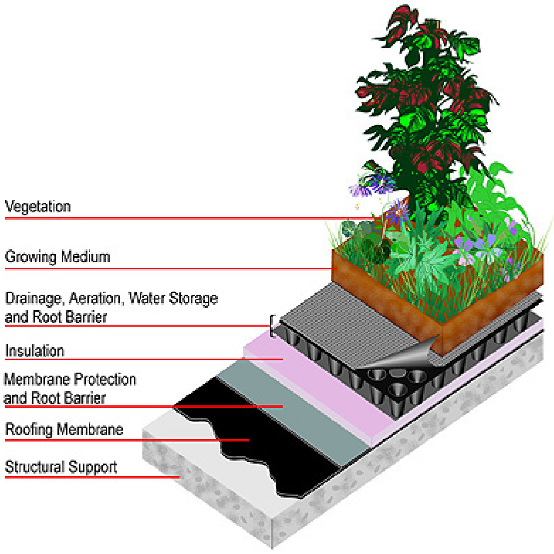
The green roof feasibility study will look into the application of intensive green roofs to most of the building's roof surfaces – see image below - with the 4th floor roof over the library designated as an extensive green roof garden for building occupant use. This would result in approximately 1400sf of usable space and 4800sf of green roof area. The soil thickness would be thicker in the garden area, but the basic construction of the two systems is similar (see below, right).

The garden will insure an enduring green space for the medical campus community, while the green roof will also benefits such as a higher R-value for thermal insulation, a reduction in the heat island effect, cleaner runoff water, and reduced runoff volume.

The green roof will require a redesign of the affected structural system, especially the system supporting the garden area, which will have a new 100psf live load.

These structural changes, as well the costs of purchasing the more expensive green roof system will result in higher first costs for the project, which can be overcome by long term benefits in energy and storm water treatment. Unfortunately a complete analysis would require extensive thermal modeling which is not included in the scope of this project; however, an initial cost estimate can begin to provide quantification for the pros and cons of this system.

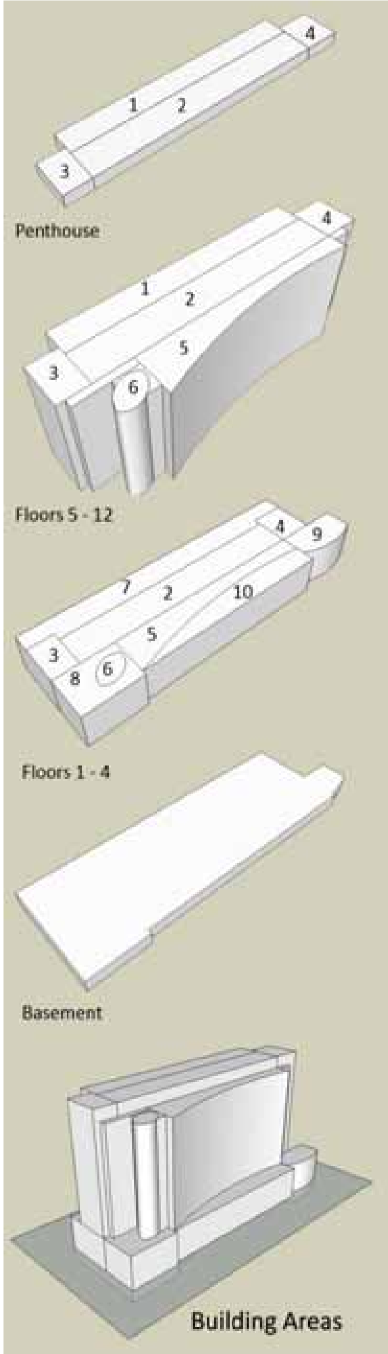




Building Information

Original Design: Architecture Overview

The Temple University Multipurpose Health Science Center is a new 480,000 square foot, $150 million addition to Temple University's medical campus north of Center City Philadelphia, Pennsylvania. The 13 story steel frame building contains offices, a café, a large library (building area 8), and three level atrium (area 10) with spaces primarily allocated to laboratories, classrooms, and their support facilities. The architectural features are governed by the concept of expressing internal functions externally, where the curved glass façade of the east elevation indicating the office areas (area 5) with the brick curtain-wall rectangle behind it expressing the laboratories and classrooms (area 1 and 2), and the oval tower expressing student meeting and studying spaces. Built on a previous parking lot, the steel framed building will connect to an existing building via a bridge and tunnel.



Building Areas

Original Design: Structural Description

Original Design: Foundations

The geotechnical survey justified a hybrid foundation system for the site, consisting of 40% footings and 60% caissons which terminate at solid bedrock, present at 30' to 50' depths. The upper layer of soil, between 19'to 35', consists of medium to very compact micaceous silty fines to coarse sands and varying gravel. Deeper soils, between 24' to 50', consist of more compact micaceous silty fines to coarse sands and gravel with borings terminating at intact mica bedrock. The building's excavation is between 78' to 83' with street level at approximately 100', placing the majority of the foundation between these two layers.

The expected column loadings are around 3,100 kips for the braced frame columns and about 1,000 kips for the majority of the columns. The higher bearing capacity of the lower layer of soil coupled with the required bearing of the capacity of the columns justified a hybrid system with braced frame columns resting on caissons.

The concrete used is 28-day, normal weight concrete at f’c=4000 psi for most areas, with the primary exception being concrete exposed to weather-for example, the truck ramp-which should be air-entrained, normal weight at f’c=5000. Reinforcing is grade 60.

Slab

The typical basement slab consists 6” of concrete over a vapor barrier and 4” of crushed stone, with 6”x6” W4.0xW4.0 WWF. The primary areas where exceptions occur are underneath the library, mechanical and electrical equipment, the loading docks, and areas underneath the auditorium. Slab thicknesses in these areas are either 8” or 12”.

View of structural systems

Footings

The shallow foundation system consists of steel columns sitting on concrete piers and footings, which are connected by grade beams. Footing thickness ranges from 1’4” to 4’4”, with most in the 1’10” to 2’4” range. Sizes generally range from 4’x4’ to 9’x9’.

Caissons

The deep foundation system consists of steel columns sitting on concrete piers, caps and caissons. Sixty-six of the one-hundred thirteen basement columns rest on these caissons, which vary in diameter from 36” to 96”. The top of the basement slab is at either 78’ or 83’ elevation, with caisson estimated bearing elevations ranging between 45’ to 70’, with the most around 60’.

Original Design: Columns

The framing system consists primarily of ASTM A992 Grade 50 rolled W-shapes with depths of 12" and 14". There are several 10" deep W-shapes in the basement through fourth floors and some HSS shapes in the auditorium. Sizes vary greatly with upper floor columns in the 100-120lb range, and lower floor columns in the 200lb range. The columns are spliced 4' above floor level and span two floors with lengths typically at 25' to 30'.

Original Design: Floor System

Slabs are typically 2.5", f'c=4,000 psi, NWC on 3"deep, 20 gage, galvanized composite steel deck, with 6x6-W2.9xW2.9 WWF. Decking is applied perpendicular to beams and parallel to girder. The primary exception is penthouse mezzanine and roof level, where the slab is thinner.

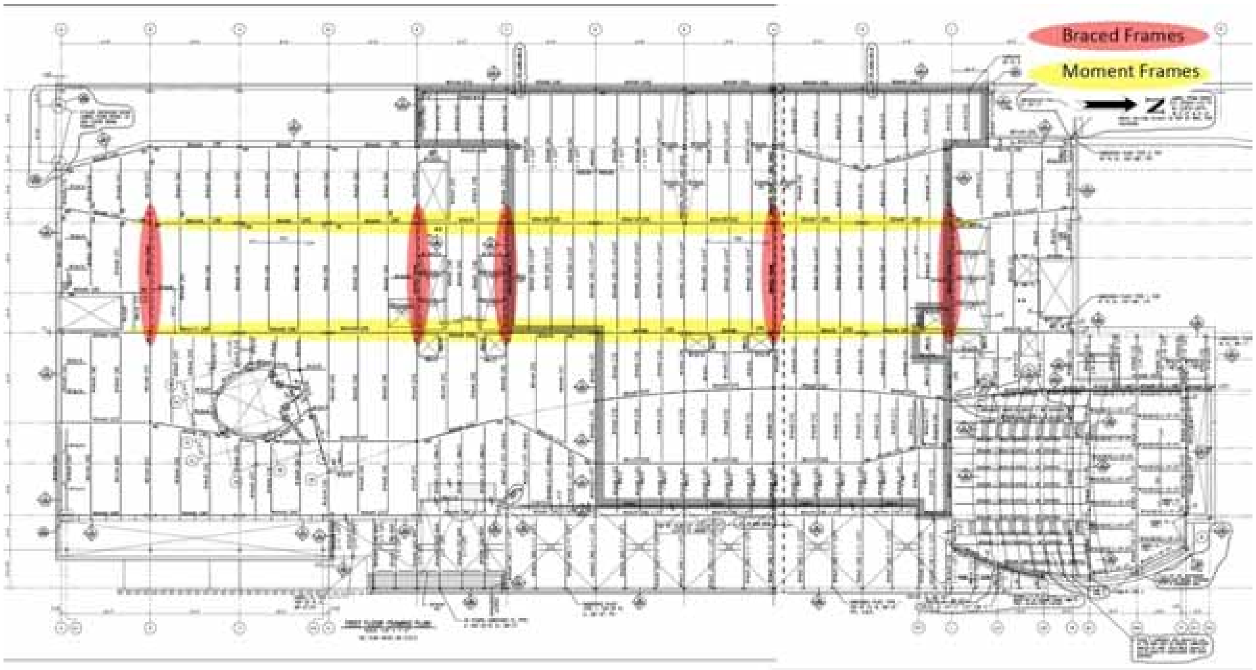
Bay sizes are typically approximately 30'x30', with special framing required to complete the curved façade and tower areas. Girders and beams are typically W24's, but vary considerably from one framing area to another (see description on page 12). The floor system is composite with approximately one stud per foot and is designed to be un-shored during construction.

This building also has three transfer trusses which take column point loads from above and redistribute them to offset columns at a lower level. Two of these trusses are located between the first and second floors, are 15'4" deep, and span 46.5' in order to clear space for the loading dock below. A third truss is located between the 5th and 6th floors, is 14'8" deep, and spans 62' in order to relocate columns for corridors on lower levels.

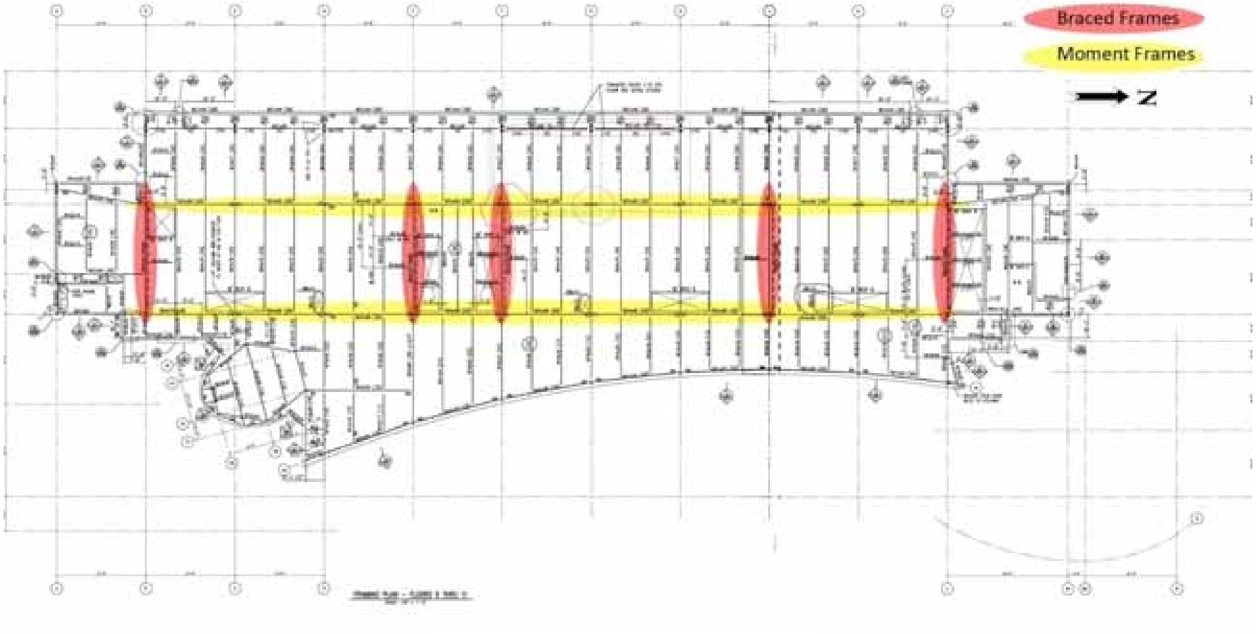
Original Design: Lateral System

Due to the slender shape of the building lateral resistance is primarily needed in the East-West direction (see image below). This resistance is provided by four sets of concentric braced frames which run the full height of the building. A review of detailed drawings of the connections did not indicate the use of moment connections, indicating regulary braced frames. The vertical members range from W14x109 at the top to W14x550 at the bottom. Horizontal members are typically W24x55 but range from W21x44 to W27x161. Diagonal members range from W10x49 within the upper four floors to W12x190 at the bottom. All of these frames are supported by caissons.

North-South wind loads are taken by moment frames running along the length of the building (see images below) and are also supported by caissons. Secondary North-South brace appear at the penthouse levels, with an additional set of secondary moment frames appearing along the first and second floor library façade.



First Floor Framing Plan



Tower Levels Typical Framing Plan

[ARCHITECTURAL BREADTH]

Introduction – Schematic Design – Design Development – Life Safety Check – Conclusions

Introduction

The Multipurpose Health Science Center (MHSC) benefits from its urban location, incorporation to the Temple University medical campus, and proximity to downtown Philadelphia. Unfortunately, this location comes with a lack of green space and minimal room to expand facilities. Considering the constant need for university expansion, Temple University has expressed the need for design services in determining the feasibility of adding to the existing MHSC design.

After reanalyzing their needs, the university has decided to investigate the addition of at least 120,000 SF of laboratory, classroom, and office space to the existing MHSC design. It has been decided that the best way to expand is vertically since the local zoning is C3, which does not limit building heights. Vertical expansion is certainly the best option considering the constriction of surrounding residential neighborhoods and the Temple Campus, as well as the desire to maintain the green space available with the current design. They have also requested an investigation into the addition of a green roof and garden to increase the availability of green space to the building occupants, as well as the medical campus community.

These programmatic requirements are met through the design outlined in the Architectural Breadth section, with the green roof requirements analyzed and designed in the Green Roof Breadth section. The goal is to create a design which will meet the new programming requirements, while using the following technical and creative guidelines:

- Consider the scale of the surrounding site and neighborhood.
- Consider the original design's massing, programming, cladding, and preexisting design philosophy.
- Check life safety and egress requirements per the IBC 2006.
- Optimize the design to create the most efficient structural system.

These goals will then be applied and developed to a schematic design for the building addition, with a focus on massing and fulfilling basic program requirements. Once these criteria have been met, the design will be refined to optimize structural considerations and create more detailed floor plans, and framing layouts. Lastly, the life safety and egress requirements will be checked.

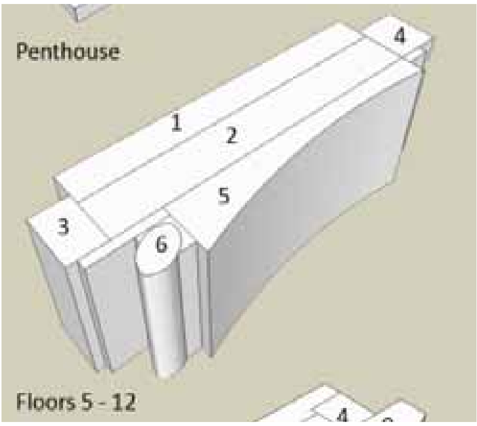
Schematic Design

Three key areas will have the largest affect on the schematic design: the surrounding site and neighborhood, the preexisting design philosophy and meeting the program requirements.

The building site is currently a parking lot wedged between residential neighborhoods and Temple's medical campus, the former consisting of low-rise row-homes and the latter of mid-rise university buildings. The aerial image of the site combined with the medical campus map gives a sense of the horizontal scale of the site, while the images of campus and surrounding residential buildings give a sense of vertical scale (right, and below). It is important to address these issues in building's massing, which the original does by stepping back the façade plane from the street edge at the 4th and 12th levels, as demonstrated by the image on the following page.

The second issue to take into consideration is the preexisting design philosophy, which is the idea of expressing interior functions externally. This is accomplished with massing and materials. For example, areas 1 and 2 of the tower section labeled below contain laboratories which are externally clad in brick, 3 and 4 are stairwells which jut out of the structure, and the office space of area 5 is expressed by the geometry and glass cladding of the curved façade.

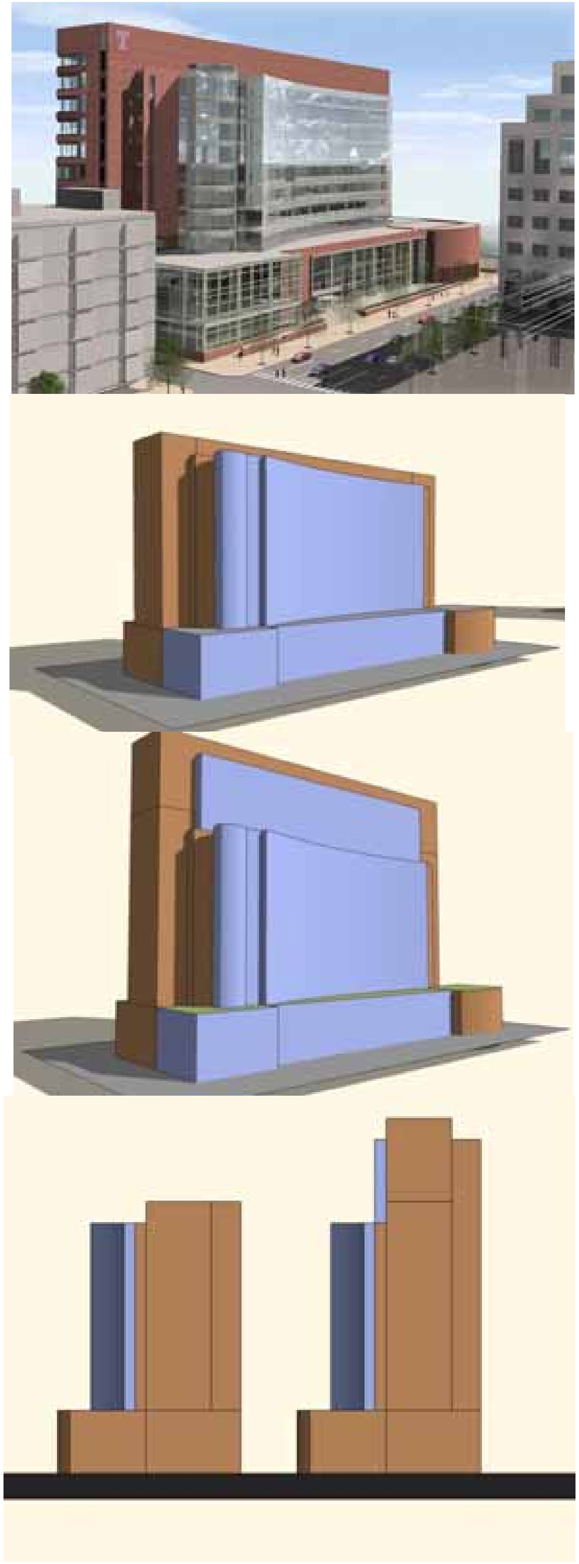
Residential buildings, above – Campus buildings below.



Campus map left - Aerial photograph, right

Lastly and most importantly, the additional programming requirements need to be met for the owner, who has specified a need for approximately 100,000 additional square feet to the original 480,000. The floor area of the typical tower level depicted on the previous page is approximately 28000 SF. With approximately 4260 square feet of this dedicated to corridors, stairwells, and elevators, the design obtains 85% efficiency in its layout, where efficiency is defined as the ratio of non-rentable square footage to rentable square footage. A good rule of thumb for adequate efficiency is 80%. This square footage is the maximum for each level added to the building and will control the number of levels added to the building.

These footage requirements combined with the consideration of stepping back the façade and the philosophy of expressing interior functions externally are the guiding principles for the massing and basic plan for the building. If the existing plan is utilized for an additional five levels without taking the other guiding principles into account, 118,700 SF of rentable space is obtained, which is a bit over the specified area. However, it is proposed to continue the step-back pattern by stretching the tower section vertically while excluding the curved front façade (see image below). This concept also fulfills the guiding design philosophy of expressing internal function externally, since the stairwell section and primary lab spaces are clad in brick and are geometrically separated from the other areas. The new floor area is smaller at 25,200 SF, with 20,940 SF of that total space rentable. This yields a total rentable square footage of 104,675 SF at 83% efficiency.



Images top to bottom: Original design rendering (image from Temple's website), original design mass model, new design mass model, comparison of step-backs with the new design on the right.

Design Development

Once the basic massing and planning criteria were met, the design was further developed to create more detailed floor plans, framing layouts and cladding choices, as well as optimize structural layouts.

A typical tower layout appears below, with laboratories labeled in blue, corridors and stairwells in yellow, office spaces in red, and building areas that are below in gray. The massing of the addition calls for the vertical continuation of the floor area without the curved façade, which encompasses the laboratory, stairwells and corridor areas. Although this does not include extra office area, it is important to note specific tenant-fit-out and interior designs were not within the scope of this project; therefore it is acceptable to use a modified form of the existing space design to furnish a typical floor layout for the addition levels, depicted by the new typical tower floor plan on the following page.

Original typical tower floor plan.

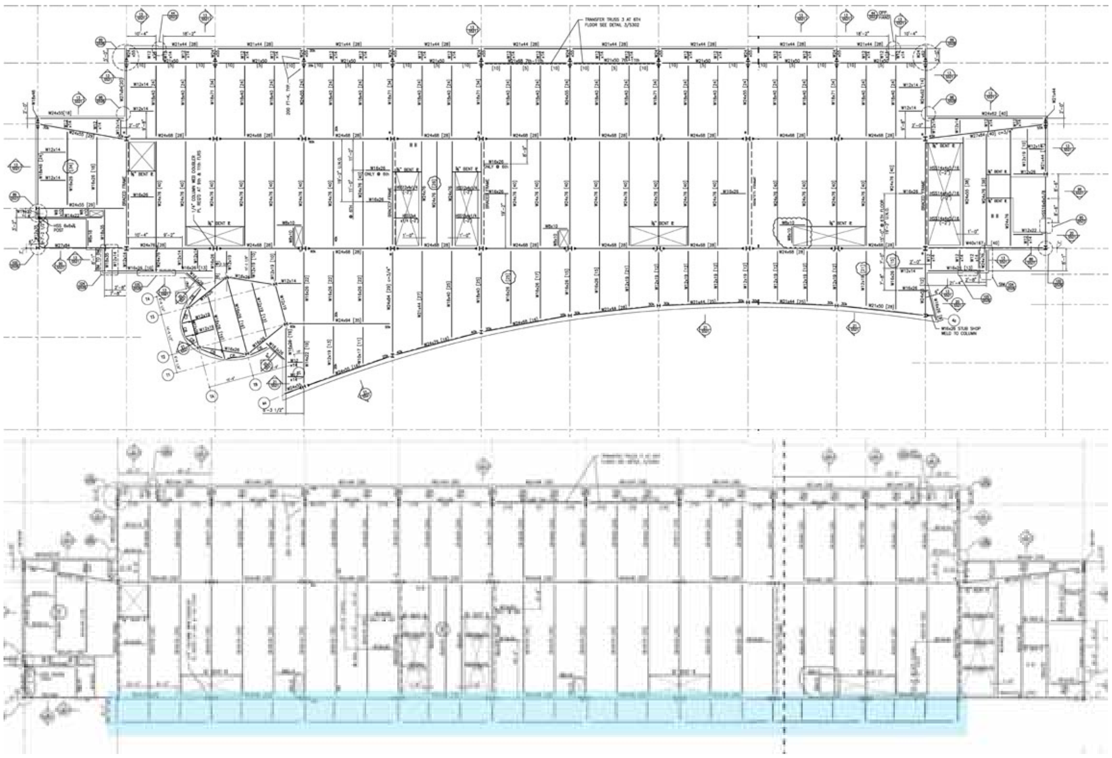
An important issue for designing the floor layouts was the consideration of the mechanical system. The original penthouse still remains on the top of the building, with the five added levels inserted at the old penthouse level, here creating the usable floors 12-16. The new Penthouse is at the 17th floor level. All five of the additional levels could be used as laboratory space, which place high demands on heating, cooling, and ventilation capacities. Since all of the penthouse space already has HVAC equipment placed there, or has space already allocated for future required equipment, new space for the addition's HVAC equipment had to be found. A rough estimate of the square footage required was made by comparing the square footage required by the existing HVAC units and the square footage serviced by those units. This was then compared to the new square footage added to the building to obtain a rough estimate of the square footage that needed to be allocated to equipment. It was determined that each of the 3 existing AHU's use 640 SF and serve 5 existing floors of laboratory space; therefore another 3 AHU's will most likely be needed. The blue area above the stairwell on the left part of the floor plan is currently a break out space of 700 SF which could be potentially used for an HVAC system (see are boxed in red on the layout below). The structural design will need to account for the increased mechanical equipment loading of psf. This will be detailed again in the Structural Breadth section.

New typical tower floor plan, with proposed mechanical area boxed in red.

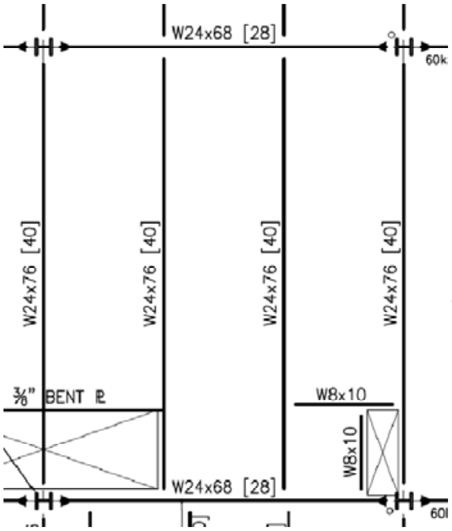
The specific programming design came hand in hand with an analysis of the existing structural layout, with the goal of minimizing potential negative effects that a change in architectural layout would have on the strength design and construction process. The simplest way to achieve this was to maintain the existing framing geometries. Had this been done otherwise, shifted columns would require special structural systems like transfer trusses to deal with the shifted load, and altering beam and girder layouts would have increased construction time. These would add time and money to the project.

Another critical issue was to insure that the addition would be suitable for future laboratory spaces containing sensitive equipment which can be potentially sensitive to vibration caused by walking excitation. Later, in the Structural Depth section, it will be shown through analysis that the existing system is indeed suitable for most types of laboratory equipment.

Therefore, the floor system design and framing layout was maintained for the additional levels, eliminating potentially large cost increases. The only items that would require significant redesign are the gravity columns, and a cantilevered portion of the framing layout, depicted in blue on the new layout on the following page. Several choices will be considered in designing this area, such as a hanging structure, moment connections, or continuous girders. The best option will be picked by a rough cost estimate of the different systems, outlined in the Structural Breadth section.



Above: Original gravity framing layout. Below: the new layout, with the cantilevered section highlighted in blue.

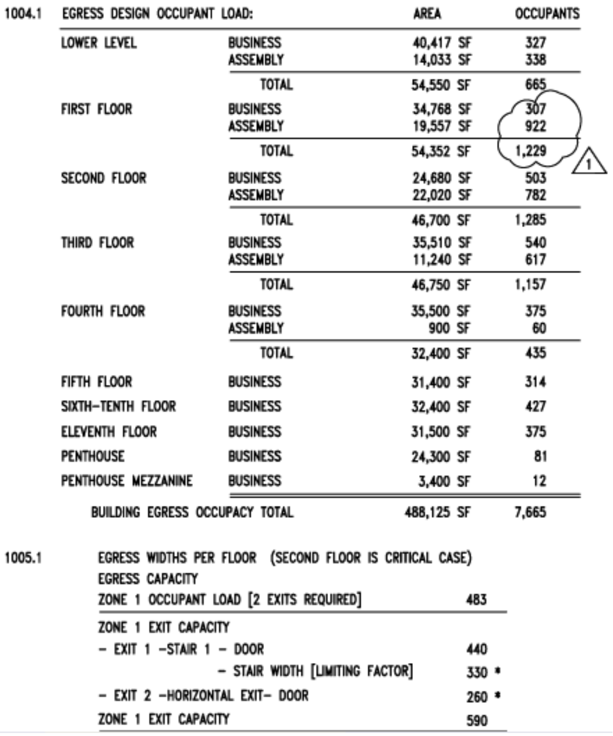


Typical floor framing system, with composite deck

Life Safety Check

A critical issue in any design is to meet the egress and life safety requirements for the building. The MHSC occupancy type is mixed business and assembly for the lower floors, and business for the upper floors, including the levels of the building addition. Egress capacities were checked using the chapter 10 of the IBC 2006 to insure that the building addition would still meet the life safety requirements. Exit width requirements are determined per floor; however, the final exit width must be sized for the greatest occupancy load.

This means that large floor area and seating capacity of the existing auditorium and library –a total occupancy of 1285 persons - at the second story controlled the widths of egress passageways and doors; however exits above this level may be smaller if the occupancy load is smaller.



1004.1	EGRESS DESIGN OCCUPANT LOAD:		AREA	OCCUPANTS
	LOWER LEVEL	BUSINESS	40,417 SF	327
		ASSEMBLY	14,033 SF	338
		TOTAL	54,550 SF	665
	FIRST FLOOR	BUSINESS	34,768 SF	307
		ASSEMBLY	19,557 SF	922
		TOTAL	54,352 SF	1,229
	SECOND FLOOR	BUSINESS	24,680 SF	503
		ASSEMBLY	22,020 SF	782
		TOTAL	46,700 SF	1,285
	THIRD FLOOR	BUSINESS	35,510 SF	540
		ASSEMBLY	11,240 SF	617
		TOTAL	46,750 SF	1,157
	FOURTH FLOOR	BUSINESS	35,500 SF	375
		ASSEMBLY	900 SF	60
		TOTAL	32,400 SF	435
	FIFTH FLOOR	BUSINESS	31,400 SF	314
	SIXTH–TENTH FLOOR	BUSINESS	32,400 SF	427
	ELEVENTH FLOOR	BUSINESS	31,500 SF	375
	PENTHOUSE	BUSINESS	24,300 SF	81
	PENTHOUSE MEZZANINE	BUSINESS	3,400 SF	12
	BUILDING EGRESS OCCUPACY TOTAL		488,125 SF	7,665

1005.1	EGRESS WIDTHS PER FLOOR (SECOND FLOOR IS CRITICAL CASE)	
	EGRESS CAPACITY	
	ZONE 1 OCCUPANT LOAD [2 EXITS REQUIRED]	483
	ZONE 1 EXIT CAPACITY	
	– EXIT 1 –STAIR 1 – DOOR	440
	– STAIR WIDTH [LIMITING FACTOR]	330 *
	– EXIT 2 –HORIZONTAL EXIT– DOOR	260 *
	ZONE 1 EXIT CAPACITY	590

Occupancy loads given by the MHSC design drawings.

The existing tower levels are business occupancy with a square footage of 2800 SF, which is greater than the addition levels which are 25,200 SF, so the existing floor layouts control the egress capacity requirements above the base level. Table 1004.1.1 dictates an occupancy load of 100SF per occupant; therefore the existing levels have 280 occupants, and the new levels have 252 occupants.

Since the added and existing tower level occupancy loads are less than 500, only two exits are required for each level per IBC Table 1019.1 below. This means that having two exit stairwells for the entire section is sufficient to meet egress requirements.

TABLE 1019.1
MINIMUM NUMBER OF EXITS FOR OCCUPANT LOAD

OCCUPANT LOAD (persons per story)	MINIMUM NUMBER OF EXITS (per story)
1-500	2
501-1,000	3
More than 1,000	4

Conclusions

The goal of the architectural breadth was to create a design that met the program requirements while taking several other design considerations into account. The schematic design successfully provided the programmatic requirements of increased square footage while taking into account the scale of the neighborhood and Temple's campus. Combined with the inclusion of the design philosophy of expressing internal functions externally, a building massing with a step-back form was developed which accommodated the square footage required.

This massing was further developed into detailed architectural and structural layouts which met the 80% efficiency rule of thumb, optimized the structural design, and took special considerations such as vibration issues and mechanical equipment space allocation were taken into account.

Lastly, the building's egress loads and capacities were analyzed using the IBC 2006 requirements. It was found that the original design controlled the number and width of exits required; therefore eliminating the need for additional life safety design, and insuring the well-being of building occupants.

[GREEN ROOF BREADTH]

Introduction & Goals – Schematic Design – Roof Cost Estimate

Introduction & Goals

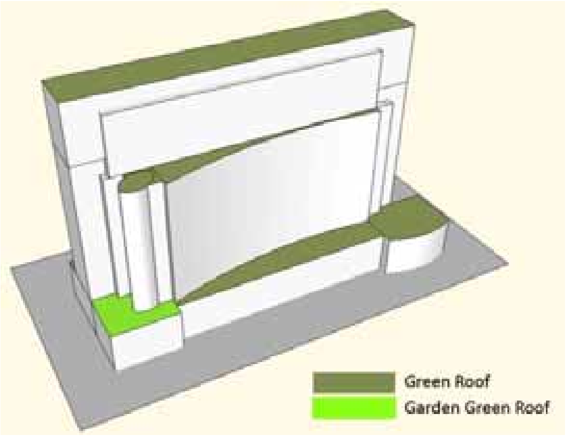
The great urban location of the new MHSC comes with a primary shortcoming: lack of green space. The building site -depicted to the right is cramped between Temple's campus and the surrounding residential neighborhoods with no roof for green expansion. The original decision makers of the original design had the foresight to include green space on the ground level for the new building; however the owner has expressed a new interest in expanding this green space and has requested a feasibility study into the addition of a green roof.

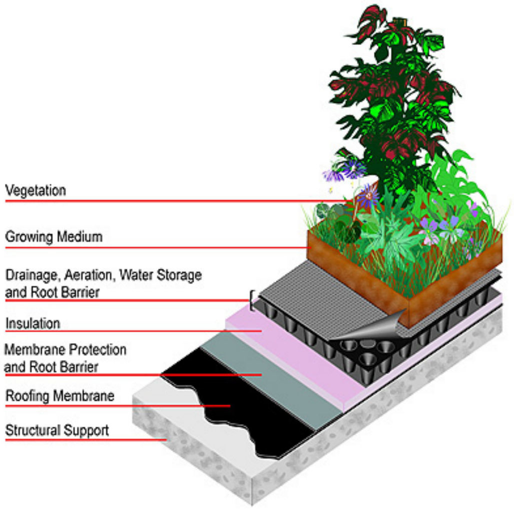
The green roof will be a dual intensive and extensive system, such that an accessible garden above the library will provide usable green space, containing a patio, seating area and plantings to provide reading space for the library users. This 1400sf garden will sit on the 4th floor roof. Users will come from the basement, 1st, and 2nd levels to the roof level to work and study, and will most likely use the elevators. There is a chance that 2nd level users will use the S. Stairwell as well. All of these users will have access to the roof through doors on either side of the oval tower, which also connect to the 4th floor corridor

The remaining 4800 sf of green roof areas will be extensive green roofs with small plantings. The potential benefits include to a higher R-value for thermal insulation, a reduction in the heat island effect, cleaner runoff water, and reduced runoff volume. A Philadelphia based green roof designer and contractor, "Roofscapes, Inc.," provides an example of the type of stormwater runoff reduction offered by these greens. Even the thinnest varieties, such as a 3.25" roof in Reading, Pennsylvania sees a 61% attenuation rate of runoff for a 2 year rainfall event, and a 47% percent attenuation rate for a 2 year rainfall event

The extensive green roof areas, located throughout the building, have multiple access points along the roof line.

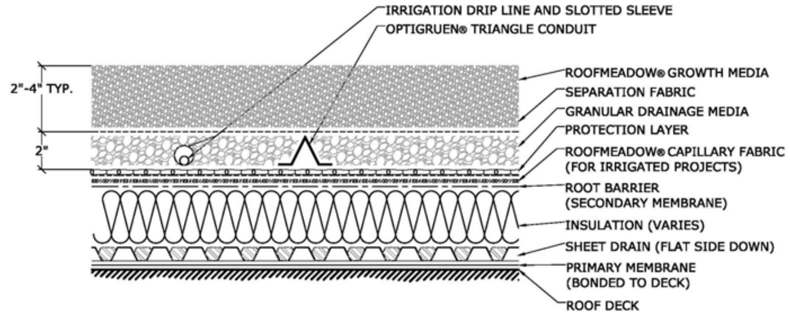
Unfortunately a complete analysis would require extensive thermal modeling, long-term energy usage calculations and rainfall analyses which are not included in the scope of this project; however, an initial cost estimate can begin to provide quantification for the pros and cons of this system.





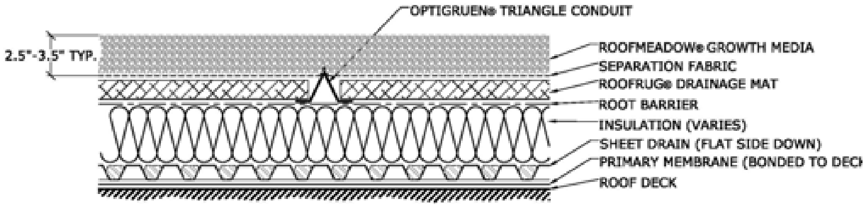
Schematic Design

The 4th floor library addition will be an intensive green roof garden, with approximately 1400sf of usable space. The system picked for this area is the “Savanna” system, which is a green roof system designated by a Philadelphia based green roof designer and contractor, “Roofscapes, Inc.” This type of roof is typically 4-6” deep, weighs 27-40 psf when saturated, and supports sedums, meadow grasses and meadow perennials. This particular project will use the 6” version to allow the largest possible variety and size of plants. A section of the system is presented below, from which the cost estimate is based.



S-3 XX ROOFMEADOW® SAVANNAH - OPTION C
NTS PRM CONFIGURATION

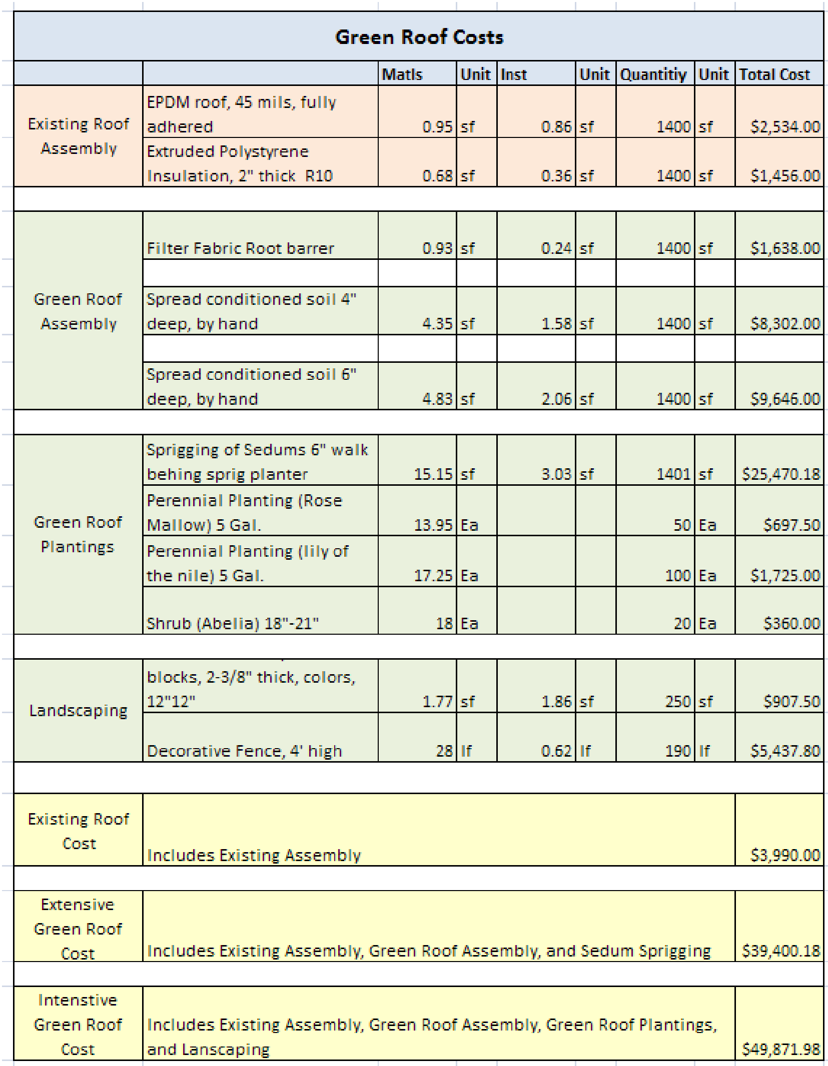
The remaining 4800 sf of roof areas in the building will be covered with the “Flower Carpet” system, which is typically 3-5” deep with a saturated weight of 20-27psf. This roof is meant mostly for the energy and runoff benefits, not accessibility, so plant varieties are limited to sedums and small herbs. A section of the system is presented below, from which the cost estimate is based. Neither of these roofs require irrigation.



FC-3 XX ROOFMEADOW® FLOWER CARPET - OPTION C
NTS PRM CONFIGURATION

Roof Cost Estimate

RS Means Assemblies, and RS Means Site Work & Landscape Cost Data were used to perform a cost estimate of the garden roof which compares the cost of the various roof systems. The totals show that the cost is quite significant in comparison to the existing roof system. This increase is primarily controlled by the sprigging of sedums on the roof which needs to be done for both systems; however, the shallower soil depth, lack of bigger plantings, and lack of landscaping features makes the extensive green roof over $10,000 dollars cheaper than the intensive green roof. The total cost of $49,871 for the intensive green roof seems like a hefty sum, but it is only 0.03% of the total $150 million budget. Considering the life-time aesthetic benefits and reductions in storm water treatment and energy bills, this addition may be well worth the initial high cost.

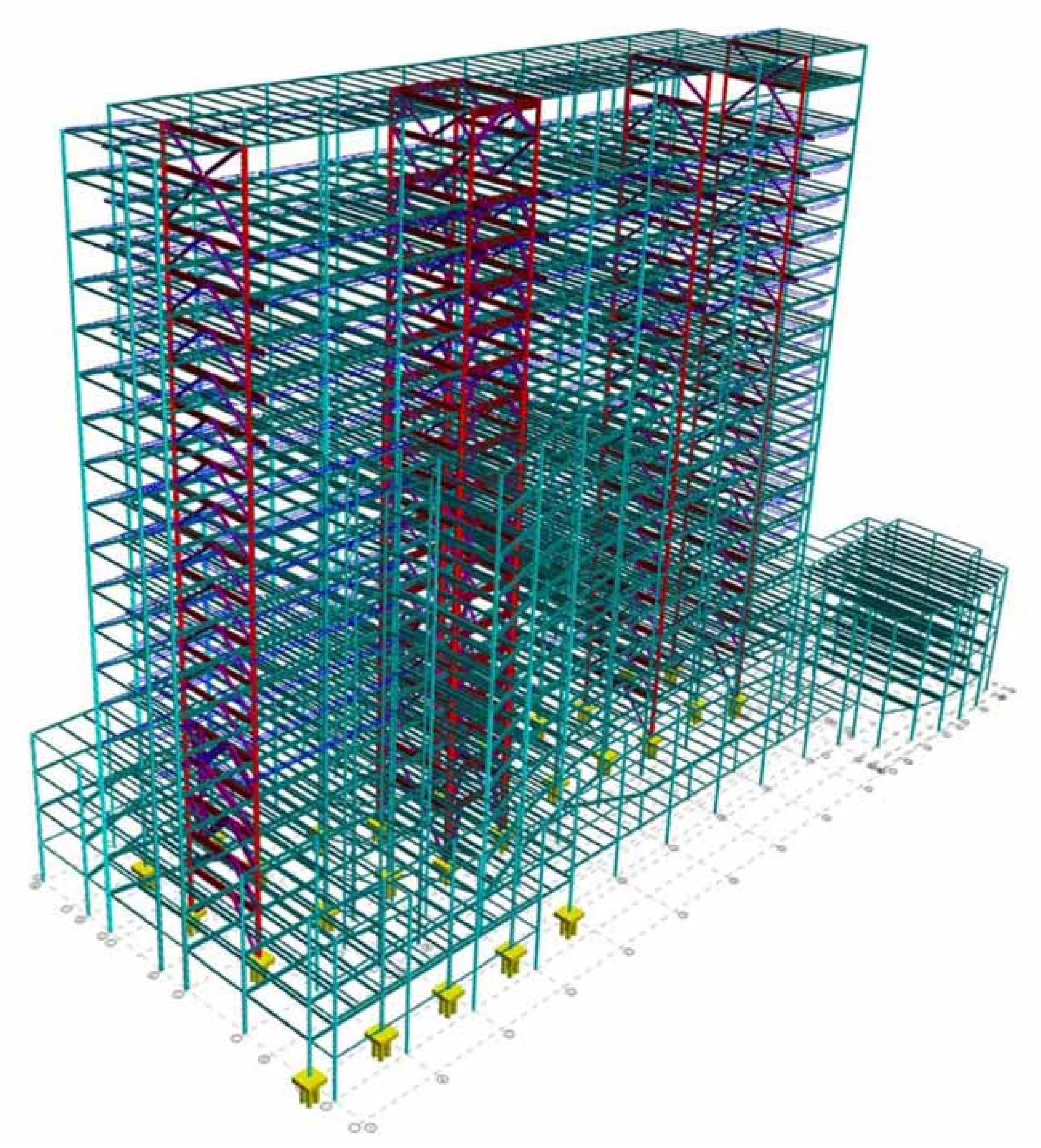


Green Roof Costs

		Matls	Unit	Inst	Unit	Quantitiy	Unit	Total Cost
Existing Roof Assembly	EPDM roof, 45 mils, fully adhered	0.95	sf	0.86	sf	1400	sf	$2,534.00
	Extruded Polystyrene Insulation, 2" thick R10	0.68	sf	0.36	sf	1400	sf	$1,456.00
Green Roof Assembly	Filter Fabric Root barrer	0.93	sf	0.24	sf	1400	sf	$1,638.00
	Spread conditioned soil 4" deep, by hand	4.35	sf	1.58	sf	1400	sf	$8,302.00
	Spread conditioned soil 6" deep, by hand	4.83	sf	2.06	sf	1400	sf	$9,646.00
Green Roof Plantings	Sprigging of Sedums 6" walk behing sprig planter	15.15	sf	3.03	sf	1401	sf	$25,470.18
	Perennial Planting (Rose Mallow) 5 Gal.	13.95	Ea			50	Ea	$697.50
	Perennial Planting (lily of the nile) 5 Gal.	17.25	Ea			100	Ea	$1,725.00
	Shrub (Abelia) 18"-21"	18	Ea			20	Ea	$360.00
Landscaping	blocks, 2-3/8" thick, colors, 12"12"	1.77	sf	1.86	sf	250	sf	$907.50
	Decorative Fence, 4' high	28	lf	0.62	lf	190	lf	$5,437.80
Existing Roof Cost	Includes Existing Assembly							$3,990.00
Extensive Green Roof Cost	Includes Existing Assembly, Green Roof Assembly, and Sedum Sprigging							$39,400.18
Intensive Green Roof Cost	Includes Existing Assembly, Green Roof Assembly, Green Roof Plantings, and Lanscaping							$49,871.98

[STRUCTURAL DEPTH]

Introduction – Loads & Codes – Lateral Design – Gravity Design – Vibration Check– Conclusions



Introduction

The addition of five levels to the MHSC will require significant changes to the existing structural system; however it is the intent of the designer to create a system that will not only meet the structural design criteria established by the architectural program, but to create a design that is efficient. The increased size of the building will result in significantly higher wind, seismic, and gravity loading for which the entire structural system will have to be reanalyzed and redesigned. These loading conditions as well as the design codes governing them will be discussed immediately following this section. After the loading conditions are described, the lateral and gravity system redesign results and vibration check analysis for the appropriateness of the floor system for sensitive laboratory equipment will be presented.

The first goal of the lateral system design is to redesign the EW braced frames to handle the additional loading, and to do so in a manner that will not heavily increase price. The second goal is to replace the existing NS moment frames with braced frames to achieve a more economical design for the framing system in this direction. The process for this design will begin with the creation of a RAM structural model. After the structure is sized for gravity loading, the RAM Frame module will be used to size the lateral frames for strength and serviceability. STAAD will be used to confirm the RAM analysis, and hand calculations – described in the following paragraphs - will be used to double check both of those results.

Once the frames have been designed, modeling data from RAM, as well as the MHSC design specifications will be used to perform a preliminary lateral foundation design, as part of the lateral analysis. This will be a schematic design based off of the end bearing capacity. These foundation sizes will then be compared to the original design.

The gravity system design will be finalized once the lateral system design is completed, with results based off of the RAM model. This section will also include the approach used to choose the framing system for the cantilevered area described in the Architectural Breadth section. Hand calculations based off of tributary area will be used to manually verify the column sizes. Another form of verification will take place by comparing the column sizes of the existing original design, the modeled original design, and the new design. This comparison will make evident any discrepancies between the existing original design and the computer models, and provide a check for the validity of the computer models results.

The lateral and gravity design sections will contain their own conclusion section summarizing findings and including an economics analysis which compares the original design to the new design. A direct comparison can be done of the two designs, even if there is a slight discrepancy between the computer-model based designs and the existing original design. This is because any differences that occurred between the modeling and the existing design will be present in both of the modeled designs.

The final step will be to perform a vibration analysis of the existing floor system. The MHSC contains many laboratory spaces with sensitive equipment, which may not function properly when exposed to vibrations caused by walking excitation. The Design Guide 11 will be used to perform this analysis to determine the suitability of the floor system for various laboratory applications.

Loads & Codes

Applicable Codes

Below are listed the codes used by the original designers.

- IBC 2003 (Philadelphia building code)
- ASCE7-02
- Concrete:
    - ACI 318 "Building Code Requirements for Structural Concrete"
    - ACI 316 "Manual of Standard Practice for Detailing Concrete Structures"
    - ACI 301, 302, 304, 305, 306, 308, 311, 318, 347
- Steel:
    - AISC "Specifications for Design, Fabrication and Erection of Structural Steel for Buildings"
    - AISC "Code of Standard Practice for Steel Buildings and Bridges"
    - American Welding Society (AWS) D1.1 "Structural Welding Code – Steel."
    - American Welding Society (AWS) D1.1 "Structural Welding Code – Steel."
    - ASTM A6 " General Requirements for Rolled Steel Plates, Shapes, Sheet Piling, and Bars for Structural Use."
    - ASTM A325 "Specifications for Structural Joints"
    - Steel Deck Institute "Design Manual for Composite Decks, Form Decks, and Roof Decks"

The following codes and design guides were used for this design:

- IBC 2006
- ASCE7-05
- Steel Design Guide Series 11: Floor Vibration Due to Human Activity
- United Steel Deck design manual and catalog of products
- AISC Steel Construction Manual
- RS Means Site Work & Landscape Cost Data
- RS Means Assemblies Cost Data

Live & Dead Loads

Live Loads	
	Load (psf)
High Density Storage Area	300
Office/corridor	100
Library	150
Penthouse Floor	150
Penthouse Mez	100
Roof	30
Roof-Garden	100
Slab on Grade	150
Truck Drive Aisle	300
Dead Loads	
	Load (psf)
Decking	50.1
Girders & Beams	7
MEP	20
Green Roof-Intensive	40
Green Roof-Extensive	20
Snow Loads	
	Load (psf)
Flat-roof snow load	22
Snow Exposure Factor	0.9
Snow Load Importance Factor	1.1
Thermal Factor	1

The loads in the following tables were determined by reviewing the building documents and noting the loads used by the original designers, who based their loading off of the IBC 2003, the adopted building code of Philadelphia, Pennsylvania. Most areas in the building used the office corridor loading, most likely to cover the demands of heavier lab equipment and furnishings. The 150psf penthouse loading is appropriate to cover the weight of mechanical equipment.

Design dead loads were not presented in the building documents, so several references were used to make assumptions. This loading was then used for determining building weight for seismic calculations. The member loads were determined by taking the average unit weights of members in a typical bay and dividing by that bay's area, while the United Steel Deck design manual gave the decking load. MEP dead loads are often taken to be 15 psf, but research into ASCE 7-05 Minimum Design Dead Loads and material unit weights for plumbing determined that 20 psf would be a more appropriate number for a building dominated by laboratory space.

Once the intensive and extensive green roof systems were picked, unit weights for the two systems based off of maximum saturated soil conditions were determined by reviewing specification information provided by a Philadelphia green roof designer, Roofscapes, Inc. (www.roofmeadow.com). These unit weights are used for the dead loads, while the IBC 2006 requires for a 20psf live load for roofs (including landscaped roofs); however the original design documents call for 30 psf live load roof loading, which was the controlling number used for the redesign. Below is a summary of the loading for the two green roof types:

- Intensive green roof:
  - 100 psf assembly live load
  - 40 psf green roof dead load
- Extensive green roof:
  - 30 psf live load prescribed by building documents (IBC specifies 20 psf for landscaped roofs)
  - 40 psf green roof dead load

Wind Loads

Wind lateral loads were based off of the ASCE7-05-6.5 Analytical Procedure. Basic assumptions about the building and site - listed below- were used to determine the gust factor G_f, velocity pressure q_z, and final design wind pressures as seen in the table below and the wind diagram on the following page. The large seating capacity of the library exceeding 300 persons resulted in the higher occupancy III classification, which resulted in the 1.15 importance factor. The gust factor value essentially did not change (compared to the original .803 value), when comparing the wind calculations for the new versus the original design,

Distribution was accomplished by taking the wind pressures at each level and the respective portion of the individual story height affected by the wind pressure and multiplying the length of the floor level to obtain a story shear. Lastly the individual story shears were summed to obtain a base shear (See page following the wind diagram). These values were directly imputed into RAM correctly create the model wind loading case, in other words the RAM generated wind load values were not used.

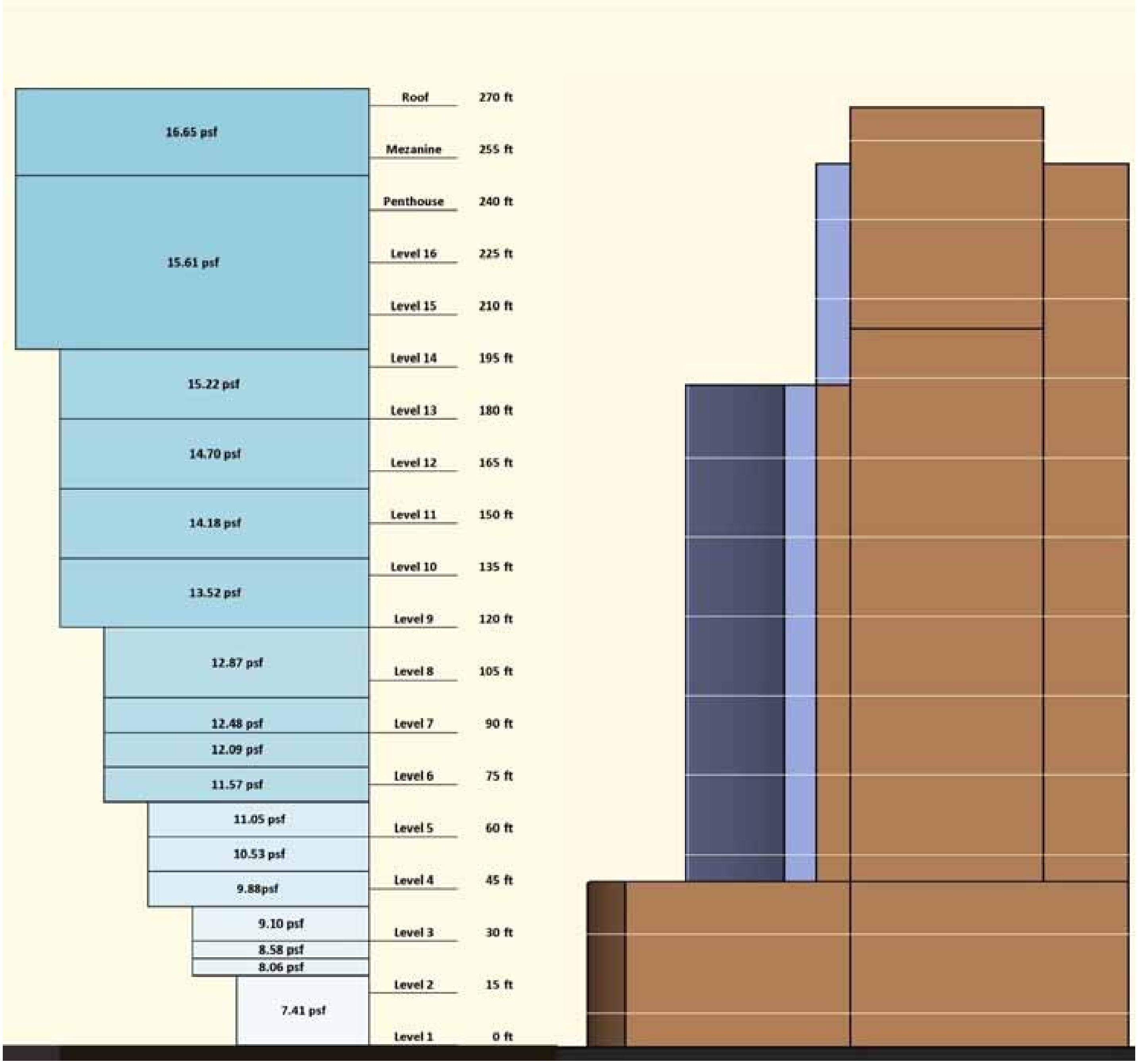
The increased surface are of the added levels significantly increased the total wind load, resulting much higher base shears than the original, which will require a significant redesign of the lateral system. The base shear was 2322 kips in the East-West direction to the large area of those elevations (1525 in the original design), while the base shear for the thinner North-South direction was only 714 kips (506 kips in the original design). The final wind pressure calculation appears in the table below next to the guiding assumptions; however, full calculations can be found in the appendix.

Assumptions

Basic wind speed	-	90mph
Exposure category	-	B
Importance factor	-	1.15
Occupancy type	-	III
Topographic factor	-	1
G_f	-	0.802
C_{pi}	-	±0.18

Windward Pressure (psf)

Height	kz	qz=20.269*kz	p =	$q*G_f*C_p$	± qi*(GCpi)	=	p (psf)
0-15	0.57	11.55		7.41	0	=	7.41
20	0.62	12.57		8.06	0	=	8.06
25	0.66	13.38		8.58	0	=	8.58
30	0.7	14.19		9.10	0	=	9.10
40	0.76	15.40		9.88	0	=	9.88
50	0.81	16.42		10.53	0	=	10.53
60	0.85	17.23		11.05	0	=	11.05
70	0.89	18.04		11.57	0	=	11.57
80	0.93	18.85		12.09	0	=	12.09
90	0.96	19.46		12.48	0	=	12.48
100	0.99	20.07		12.87	0	=	12.87
120	1.04	21.08		13.52	0	=	13.52
140	1.09	22.09		14.18	0	=	14.18
160	1.13	22.90		14.70	0	=	14.70
180	1.17	23.71		15.22	0	=	15.22
200	1.2	24.32		15.61	0	=	15.61
250	1.28	25.94		16.65	0	=	16.65



Above: Wind loading diagram

Detailed Calculations - E-W								
				Windward	Leeward	Windward kips	Leeward kips	Total Kips
Floor	Height	Elevation	Bldtg Width	PLF vert W	PLF vert L	Bldg Load	Bldg Load	
Level 1	15.33	0	399.8333	54.36	76.27	21.73	30.49	52.23
Level 2	15.33	15.33	399.8333	120.35	159.38	48.12	63.73	111.85
Level 3	15.33	30.66	399.8333	145.03	159.47	57.99	63.76	121.75
Level 4	14.67	45.99	399.8333	158.65	156.00	63.43	62.37	125.81
Level 5	14.67	60.66	352	203.04	187.11	71.47	65.86	137.33
Level 6	14.67	75.33	352	177.48	152.62	62.47	53.72	116.19
Level 7	14.67	90	352	185.97	152.53	65.46	53.69	119.15
Level 8	14.67	104.67	352	196.63	152.53	69.21	53.69	122.91
Level 9	14.67	119.34	352	202.70	152.53	71.35	53.69	125.04
Level 10	14.67	134.01	352	208.60	152.53	73.43	53.69	127.12
Level 11	14.67	148.68	352	215.53	152.53	75.87	53.69	129.56
Level 12	14.67	163.35	352	213.45	152.53	75.13	53.69	128.83
Level 13	14.67	178.02	352	218.30	152.53	76.84	53.69	130.53
Level 14	14.67	192.69	352	223.16	152.53	78.55	53.69	132.24
Level 15	14.67	207.36	352	244.14	152.53	85.94	53.69	139.63
Level 16	14.67	222.03	352	244.14	152.53	85.94	53.69	139.63
Level 17 PentFlr	17.75	236.7	352	269.80	168.57	94.97	59.34	154.31
Level 18 PentMez	12.96	254.45	352	255.58	159.68	89.97	56.21	146.17
Level 19 Roof	0	267.41	352	107.85	67.38	37.96	23.72	61.68
Total						1305.84	1016.12	2321.96

Detailed Calculations - N-S								
				Windward	Leeward	Windward kips	Leeward kips	Total Kips
Floor	Height	Elevation	Bldtg Width	PLF vert W	PLF vert L	Bldg Load	Bldg Load	
Level 1	15.33	0	160.1667	54.36	76.27	8.71	12.22	20.92
Level 2	15.33	15.33	160.1667	120.35	159.38	19.28	25.53	44.80
Level 3	15.33	30.66	160.1667	145.03	159.47	23.23	25.54	48.77
Level 4	14.67	45.99	160.1667	158.65	156.00	25.41	24.99	50.40
Level 5	14.67	60.66	128.6667	203.04	187.11	26.12	24.08	50.20
Level 6	14.67	75.33	128.6667	177.48	152.62	22.84	19.64	42.47
Level 7	14.67	90	128.6667	185.97	152.53	23.93	19.63	43.55
Level 8	14.67	104.67	128.6667	196.63	152.53	25.30	19.63	44.93
Level 9	14.67	119.34	128.6667	202.70	152.53	26.08	19.63	45.71
Level 10	14.67	134.01	128.6667	208.60	152.53	26.84	19.63	46.47
Level 11	14.67	148.68	128.6667	215.53	152.53	27.73	19.63	47.36
Level 12	14.67	163.35	77.9167	213.45	152.53	16.63	11.88	28.52
Level 13	14.67	178.02	77.9167	218.30	152.53	17.01	11.88	28.89
Level 14	14.67	192.69	77.9167	223.16	152.53	17.39	11.88	29.27
Level 15	14.67	207.36	77.9167	244.14	152.53	19.02	11.88	30.91
Level 16	14.67	222.03	77.9167	244.14	152.53	19.02	11.88	30.91
Level 17 PentFlr	17.75	236.7	77.9167	269.80	168.57	21.02	13.13	34.16
Level 18 PentMez	12.96	254.45	77.9167	255.58	159.68	19.91	12.44	32.36
Level 19 Roof	0	267.41	77.9167	107.85	67.38	8.40	5.25	13.65
Total						393.87	320.36	714.24

Above: Detailed calculations for story shear

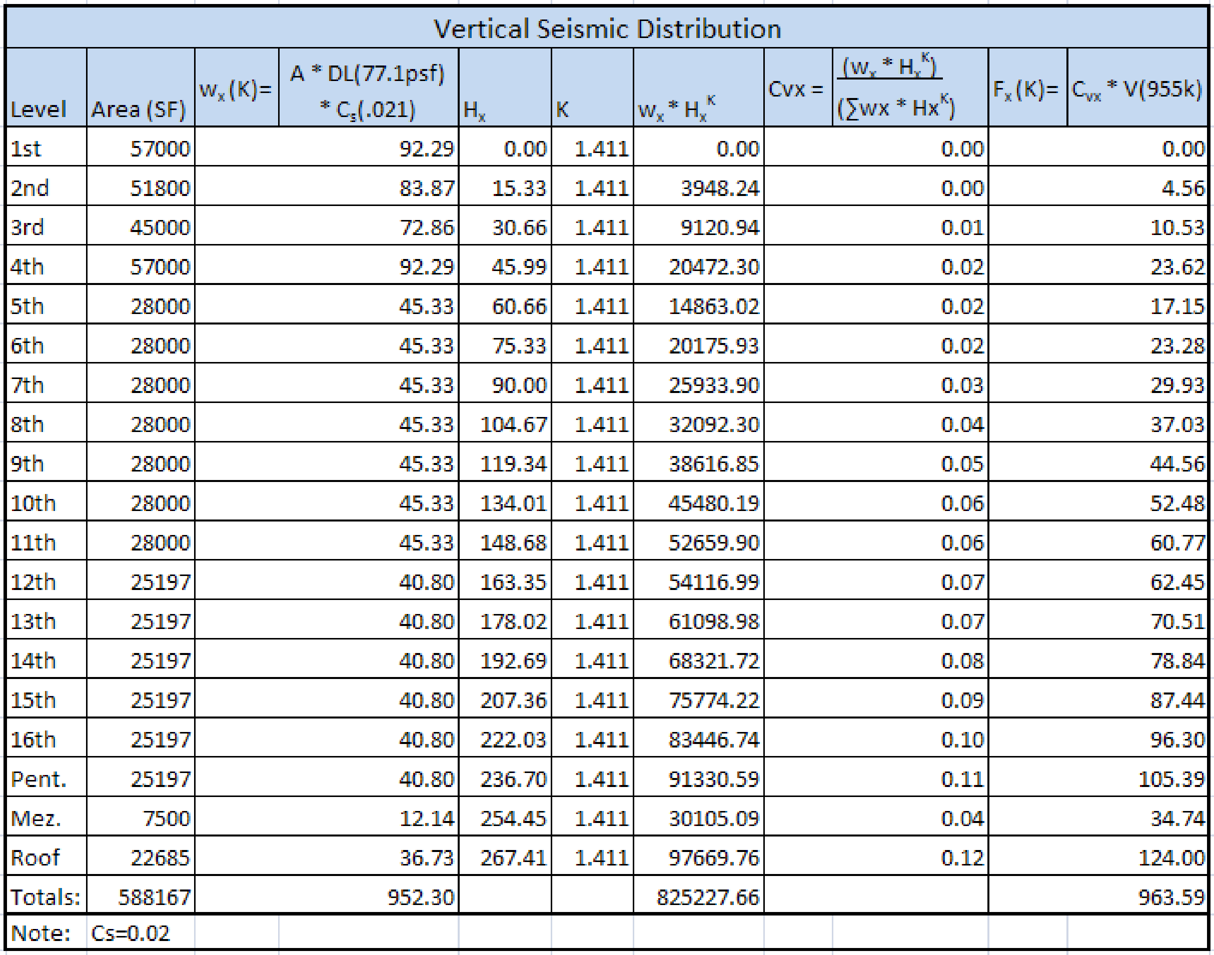
Seismic Loads

The ASCE7-05 code was used to investigate the seismic loads for the building which were expected to be relatively low, given the building's location in Philadelphia, Pennsylvania. The Equivalent lateral Force Procedure (ASCE 12.8.2) was used to obtain a base sheer for the building. A direct comparison with the original structural notes within the design documents was not possible since the base shear was not indicated in the notes or specifications; however, the notes did state that structural system was not specifically detailed for seismic loads. It also provided various seismic data which were used for this analysis including the site class, C, and the spectral response coefficients: S_{DS}=0.219, S_{D1}=0.068.

Ordinary steel concentrically braced frames were used to determine the response modification coefficient R=3, which combined with the site class C did not limit the building height. The estimated dead weight of 77.1psf was used to calculate the building base shear of 961 kips which is a smaller base shear than the original design's 970 kips. This decrease is due to the greater building height, which increases the fundamental period T, giving a lower seismic response coefficient C_s.

Below appears the ASCE equation for determining the vertical distribution of seismic loads, which is found in table 3, of the Lateral Force Distribution Section. This was accomplished by multiplying the base shear by the appropriate C_{vx}, which is a function of floor area, height, building weight, and the seismic response coefficient. Again, the higher importance factor was used due to the seating capacity of the library, which exceeded 300 persons.

$$Cvx = \frac{(w_x * H_x^K)}{(\sum wx * Hx^K)}$$



Vertical Seismic Distribution										
Level	Area (SF)	w_x (K)=	A * DL(77.1psf) * C_s(.021)	H_x	K	$w_x * H_x^K$	Cvx =	$\frac{(w_x * H_x^K)}{(\sum wx * Hx^K)}$	F_x (K)=	C_{vx} * V(955k)
1st	57000		92.29	0.00	1.411	0.00		0.00		0.00
2nd	51800		83.87	15.33	1.411	3948.24		0.00		4.56
3rd	45000		72.86	30.66	1.411	9120.94		0.01		10.53
4th	57000		92.29	45.99	1.411	20472.30		0.02		23.62
5th	28000		45.33	60.66	1.411	14863.02		0.02		17.15
6th	28000		45.33	75.33	1.411	20175.93		0.02		23.28
7th	28000		45.33	90.00	1.411	25933.90		0.03		29.93
8th	28000		45.33	104.67	1.411	32092.30		0.04		37.03
9th	28000		45.33	119.34	1.411	38616.85		0.05		44.56
10th	28000		45.33	134.01	1.411	45480.19		0.06		52.48
11th	28000		45.33	148.68	1.411	52659.90		0.06		60.77
12th	25197		40.80	163.35	1.411	54116.99		0.07		62.45
13th	25197		40.80	178.02	1.411	61098.98		0.07		70.51
14th	25197		40.80	192.69	1.411	68321.72		0.08		78.84
15th	25197		40.80	207.36	1.411	75774.22		0.09		87.44
16th	25197		40.80	222.03	1.411	83446.74		0.10		96.30
Pent.	25197		40.80	236.70	1.411	91330.59		0.11		105.39
Mez.	7500		12.14	254.45	1.411	30105.09		0.04		34.74
Roof	22685		36.73	267.41	1.411	97669.76		0.12		124.00
Totals:	588167		952.30			825227.66				963.59
Note:	Cs=0.02									

Load Combinations

The following IBC 2006 load combinations were analyzed using RAM Structural System. The second and third groups of Load combinations had the most significant impact on the lateral system, while the bold underlined load combinations controlled.

Per IBC 2006 1607.9.1 live load reduction was not utilized for the most part in the design, since reductions are not allowed for loads exceeding 100psf, which is the case for most areas in the building.

Eq 16-1

$1.4D$

Eq 16-2

$1.2D + 1.6L + 0.5L_r$

$1.2D + 1.6L + 0.5S$

Eq 16-3

$1.2D + 1.6L_r \pm 0.8W$

$1.2D + 1.6S \pm 0.8W$

Eq 16-4

$1.2D + 1.6W + 0.5L_r$

$1.2D + 1.6W + 0.5S$

Eq 16-5

$1.2D \pm 1.0E$

Eq 16-6

$0.9D \pm 1.6W$

Eq 16-7

$0.9D \pm 1.0E$

Lateral Design

Goals

The first goal of the lateral system design is to redesign the EW braced frames to handle the additional loading, and to do so in a manner that will not heavily increase price. The second goal is to replace the existing NS moment frames with braced frames to achieve a more economical design for the framing system in this direction. Two overarching goals guided the design of both systems: Minimizing the frame design's impact on architectural planning and aesthetics, and creating the most efficient design possible.

Design Process

The process for this design will begin with the creation of a RAM structural model. After the structure is sized for gravity loading, the RAM Frame module will be used to size the lateral frames for first for strength and then for serviceability. Afterwards a final strength check within RAM will be made.

STAAD will be used to confirm the RAM analysis, and hand calculations will be used to double check both of those results. Hand calculations based off of tributary area will be used to manually verify the column sizes at the basement level. Another form of verification will take place by comparing the column sizes of the existing original design, the modeled original design, and the new design. This comparison will make evident any discrepancies between the existing original design and the computer models, and provide a check for the validity of the computer models results. These hand calculations depend on the final gravity analysis, and will therefore be discussed in detail in that section, with only their results presented in the Lateral Design Section.

Once the frames have been designed, modeling data from RAM, as well as the MHSC design specifications will be used to perform a preliminary lateral foundation design, as part of the lateral analysis. This will be a schematic design based off of the end bearing capacity. These foundation sizes will then be compared to the original design.

The economical analysis consists of comparing the ratio of structural system tonnage to square footage for the original and new systems. The goals is to keep the increase in the ratio to a minimum, in order to support the owners decision to add to the existing design as opposed to constructing a new building to meet the university's expansion needs. This analysis will be summarized with the findings of this section, the gravity design, and vibration check in the conclusion section at the end of the Structural Breadth section.

Schematic Design

The first step in the design process was to determine the frame geometries and locations. For the East-West braced frames, this merely meant continuing the framing up vertically with chevron bracing. This decision was possible since the architectural layout of the additional floors was similar to the existing floors, which already incorporated chevron bracing; therefore, the existing frame configuration was suitable to insure no conflict with floor plan layouts.

The design of the N-S braced frames, which replaced the preexisting moment frames, was more difficult to achieve. This required a schematic calculation for determining the basic number of frames required, an analysis for the best location in terms of architecture, and the design of the bracing geometry.

A schematic calculation was used to get a rough idea of the number of frames required to resist the N-S wind loading. This was accomplished by comparing the ratio of $\frac{Number\ of\ braced\ frames}{wind\ loading}$ for the original EW frames and wind loads, to the new NS loads to obtain the number of NS frames required. Wind loading controlled in the original design, so those numbers were used in the estimate to obtain a requirement of 2.3 frames, which was simplified to 2 frames (see calculation below).

$$\frac{5\ EW\ Frames\ (original)}{1525\ kips} = \frac{\#\ of\ NS\ frames\ (new)}{714\ kips}$$

With the number of frames calculated a suitable location had to be determined which would minimize stiffness eccentricities and limit architectural disruption. Placing the two frames about the elevator core (highlighted in blue on the framing plan on the next page) was an appropriate location choice because the symmetry and central location would insure that the frame location and geometry would eliminate torsional issues. Torsion is the result of eccentricity between the centers of mass and rigidity which can result in additional loading on the frame structure, resulting in a bigger design; however, the central location minimized eccentricity, especially in the tower floors. This is confirmed by the RAM modeling data (presented on the right), which is

Floor Eccentricity

	Center of Rigidity		Center of Mass		Eccentricty	
Level	Xr ft	Yr ft	Xm ft	Ym ft	Xr ft	Yr ft
19roof	175.85	109.77	174.33	118.99	1.52	-9.22
18pentme	175.96	109.82	191.2	126.12	-15.24	-16.3
17th	176.23	109.94	174.32	117.3	1.91	-7.36
16th	176.45	109.89	174.33	117.34	2.12	-7.45
15th	176.71	109.58	174.31	117.34	2.4	-7.76
14th	177.01	109	174.32	117.38	2.69	-8.38
13th	177.35	108.45	174.32	117.42	3.03	-8.97
12th	177.77	108	171.69	110.14	6.08	-2.14
11th	178.26	107.68	171.6	110	6.66	-2.32
10th	178.86	107.75	171.52	110.03	7.34	-2.28
9th	179.6	107.86	171.45	110.06	8.15	-2.2
8th	180.52	107.95	171.47	110.08	9.05	-2.13
7th	181.69	107.99	171.48	110.11	10.21	-2.12
6th	183.28	107.88	171.51	110.13	11.77	-2.25
5th	185.65	107.77	173.36	110.95	12.29	-3.18
4th	184.09	107.61	186.4	95.24	-2.31	12.37
3rd	184.13	106.79	199.03	106.14	-14.9	0.65
2nd	184.21	107.39	184.53	100.47	-0.32	6.92
1st	184.6	108.1	194.92	93.98	-10.32	14.12

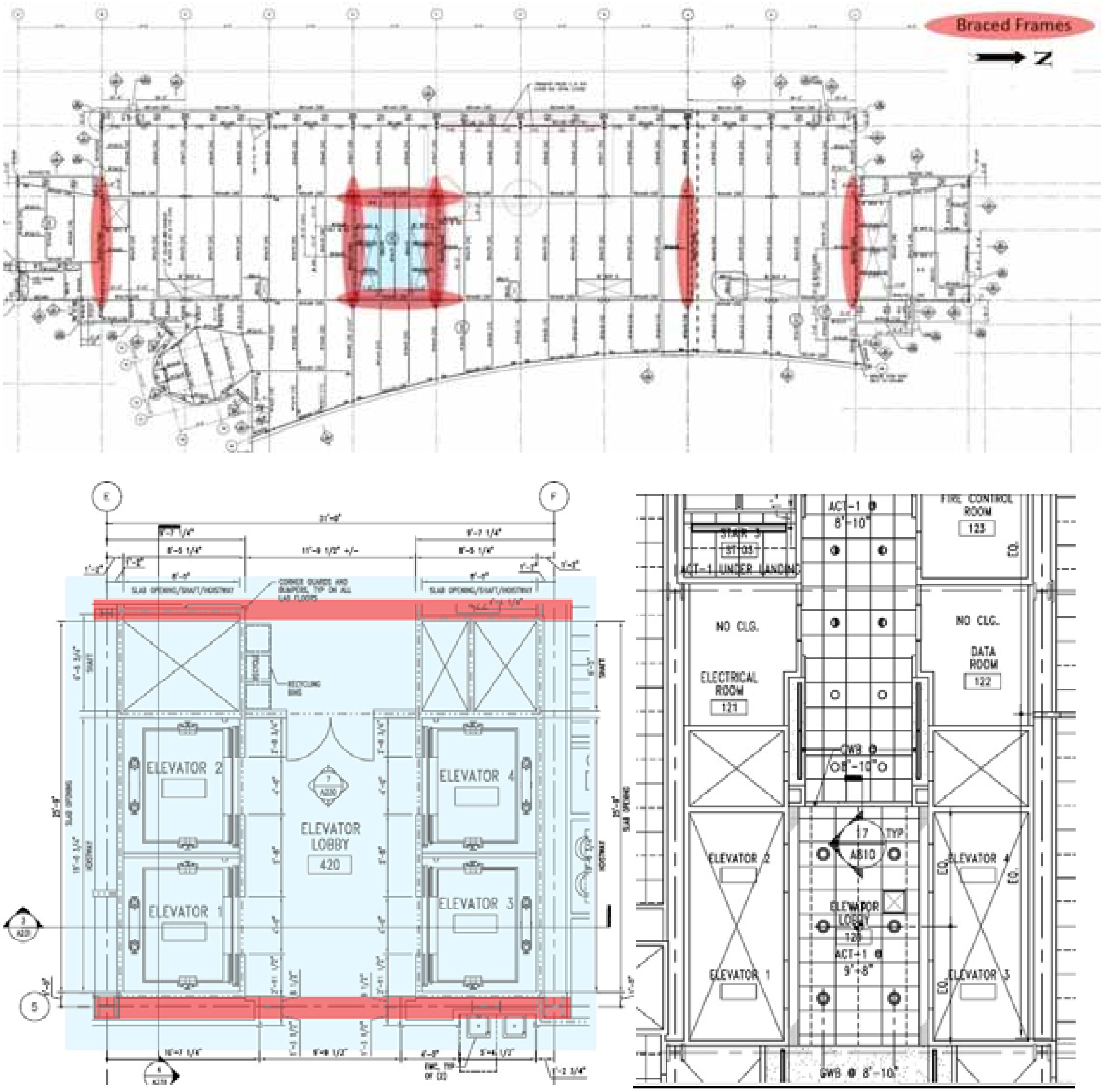
* y distances pertain to eccentrictiy in the E-W direction

* x distances pertain to eccentrictiy in the N-S direction

* x and y values are derived from the modeling origin in RAM

discussed in greater detail later. Torsion is also affected by differences in stiffness, but the symmetrical opening of the elevator core allows the frames to have the same geometry, insuring similar stiffness, and once again limiting torsion.

The actual geometry of the N-S frame was determined by the 8’ length of elevator core walls, (highlighted on the plan on the previous page) and the highest ceiling height of 8”10” at the frame intersection. These dimensions determined the brace points (see frame model on the following page)



Top: Locations of braced frames in typical tower level framing plan, with elevator core highlighted in blue.
Below left: Elevator core detail showing.
Below right: Reflected ceiling plan with limiting ceiling heights.

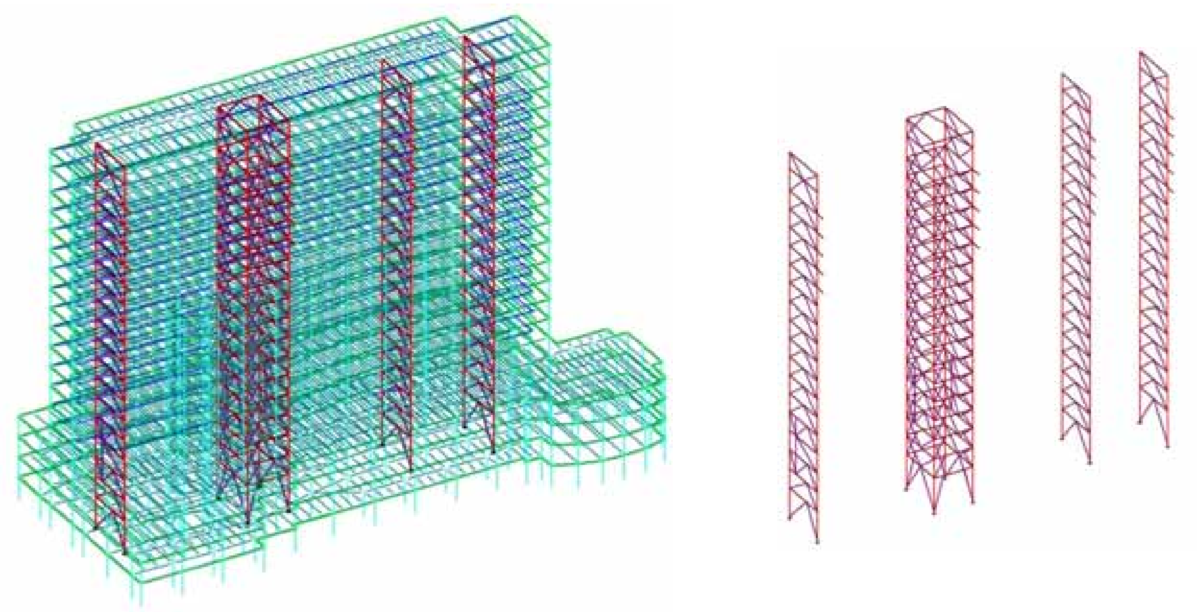
Strength Design

Creating the RAM Model

RAM structural system is a powerful tool because with one model, the designer has the ability to complete a structural design for a variety of building systems: lateral, gravity, foundation, etc. It was advantageous to use RAM to design these systems especially since the new model could be easily derived from the original, avoiding redundant design processes.

Unfortunately, a black box analysis can create the dangerous situation where the designer inputs data without knowing how the program uses it. Great care was taken to eliminate this possibility by attempting to faithfully model the building, clearly define assumptions, and by hand calculating loading.

The model – as depicted below in its entirety and with the frames emphasized- included accurately modeled composite decking, beams, columns, and girders. Large floor openings and varying load conditions, such as the green roof, mechanical, and office loads were also modeled accurately. The wind and seismic loading calculations, IBC load combinations, and live load reduction criteria presented earlier in the Structural Breadth section were inputted by hand into the RAM model. RAM was not used to automatically generate these conditions. Other criteria that were important to specify in the program were the use of braced connections, using the nomographs to determine KL values, beam and brace nodes connected to the rigid diaphragm, and the inclusion of the P-delta effect for the modeling.



Left: The entire RAM model. Right: The braced frames only.

Strength Design

The first set of design iterations with the RAM model focused on obtaining the most efficient strength design as possible. In most cases, the window load combination
$1.2D + 1.6W + 0.5L_r$ controlled the strength design. The first step was to assign member sizes to the frames. RAM does not automatically design frame elements, so gravity analysis results were used to assign trial column and beam sizes for the frames, while an oversized members were assigned to the braces. RAM frame was then used to analyze the frames for strength. Brace strength was based off of axial forces only, while beams and columns were analyzed according the interaction equations (written below) for combined axial and flexural loading. In most equations, equation H1-1a controlled the column design, while equation H1-1b controlled the beam design.

Equation H1-1a:

For $\frac{P_r}{P_c} \geq 0.2$

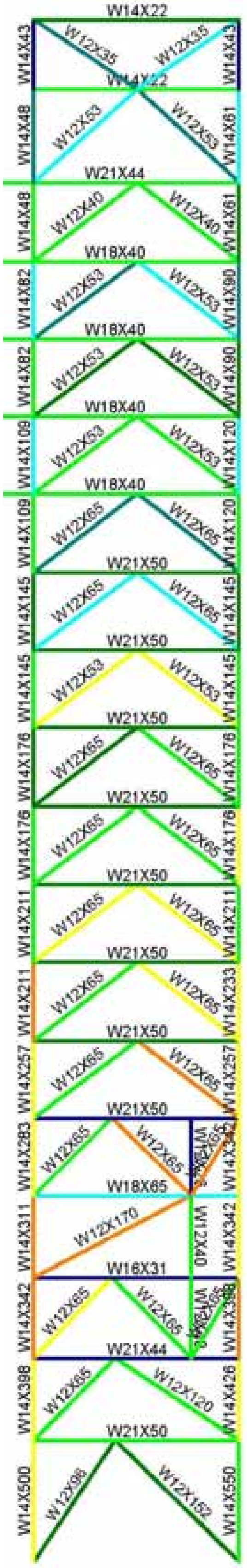
$$\frac{P_r}{P_c} + \frac{8}{9}\left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right) \leq 1.0$$

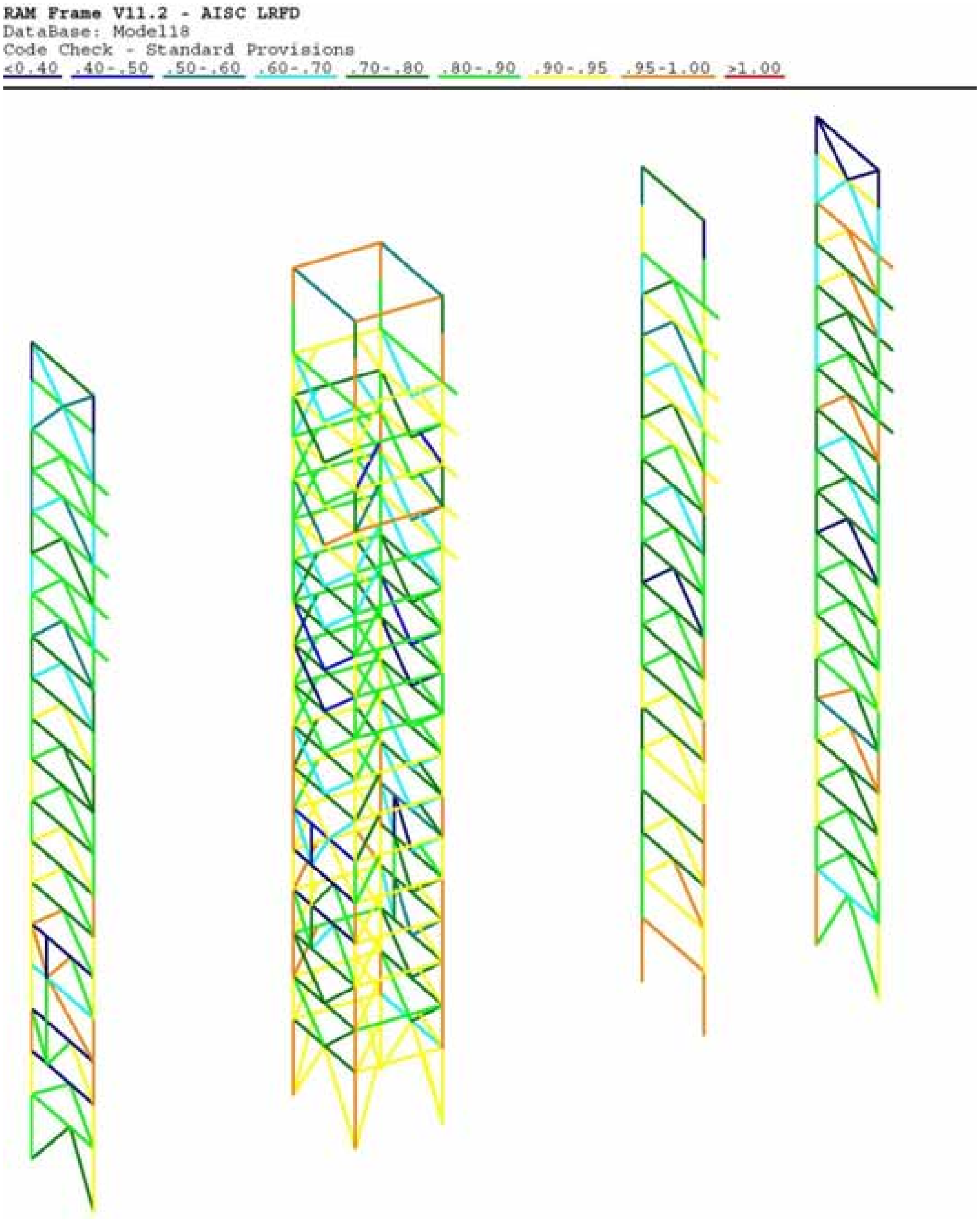
Equation H1-1b:

For $\frac{P_r}{P_c} < 0.2$

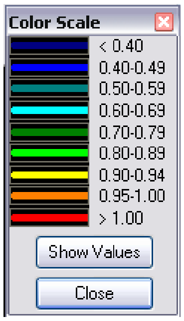
$$\frac{P_r}{2P_c} + \left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right) \leq 1.0$$

The percentage of the strength capacity actually used by the member is graphically displayed in RAM, with dark blue indicating an under-stressed design, and red indicating a design which fails the interaction equation (See image on right and on next page). After the strength design was complete, spot checks were made by hand by designing the member in question using Table 6-1 of the Steel Construction manual, and the member loads from RAM. The results matched the model and confirmed the KL criteria used in RAM.



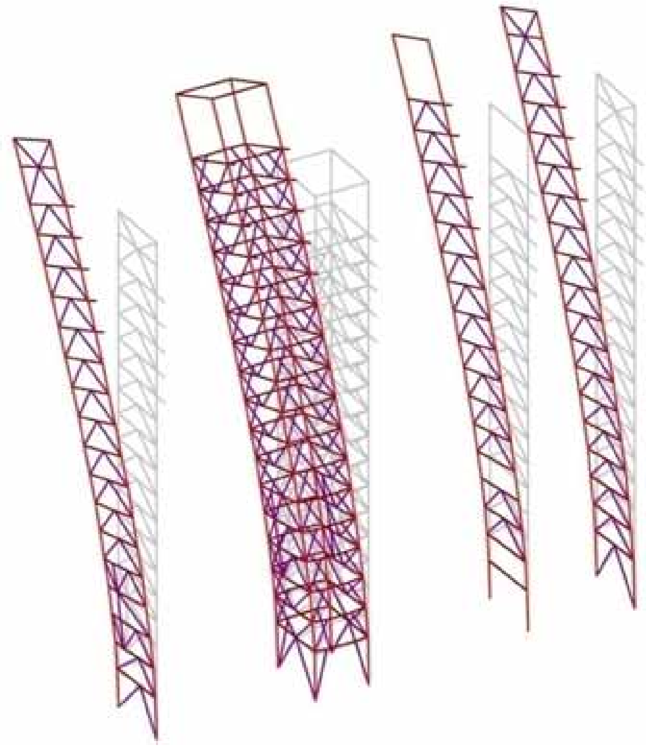


```
RAM Frame V11.2 - AISC LRFD
DataBase: Model18
Code Check - Standard Provisions
```



Drift Design

After the initial strength design was completed, a drift analysis was done to check maximum total and story drift, which is limited to H/400 = 267'/400 = 8.83". The image below clearly shows the design failing for E-W wind loading with 11.8" drift. Multiple design iterations were performed in an attempt to decrease drift within the required limit. All of the N-S and E-W frame member sizes were initially increased to reach this goal, with iterations afterwards focusing on controlling the E-W drift; however, after dozens of iterations, drift was still too large, despite drastic increases in member sizes. At this point, STAAD was used to verify the accuracy of the RAM model.



Deflected shape of strength designed model

It was assumed that some error within the RAM model was causing the continuously too high drifts. The table below shows the comparison procedure of a typical E-W frame, with drift limits set by the IBC 2006 code compared to the RAM model's drift, and the STAAD frame's drift. (The final model, which was within the drift requirements, is included in the table as well.) The RAM story drift was used instead of the RAM frame drift to provide a more average drift value to compare the STAAD data to. Also, two sets of STAAD models were made to compare to the RAM model. "STAAD Model 37" utilized the frame shear calculated by RAM, which uses the relative stiffness method to distribute the loading. "STAAD Model 37mod" used the hand calculated story wind shears (see table in wind loading section), assuming that the loads were evenly distributed to each of the E-W frames. The advantage of using both of these STAAD models for comparison was that the potential error in one balanced out the error in the other. “Model 37” would take eccentricity into account but would have the potential RAM error embedded in shear story loads leading to an inaccurate design. “Model 37mod” on the other hand, may incorrectly assume that there is no eccentricity in the building, but would insure that no internal RAM modeling error was corrupting the output.

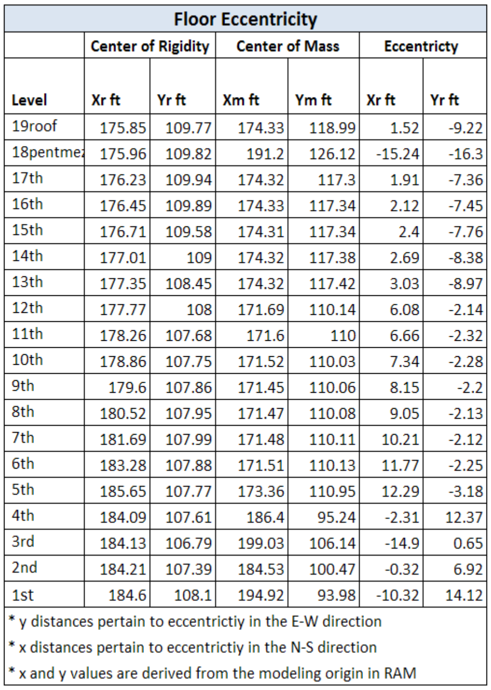
Drift Comparison

Required					RAM Model 37		Staad Model 37		Staad Model 37mod		RAM Model Final	
Level	**Height**	**h + lower lvl**	**Max = H/400**	**Change Delta**	**Delta**	**Change Delta**	**Delta**	**Change Delta**	**Delta**	**Change Delta**	**Delta**	**Change Delta**
Roof	267.41	294.41	8.832	0.389	9.321	0.513	10.504	0.494	9.895	0.465	6.7918	0.3814
Mez.	254.45	281.45	8.444	0.533	8.808	0.661	10.01	0.713	9.43	0.663	6.4105	0.5365
Pent.	236.70	263.70	7.911	0.440	8.147	0.572	9.297	0.599	8.767	0.563	5.874	0.4468
16th	222.03	249.03	7.471	0.440	7.575	0.577	8.698	0.611	8.204	0.572	5.4271	0.4481
15th	207.36	234.36	7.031	0.440	6.998	0.581	8.087	0.615	7.632	0.582	4.979	0.4435
14th	192.69	219.69	6.591	0.440	6.417	0.584	7.472	0.623	7.05	0.587	4.5356	0.4309
13th	178.02	205.02	6.151	0.440	5.833	0.580	6.849	0.626	6.463	0.590	4.1047	0.4086
12th	163.35	190.35	5.711	0.440	5.253	0.580	6.223	0.627	5.873	0.591	3.6961	0.3815
11th	148.68	175.68	5.270	0.440	4.673	0.572	5.596	0.623	5.282	0.587	3.3147	0.3765
10th	134.01	161.01	4.830	0.440	4.101	0.561	4.973	0.617	4.695	0.578	2.9382	0.3727
9th	119.34	146.34	4.390	0.440	3.541	0.544	4.356	0.603	4.117	0.566	2.5654	0.3663
8th	104.67	131.67	3.950	0.440	2.997	0.522	3.753	0.588	3.551	0.549	2.1992	0.3566
7th	90.00	117.00	3.510	0.440	2.474	0.495	3.165	0.570	3.002	0.524	1.8426	0.3441
6th	75.33	102.33	3.070	0.440	1.979	0.461	2.595	0.492	2.478	0.482	1.4985	0.3268
5th	60.66	87.66	2.630	0.440	1.518	0.439	2.103	0.549	1.996	0.429	1.1717	0.3095
4th	45.99	72.99	2.190	0.460	1.080	0.401	1.554	0.459	1.567	0.474	0.8622	0.3101
3rd	30.66	57.66	1.730	0.460	0.679	0.328	1.095	0.527	1.093	0.453	0.5521	0.2436
2nd	15.33	42.33	1.270	0.460	0.351	0.216	0.568	0.348	0.64	0.323	0.3085	0.1636
1st	0.00	27.00	0.810	0.810	0.135	0.135	0.22	0.220	0.317	0.317	0.1449	0.1449

It was assumed that some error within the RAM model was causing the continuously too high drifts; however, the STAAD model actually confirmed the RAM results within 10%. The STAAD model results show that under the two load conditions, the frame fails story drift at the 3rd level, and total drift at the 10th level, whereas the RAM model fails at the 6th and 16th respectively. This provides two conclusions:

1) The RAM model is for the most part working correctly and the frame members sizes need to be increased. The model can be trusted.
2) This confirms that some sort of torsional issue is causing the frame drift failure.

Judging from the second conclusion, it is likely that the N-S frames are strongly affecting the relative stiffness of the floor, especially since their column strong axes are oriented for N-S loading instead of E-W. A summarization of floor eccentricities (appearing in the conclusion section) for the final design confirmed that significant eccentricity of approximately 6’ to12’ were causing torsional issues, which were not resolved until the N-S frames were designed.

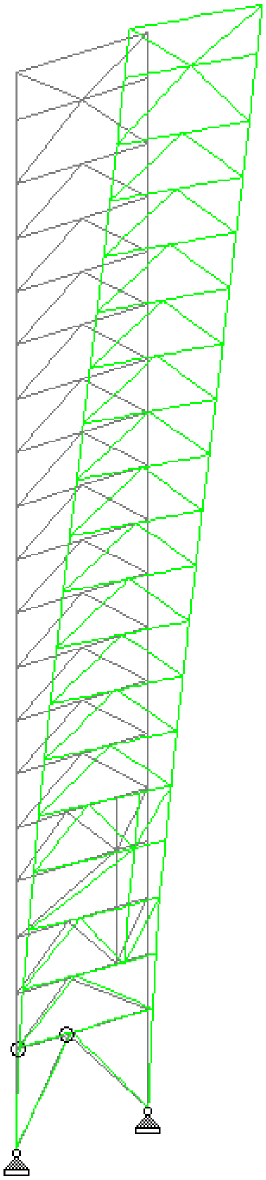


Floor Eccentricity						
	Center of Rigidity		Center of Mass		Eccentricty	
Level	Xr ft	Yr ft	Xm ft	Ym ft	Xr ft	Yr ft
19roof	175.85	109.77	174.33	118.99	1.52	-9.22
18pentme	175.96	109.82	191.2	126.12	-15.24	-16.3
17th	176.23	109.94	174.32	117.3	1.91	-7.36
16th	176.45	109.89	174.33	117.34	2.12	-7.45
15th	176.71	109.58	174.31	117.34	2.4	-7.76
14th	177.01	109	174.32	117.38	2.69	-8.38
13th	177.35	108.45	174.32	117.42	3.03	-8.97
12th	177.77	108	171.69	110.14	6.08	-2.14
11th	178.26	107.68	171.6	110	6.66	-2.32
10th	178.86	107.75	171.52	110.03	7.34	-2.28
9th	179.6	107.86	171.45	110.06	8.15	-2.2
8th	180.52	107.95	171.47	110.08	9.05	-2.13
7th	181.69	107.99	171.48	110.11	10.21	-2.12
6th	183.28	107.88	171.51	110.13	11.77	-2.25
5th	185.65	107.77	173.36	110.95	12.29	-3.18
4th	184.09	107.61	186.4	95.24	-2.31	12.37
3rd	184.13	106.79	199.03	106.14	-14.9	0.65
2nd	184.21	107.39	184.53	100.47	-0.32	6.92
1st	184.6	108.1	194.92	93.98	-10.32	14.12

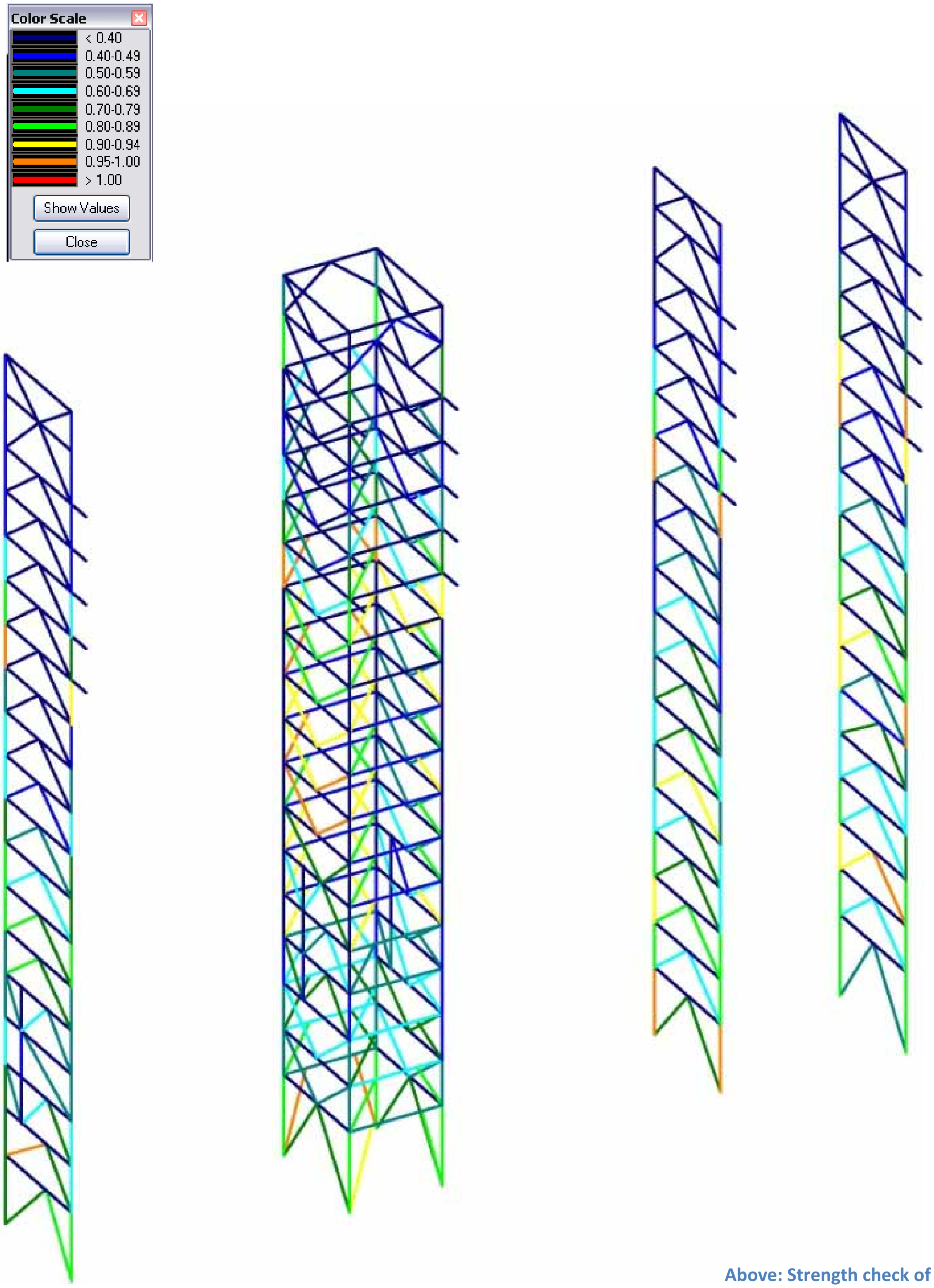
* y distances pertain to eccentrictiy in the E-W direction
* x distances pertain to eccentrictiy in the N-S direction
* x and y values are derived from the modeling origin in RAM

With this new knowledge, several design iterations were performed on the N-S frames within RAM. The sizes of the N-S members were drastically increased until the N-S drift was under control. Once this point was reached, the N-S frames had become stiff enough to minimize torsion effects and to allow a reduction in the E-W frame member sizes. It was then evident that the design was controlled by the N-S frames

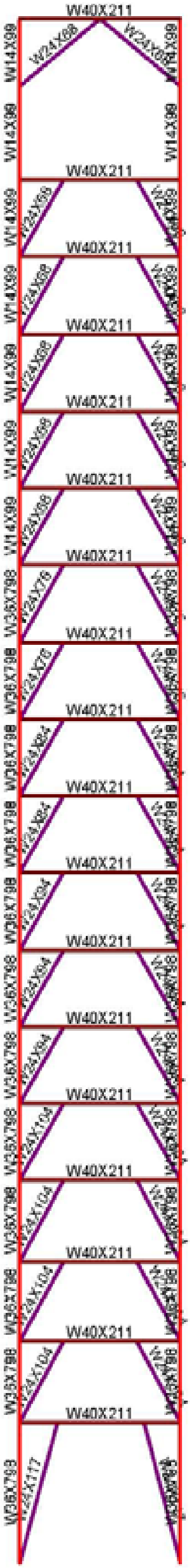
The lateral system underwent a final strength check after the N-S frames and E-W frames went through final drift design iteration. At this point the design remained for the most part unchanged; however some members did need resizing, especially at the base. The finalized strength check for the entire lateral system is presented below and a diagram including member sizes of the final N-S and E-W typical frames are presented on the next page along with a typical E-W frame from the original existing diagram.

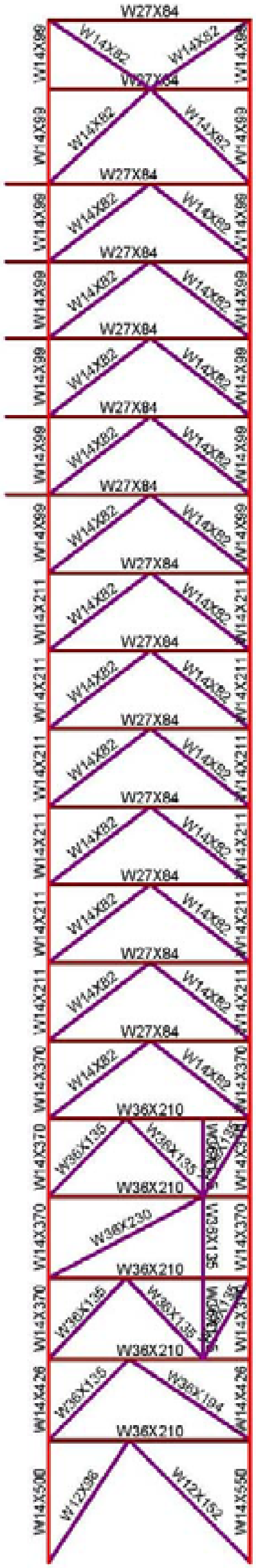


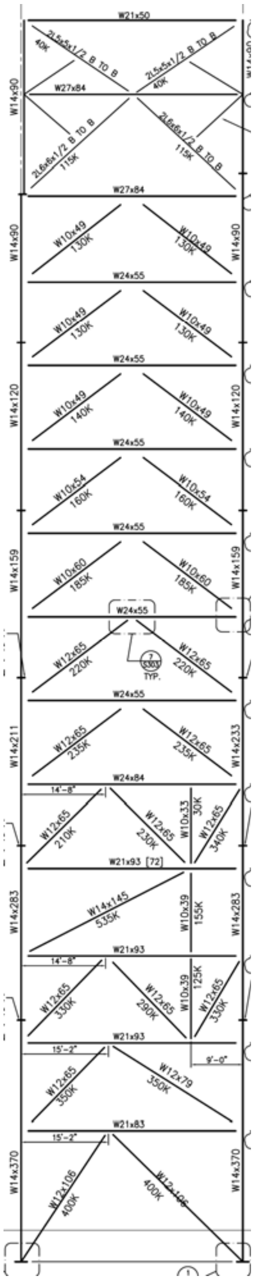
Right: STAAD “Model 37 mod” drift diagram, with total drift = 9.89”

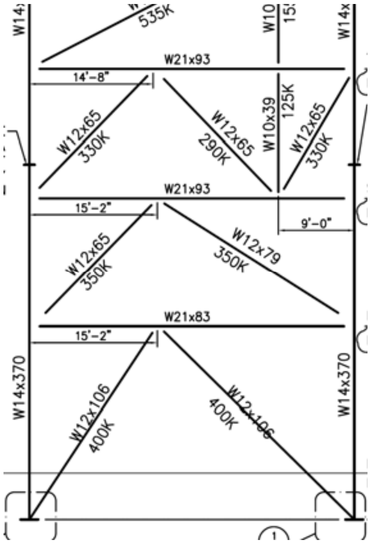


Above: Strength check of final design









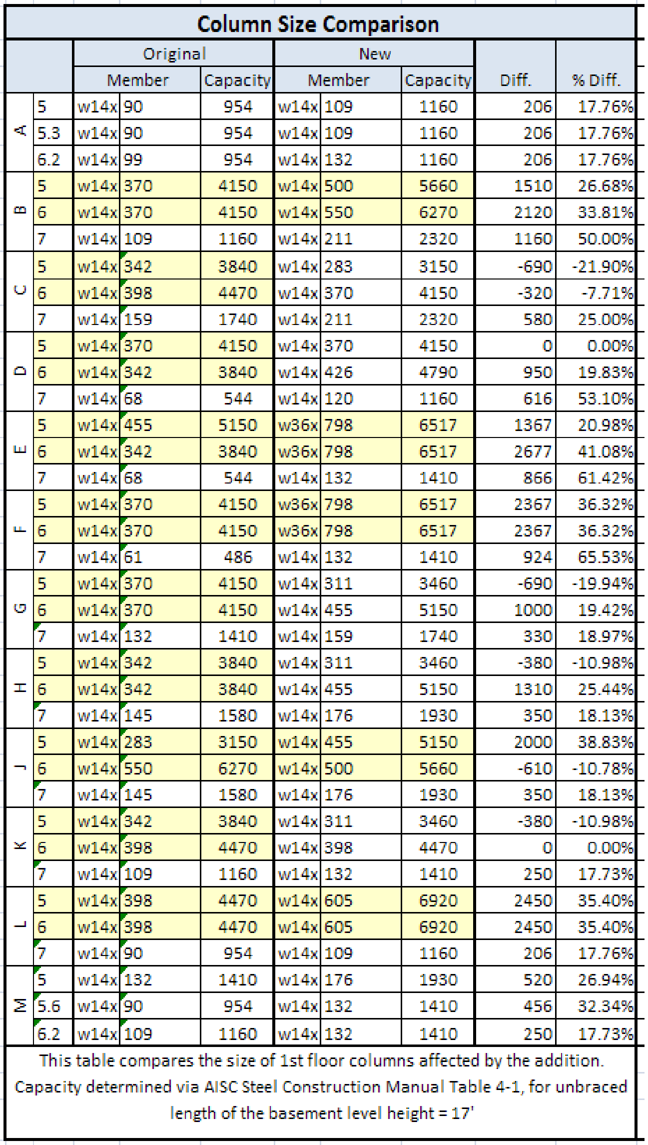
From left to right:
New N-S Braced Frame,
New E-W Braced Frame,
Original E-W braced Frame,
Detail of Original Braced Frame

Verification

Although the STAAD analysis has already determined that the RAM model is accurate, a final manual verification is important to check the relative reasonableness of the new design. The final manual verification comes from a comparison of the size of the framing systems depicted on the previous page and the column check which is conduted in the Gravity Design section.

As seen on the previous page, most of the E-W frame member sizes increased but remained in a similar range as in the existing original design. This indicates that the new E-W framing system is a reasonable design. The N-S frames are also a reasonable design considering the amount of loading they must carry, even if the design proves to be inefficient over the original moment frames; however the final sizing does make sense.

The clumn check appearing to the right provides another means of comparing the design. The check compares the basement column capacities of the original design RAM model and the new design RAM model. This comparison is described in detail in the gravity section, but it is helpful to point out here that all the columns which were part of the original lateral frame system in the orignal and new designs went up in size. Those columns which were part of the moment frame originally but are now simply gravity columns with the new design actually went down in size, which makes sense since they are carrying mostly axial loads in the new gravity condition as oposed to axial and flexural loads in the original moment frame condition.

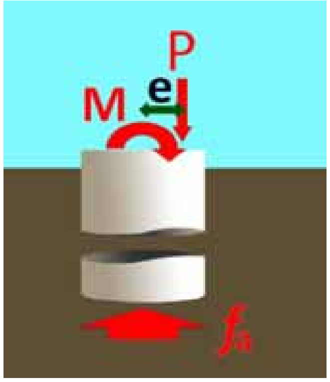


Column Size Comparison

		Original			New				
		Member		Capacity	Member		Capacity	Diff.	% Diff.
A	5	w14x	90	954	w14x	109	1160	206	17.76%
	5.3	w14x	90	954	w14x	109	1160	206	17.76%
	6.2	w14x	99	954	w14x	132	1160	206	17.76%
B	5	w14x	370	4150	w14x	500	5660	1510	26.68%
	6	w14x	370	4150	w14x	550	6270	2120	33.81%
	7	w14x	109	1160	w14x	211	2320	1160	50.00%
C	5	w14x	342	3840	w14x	283	3150	-690	-21.90%
	6	w14x	398	4470	w14x	370	4150	-320	-7.71%
	7	w14x	159	1740	w14x	211	2320	580	25.00%
D	5	w14x	370	4150	w14x	370	4150	0	0.00%
	6	w14x	342	3840	w14x	426	4790	950	19.83%
	7	w14x	68	544	w14x	120	1160	616	53.10%
E	5	w14x	455	5150	w36x	798	6517	1367	20.98%
	6	w14x	342	3840	w36x	798	6517	2677	41.08%
	7	w14x	68	544	w14x	132	1410	866	61.42%
F	5	w14x	370	4150	w36x	798	6517	2367	36.32%
	6	w14x	370	4150	w36x	798	6517	2367	36.32%
	7	w14x	61	486	w14x	132	1410	924	65.53%
G	5	w14x	370	4150	w14x	311	3460	-690	-19.94%
	6	w14x	370	4150	w14x	455	5150	1000	19.42%
	7	w14x	132	1410	w14x	159	1740	330	18.97%
H	5	w14x	342	3840	w14x	311	3460	-380	-10.98%
	6	w14x	342	3840	w14x	455	5150	1310	25.44%
	7	w14x	145	1580	w14x	176	1930	350	18.13%
J	5	w14x	283	3150	w14x	455	5150	2000	38.83%
	6	w14x	550	6270	w14x	500	5660	-610	-10.78%
	7	w14x	145	1580	w14x	176	1930	350	18.13%
K	5	w14x	342	3840	w14x	311	3460	-380	-10.98%
	6	w14x	398	4470	w14x	398	4470	0	0.00%
	7	w14x	109	1160	w14x	132	1410	250	17.73%
L	5	w14x	398	4470	w14x	605	6920	2450	35.40%
	6	w14x	398	4470	w14x	605	6920	2450	35.40%
	7	w14x	90	954	w14x	109	1160	206	17.76%
M	5	w14x	132	1410	w14x	176	1930	520	26.94%
	5.6	w14x	90	954	w14x	132	1410	456	32.34%
	6.2	w14x	109	1160	w14x	132	1410	250	17.73%

This table compares the size of 1st floor columns affected by the addition. Capacity determined via AISC Steel Construction Manual Table 4-1, for unbraced length of the basement level height = 17'

Frame Foundations

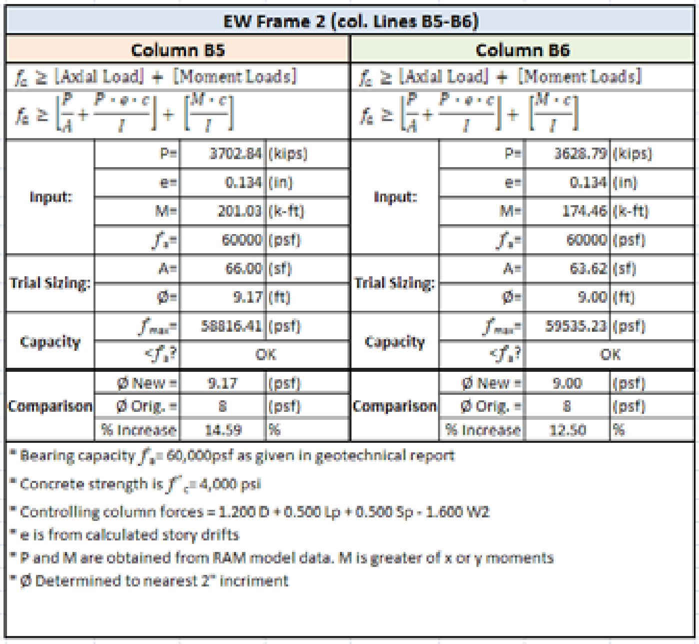


A basic foundation analysis was performed to more fully investigate the impacts on the lateral structual system caused by the increased bulding size and changing of N-S lateral system type. The framing foundation system consists of caisons which reach bedrock approximately 30’ to 50’ below grade. This type of foundation system relies primarily on the bearing capacity of the soil or bedrock. In the case of the MHSC, the intact mica bedrock provides 60,000psf bearing capacity.

The foundations were designed by taking into account the axial and flexural loading determined through the RAM model. Story drift causes a small eccentricity in the application of the axial loads, which combines with the column flexural loads and cross sectional area to create a stresss distribution, as depicted above. This stress distribution must be less than the total allowed bearing capacitiy, as shown in the equation below.

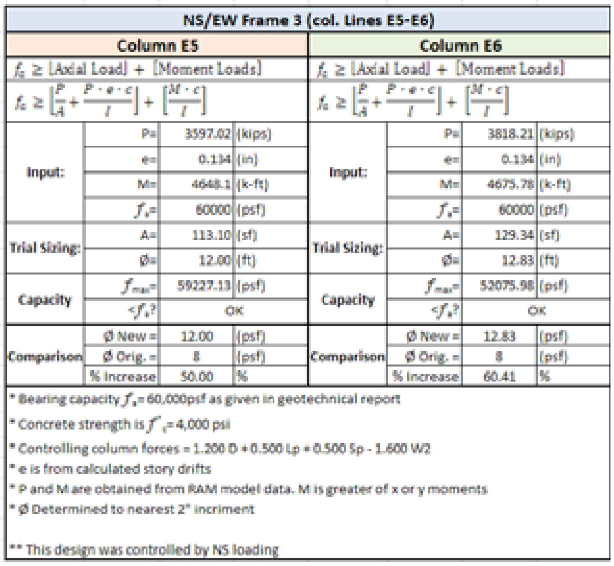
$$f_a \geq \left[\frac{P}{A} + \frac{P \cdot e \cdot c}{I}\right] + \left[\frac{M \cdot c}{I}\right]$$

The spread sheets below were created so that trial caisson diameters could be inputted. Using the above equation, the spread sheet is designed to output the resulting bearing stress, which is compared to the limiting bearing stress. Lastly, the resulting caisson sizes were compared with the orginal design.



EW Frame 2 (col. Lines B5-B6)							
Column B5				Column B6			
$f_a \geq$ [Axial Load] + [Moment Loads]				$f_a \geq$ [Axial Load] + [Moment Loads]			
$f_a \geq \left[\frac{P}{A} + \frac{P \cdot e \cdot c}{I}\right] + \left[\frac{M \cdot c}{I}\right]$				$f_a \geq \left[\frac{P}{A} + \frac{P \cdot e \cdot c}{I}\right] + \left[\frac{M \cdot c}{I}\right]$			
Input:	P=	3702.84	(kips)	Input:	P=	3628.79	(kips)
	e=	0.134	(in)		e=	0.134	(in)
	M=	201.03	(k-ft)		M=	174.46	(k-ft)
	f_a=	60000	(psf)		f_a=	60000	(psf)
Trial Sizing:	A=	66.00	(sf)	Trial Sizing:	A=	63.62	(sf)
	Ø=	9.17	(ft)		Ø=	9.00	(ft)
Capacity	f_{max}=	58816.41	(psf)	Capacity	f_{max}=	59535.23	(psf)
	$<f_a$?	OK			$<f_a$?	OK	
Comparison	Ø New =	9.17	(psf)	Comparison	Ø New =	9.00	(psf)
	Ø Orig. =	8	(psf)		Ø Orig. =	8	(psf)
	% Increase	14.59	%		% Increase	12.50	%

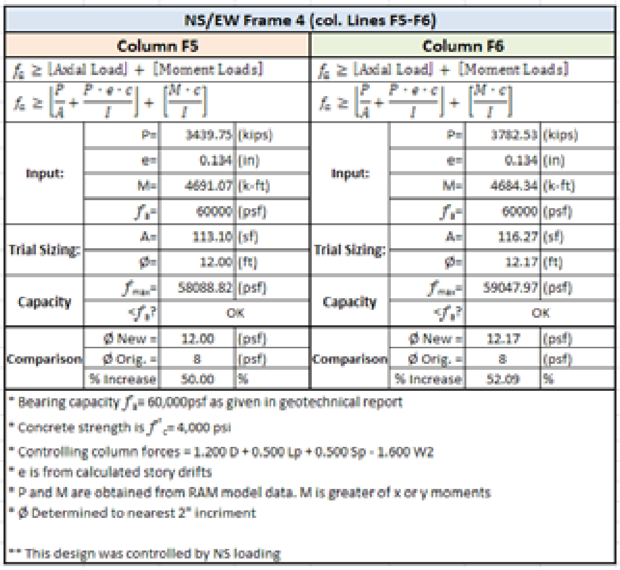
* Bearing capacity f_a= 60,000psf as given in geotechnical report
* Concrete strength is f'_c= 4,000 psi
* Controlling column forces = 1.200 D + 0.500 Lp + 0.500 Sp - 1.600 W2
* e is from calculated story drifts
* P and M are obtained from RAM model data. M is greater of x or y moments
* Ø Determined to nearest 2" incriment



NS/EW Frame 3 (col. Lines E5-E6)							
Column E5				Column E6			
$f_c \geq$ [Axial Load] + [Moment Loads]				$f_c \geq$ [Axial Load] + [Moment Loads]			
$f_c \geq \left[\frac{P}{A}+\frac{P\cdot e\cdot c}{I}\right]+\left[\frac{M\cdot c}{I}\right]$				$f_c \geq \left[\frac{P}{A}+\frac{P\cdot e\cdot c}{I}\right]+\left[\frac{M\cdot c}{I}\right]$			
Input:	P=	3597.02	(kips)	Input:	P=	3818.21	(kips)
	e=	0.134	(in)		e=	0.134	(in)
	M=	4648.1	(k-ft)		M=	4675.78	(k-ft)
	f_s=	60000	(psf)		f_s=	60000	(psf)
Trial Sizing:	A=	113.10	(sf)	Trial Sizing:	A=	129.34	(sf)
	Ø=	12.00	(ft)		Ø=	12.83	(ft)
Capacity	f_{max}=	59227.13	(psf)	Capacity	f_{max}=	52075.98	(psf)
	<f_s?	OK			<f_s?	OK	
Comparison	Ø New =	12.00	(psf)	Comparison	Ø New =	12.83	(psf)
	Ø Orig. =	8	(psf)		Ø Orig. =	8	(psf)
	% Increase	50.00	%		% Increase	60.41	%

* Bearing capacity f_s= 60,000psf as given in geotechnical report
* Concrete strength is f'_c= 4,000 psi
* Controlling column forces = 1.200 D + 0.500 Lp + 0.500 Sp - 1.600 W2
* e is from calculated story drifts
* P and M are obtained from RAM model data. M is greater of x or y moments
* Ø Determined to nearest 2" incriment

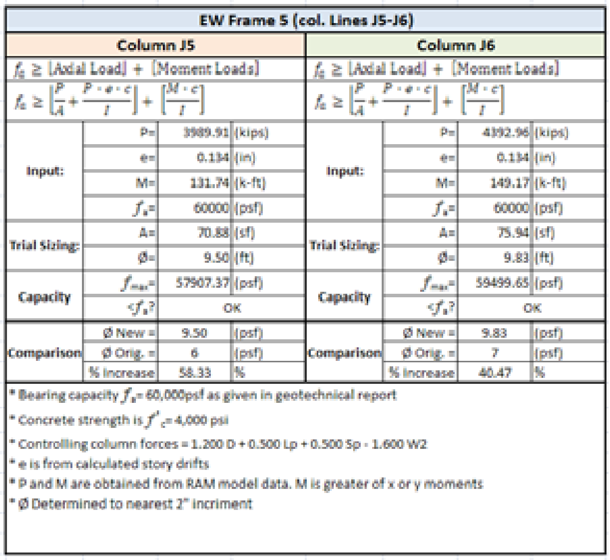
** This design was controlled by NS loading



NS/EW Frame 4 (col. Lines F5-F6)							
Column F5				Column F6			
$f_c \geq$ [Axial Load] + [Moment Loads]				$f_c \geq$ [Axial Load] + [Moment Loads]			
$f_c \geq \left[\frac{P}{A}+\frac{P\cdot e\cdot c}{I}\right]+\left[\frac{M\cdot c}{I}\right]$				$f_c \geq \left[\frac{P}{A}+\frac{P\cdot e\cdot c}{I}\right]+\left[\frac{M\cdot c}{I}\right]$			
Input:	P=	3439.75	(kips)	Input:	P=	3782.53	(kips)
	e=	0.134	(in)		e=	0.134	(in)
	M=	4691.07	(k-ft)		M=	4684.34	(k-ft)
	f_s=	60000	(psf)		f_s=	60000	(psf)
Trial Sizing:	A=	113.10	(sf)	Trial Sizing:	A=	116.27	(sf)
	Ø=	12.00	(ft)		Ø=	12.17	(ft)
Capacity	f_{max}=	58088.82	(psf)	Capacity	f_{max}=	59047.97	(psf)
	<f_s?	OK			<f_s?	OK	
Comparison	Ø New =	12.00	(psf)	Comparison	Ø New =	12.17	(psf)
	Ø Orig. =	8	(psf)		Ø Orig. =	8	(psf)
	% Increase	50.00	%		% Increase	52.09	%

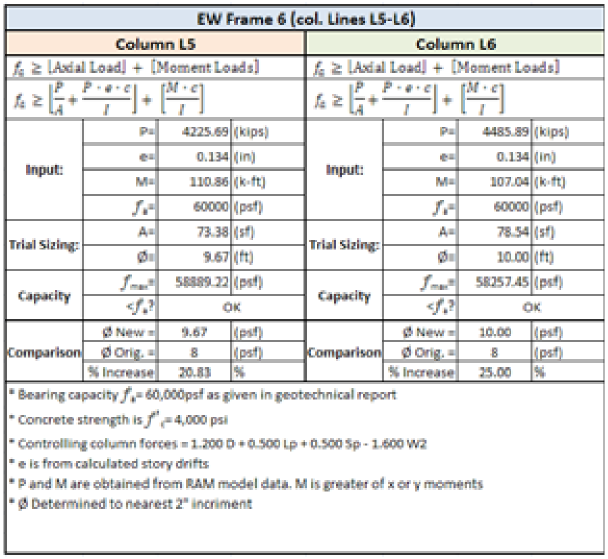
* Bearing capacity f_s= 60,000psf as given in geotechnical report
* Concrete strength is f'_c= 4,000 psi
* Controlling column forces = 1.200 D + 0.500 Lp + 0.500 Sp - 1.600 W2
* e is from calculated story drifts
* P and M are obtained from RAM model data. M is greater of x or y moments
* Ø Determined to nearest 2" incriment

** This design was controlled by NS loading



EW Frame 5 (col. Lines J5-J6)							
Column J5				Column J6			
$f_c \geq$ [Axial Load] + [Moment Loads]				$f_c \geq$ [Axial Load] + [Moment Loads]			
$f_c \geq \left[\frac{P}{A}+\frac{P\cdot e\cdot c}{I}\right]+\left[\frac{M\cdot c}{I}\right]$				$f_c \geq \left[\frac{P}{A}+\frac{P\cdot e\cdot c}{I}\right]+\left[\frac{M\cdot c}{I}\right]$			
Input:	P=	3989.91	(kips)	Input:	P=	4392.96	(kips)
	e=	0.134	(in)		e=	0.134	(in)
	M=	131.74	(k-ft)		M=	149.17	(k-ft)
	f_s=	60000	(psf)		f_s=	60000	(psf)
Trial Sizing:	A=	70.88	(sf)	Trial Sizing:	A=	75.94	(sf)
	Ø=	9.50	(ft)		Ø=	9.83	(ft)
Capacity	f_{max}=	57907.37	(psf)	Capacity	f_{max}=	59499.65	(psf)
	<f_s?	OK			<f_s?	OK	
Comparison	Ø New =	9.50	(psf)	Comparison	Ø New =	9.83	(psf)
	Ø Orig. =	6	(psf)		Ø Orig. =	7	(psf)
	% Increase	58.33	%		% Increase	40.47	%

* Bearing capacity f_s= 60,000psf as given in geotechnical report
* Concrete strength is f'_c= 4,000 psi
* Controlling column forces = 1.200 D + 0.500 Lp + 0.500 Sp - 1.600 W2
* e is from calculated story drifts
* P and M are obtained from RAM model data. M is greater of x or y moments
* Ø Determined to nearest 2" incriment



EW Frame 6 (col. Lines L5-L6)							
Column L5				Column L6			
$f_c \geq$ [Axial Load] + [Moment Loads]				$f_c \geq$ [Axial Load] + [Moment Loads]			
$f_c \geq \left[\frac{P}{A}+\frac{P\cdot e\cdot c}{I}\right]+\left[\frac{M\cdot c}{I}\right]$				$f_c \geq \left[\frac{P}{A}+\frac{P\cdot e\cdot c}{I}\right]+\left[\frac{M\cdot c}{I}\right]$			
Input:	P=	4225.69	(kips)	Input:	P=	4485.89	(kips)
	e=	0.134	(in)		e=	0.134	(in)
	M=	110.86	(k-ft)		M=	107.04	(k-ft)
	f_s=	60000	(psf)		f_s=	60000	(psf)
Trial Sizing:	A=	73.38	(sf)	Trial Sizing:	A=	78.54	(sf)
	Ø=	9.67	(ft)		Ø=	10.00	(ft)
Capacity	f_{max}=	58889.22	(psf)	Capacity	f_{max}=	58257.45	(psf)
	<f_s?	OK			<f_s?	OK	
Comparison	Ø New =	9.67	(psf)	Comparison	Ø New =	10.00	(psf)
	Ø Orig. =	8	(psf)		Ø Orig. =	8	(psf)
	% Increase	20.83	%		% Increase	25.00	%

* Bearing capacity f_s= 60,000psf as given in geotechnical report
* Concrete strength is f'_c= 4,000 psi
* Controlling column forces = 1.200 D + 0.500 Lp + 0.500 Sp - 1.600 W2
* e is from calculated story drifts
* P and M are obtained from RAM model data. M is greater of x or y moments
* Ø Determined to nearest 2" incriment

Lateral Conclusions & Economics Analysis

Economics Analysis

An economic analysis can be performed between the new and original systems by comparing the unit cost of the two design and by determining the percent increase in cost. Cost is expressed in tonage of steel required; therefore the unit cost is the ratio of steel weight to square footage. Although a full cost breakdown of the $150 million MHSC project would be more accurate, this approach provides a rough estimate in determining the efficiency and effectiveness of the new system. The table on the right lists square footages and unit weights per each level, the resulting % increase in price, and the totals for theses items. After reviewing this chart, it quickly becomes apparent that the design saves money. Most of the floors saw a decrease in unit weight, bringing the total % increase in floor area cost to -6%. This is especially impressive when one considers that the floor area increased by 27%.

Lateral Takeoff Economic Analysis						
		Original Design		New Design		
Level	Area (SF)	Weight lbs	Unit Weight (psf)	Weight lbs	Unit Weight (psf)	% Increase
1st	57000	270998	4.754351	288250	5.057018	6%
2nd	51800	193843	3.742143	174901	3.376467	-10%
3rd	45000	172399	3.831089	158733	3.5274	-8%
4th	57000	171811	3.014228	161118	2.826632	-6%
5th	28000	154833	5.52975	156054	5.573357	1%
6th	28000	142528	5.090286	134519	4.80425	-6%
7th	28000	130258	4.652071	125150	4.469643	-4%
8th	28000	129233	4.615464	124179	4.434964	-4%
9th	28000	120808	4.314571	123486	4.410214	2%
10th	28000	120578	4.306357	123486	4.410214	2%
11th	28000	114389	4.085321	122956	4.391286	7%
12th	25197		0	125545	4.982538	
13th	25197		0	72009	2.85784	
14th	25197		0	71535	2.839028	
15th	25197		0	71535	2.839028	
16th	25197		0	71535	2.839028	
Pent.	25197	123447	4.899274	71535	2.839028	-42%
Mez.	7500	46022	6.136267	55083	7.3444	20%
Roof	22685	36309	1.600573	66507	2.931761	83%
Average:	588167	1927456	4.17034	2298116	3.907251	
% Increase in floor area cost					-6%	
% Increase in floor area					27%	

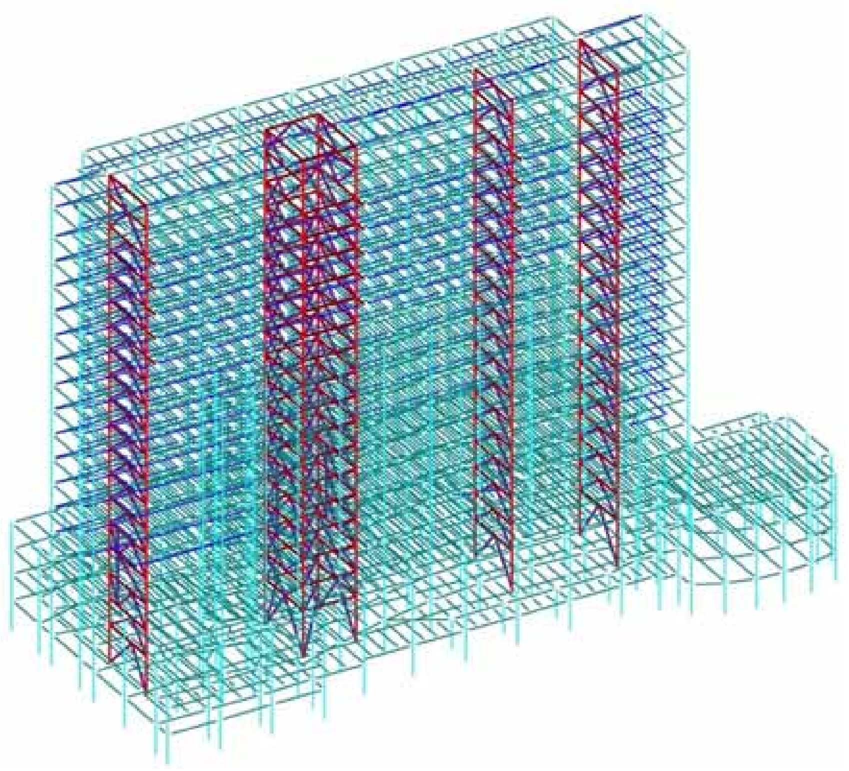
It is important to note that this estimate, which is based off of tonnage, does not take into account the cost of connections. The cost of a moment connection can be estimated by multiplying the joint penetration weld costs of approximately 1.6 lb/ft with the average of 2.5 ft of welding required for a typical Wx14, which results in 4 lb of steel per moment connection. This can add significant costs to a building like the original MHSC design, where moment frames run along the entire edge of the building. Therefore, this lateral system cost estimate is actually conservative and greater savings can be obtained. It is clear that the overall design is a cost saving approach to adding square footage to the building program, and that the decision to change the moment frames to braced frames was a success.

Lateral Conclusions

The lateral system was designed using the RAM Structural system software. An initial trial sizing of the lateral system was made based off of the gravity analysis, which was followed by a strength design. Once the members were sized for strength, several design irritations for drift were performed. After a check with STAAD to verify the RAM model accuracy, a final design of the lateral system was completed. It became clear that the overall framing design was controlled by the N-S frame, most likely due to the torsion caused by the orientation of these column's weak axes to E-W loading. After a strength check on the final drift design, manual verification was completed by comparing the frames to the existing original design, and by comparing the change in size of columns between the new and original designs.

Afterwards, frame foundations were sized, based off of the bedrock bearing capacity, yielding foundations that were 15% to 50% larger than the original designs, which is a reasonable size increase, considering the dramatically increased gravity and wind loads.

The economics analysis made clear that the new lateral system is a more efficient system, despite the large size of the members for the N-S braced frame. The total % increase in floor area cost is -6%, which is a dramatic savings, especially since it does not include the cost savings associates with eliminating moment connections.



Gravity Design

Goals & Process

This section begins with a description of the design approach taken to choose the framing system for the cantilevered area described in the Architectural Breadth section. Chronologically, this step was actually taken before the lateral or gravity analysis was completed; however, its description fits best within the context of the gravity design.

Now that the lateral analysis is complete and the RAM model has been updated, it is possible to run a final gravity design analysis. A detailed listing of finalized beam and column sizes will not be included, however, because this section is aimed primarily at verifying the RAM gravity design modeling.

The first step in this verification process is hand calculations involving the design of the basement columns along a single column line, which will then be compared to the “existing” original column design with the “RAM” original design. This analysis consists of using tributary area and the design loads previously determined in the beginning of the Structural Depth section to design the columns via the column design Table 4-1 of the AISC Steel Construction Manual. This comparison will make evident any discrepancies between the existing original design and the computer models, and provide a check for the validity of the computer models results.

The second form of verification is a comparison of the RAM original design and the RAM new design. The previous verification already established that the size are within the range of the original existing model; however this check confirms the results of the two RAM models and provides a means of comparing the gravity system design.

After the verifications, an economics analysis similar to the one conducted in the Lateral section will be carried out to determine the effectiveness of the system. This will be followed by a conclusion section summarizing the gravity analysis results.

Cantilever Area Design

A method for attaching the cantilevered section of the framing system - highlighted in blue on the plan below – needed to be determined. Several options were initially considered, including a hanging structure, moment connections, or continuous girders. It was clear early on that a hanging structure would be infeasible due to the negative impacts it would have on the existing architectural design. The better of the remaining two options was picked by a rough cost estimate of the different systems using equivalent steel weights.

The continuous girder system would require a column splice at each level so that the girder could pass through creating the cantilever. Average cost of a column splice is approximately 500 lb/ per column. With 6 levels, and 10 columns per floor, this would add 30,000 lbs of steel to the project.

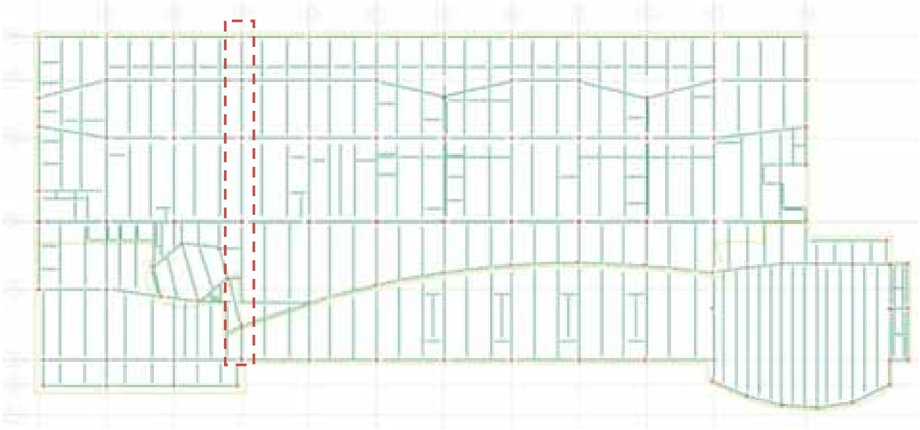
The cost of a moment connection can be estimated by multiplying the joint penetration weld costs of approximately 1.6 lb/ft with the average of 2.5 ft of welding required for a typical Wx14, which results in 4 lb of steel per moment connection. If the girder is connected to the column flange, stiffener plates will be required to resist web compression. These can cost up to 300 lb/ per pair of sitffeners. The total cost runs at 18,480 lbs of steel with two moment connection per beam and a pair of web stiffeners.

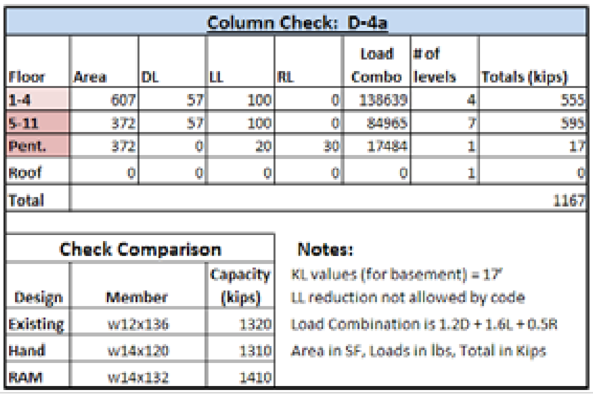
At nearly half the cost of the continuous girder system, a cantilevered framing area supported by moment connections is clearly the better choice.

Verifications

Hand Check

The first step in this verification process is a comparison of the "existing" original column design with the "RAM" original design with hand calculations. A single column line –not part of the lateral system - of basement columns was used to run the verification, as highlighted in the image below. This analysis consists of using tributary area and the design loads previously determined in the beginning of the Structural Depth section. The controlling load combination of 1.2D + 1.6L + 0.5R was then used to obtain total loads, which were used in conjunction with an effective length (KL) value of 17' to design the columns according to column design Table 4-1 of the AISC Steel Construction Manual. Finally these column sizes were compared to the "existing" original design, and the RAM" original design, to verify the relative accuracy of the RAM model. After reviewing the column checks it becomes apparent that the columns are typically within the same size range, with the existing columns typically larger. This is attributed to the fact that the original existing floor design was designed for vibration control; however, this criterion was not included in the RAM gravity model creating a slightly lighter structural system. Other discrepancies can also be attributed to modeling differences. It is important to note that despite these differences; an accurate lateral system was designed, which was the primary intent of the structural breadth.





Column Check: D-4a

Floor	Area	DL	LL	RL	Load Combo	# of levels	Totals (kips)
1-4	607	57	100	0	138639	4	555
5-11	372	57	100	0	84965	7	595
Pent.	372	0	20	30	17484	1	17
Roof	0	0	0	0	0	1	0
Total							1167

Check Comparison

Design	Member	Capacity (kips)
Existing	w12x136	1320
Hand	w14x120	1310
RAM	w14x132	1410

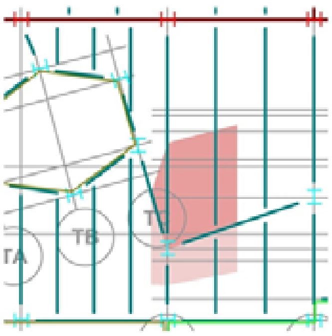
Notes:

KL values (for basement) = 17'

LL reduction not allowed by code

Load Combination is 1.2D + 1.6L + 0.5R

Area in SF, Loads in lbs, Total in Kips



* Light shade indicates areas that terminate before roof level

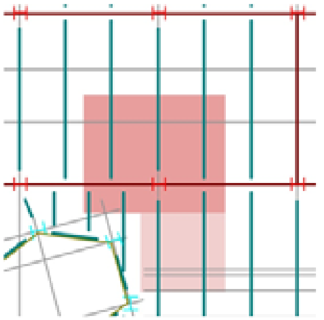
Column Check: D-5

Floor	Area	DL	LL	RL	Load Combo	# of levels	Totals (kips)
1-4	1134	57	100	0	259006	4	1036
5-11	1134	57	100	0	259006	7	1813
Pent.	1134	57	150	0	349726	1	350
Roof	1039	57	20	30	119901	1	120
Total							3319

Check Comparison

Design	Member	Capacity (kips)
Existing	w14x370	4150
Hand	w14x311	3460
RAM	-	-

Notes:
KL values (for basement) = 17'
LL reduction not allowed by code
Load Combination is 1.2D + 1.6L + 0.5R
Area in SF, Loads in lbs, Total in Kips
*Existing size was used in RAM design



* Light shade indicates areas that terminate before roof level

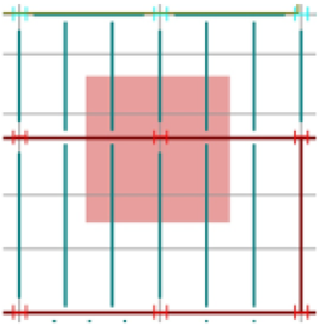
Column Check: D-6

Floor	Area	DL	LL	RL	Load Combo	# of levels	Totals (kips)
1-4	1008	57	100	0	230227	4	921
5-11	1008	57	100	0	230227	7	1612
Pent.	1008	57	150	0	310867	1	311
Roof	1008	57	20	30	116323	1	116
Total							2960

Check Comparison

Design	Member	Capacity (kips)
Existing	w14x342	3840
Hand	w14x283	3150
RAM	-	-

Notes:
KL values (for basement) = 17'
LL reduction not allowed by code
Load Combination is 1.2D + 1.6L + 0.5R
Area in SF, Loads in lbs, Total in Kips
*Existing size was used in RAM design



* Light shade indicates areas that terminate before roof level

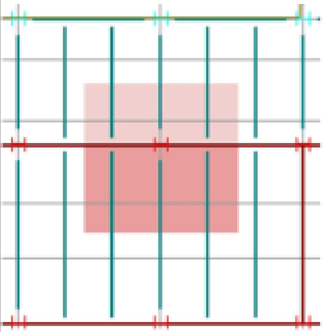
Column Check: D-7

Floor	Area	DL	LL	RL	Load Combo	# of levels	Totals (kips)
1	419	57	100	0	95700	3	287
2-3	729	57	100	0		1	0
5-11	574	57	100	0	131102	7	918
Pent.	574	57	150	0	177022	1	177
Roof	419	57	20	30	48353	1	48
Total							1430

Check Comparison

Design	Member	Capacity (kips)
Existing	w14x233	2570
Hand	w14x145	1580
RAM	w14x68	544

Notes:
KL values (for basement) = 17'
LL reduction not allowed by code
Load Combination is 1.2D + 1.6L + 0.5R
Area in SF, Loads in lbs, Total in Kips
*Difference due to modeling. See report



* Light shade indicates areas that terminate before roof level

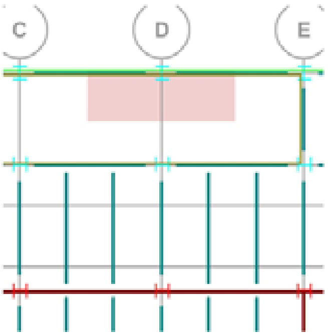
Column Check: D-8

Floor	Area	DL	LL	RL	Load Combo	# of levels	Totals (kips)
1	0	57	100	0	0	1	0
2-4	310	57	100	0	70804	3	212
5-11	0	57	100	0	0	7	0
Pent.	0	57	150	0	0	1	0
Roof	0	57	20	30	0	1	0
Total							212

Check Comparison

Design	Member	Capacity (kips)
Existing	w12x65	615
Hand	w12x40	234
RAM	w14x90	954

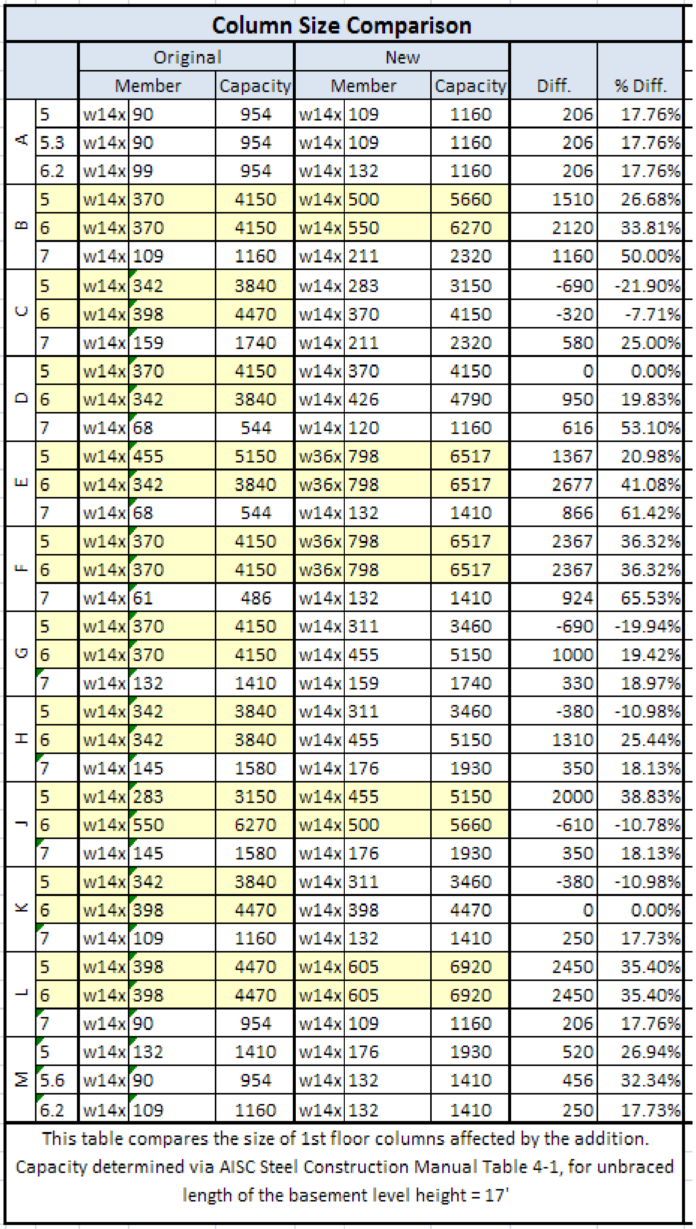
Notes:
KL values (for basement) = 17'
LL reduction not allowed by code
Load Combination is 1.2D + 1.6L + 0.5R
Area in SF, Loads in lbs, Total in Kips
*Difference due to modeling. See report



* Light shade indicates areas that terminate before roof level

Column Comparison

The second form of verification is a comparison of the RAM original design and the RAM new design. The previous verification already established that the sizes are within the range of the original existing model; however this check confirms the results of the two RAM models and provides a means of comparing the gravity system design. This table is limited to those columns which are affected by the building addition with columns partaking in a lateral system highlighted in yellow. As expected, most of the sizes went up, with the exception of those columns which were previously part of the moment frame system. Since these were not longer required to carry lateral loading, their sizes could be drastically reduced, some by 20%. Columns which are part of the braced frame system dramatically increased in size, especially those part of the N-S framing system, E/5-6 and F/5-6.



Column Size Comparison

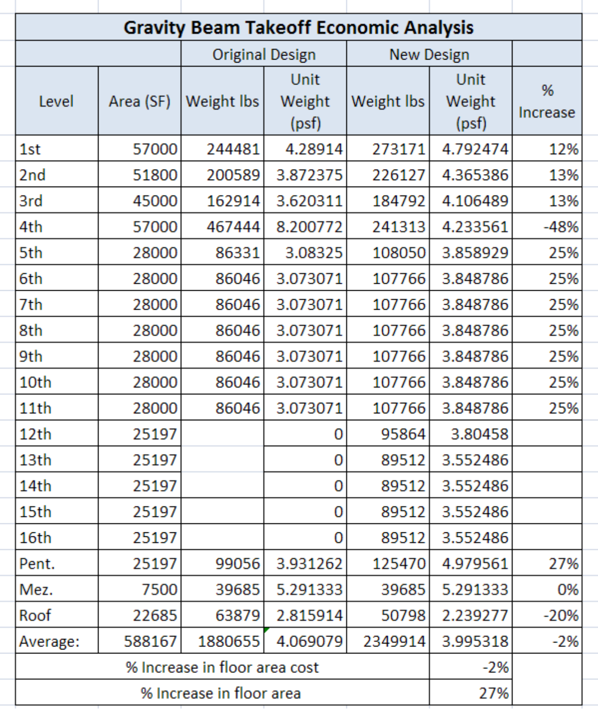
		Original			New				
		Member		Capacity	Member		Capacity	Diff.	% Diff.
A	5	w14x	90	954	w14x	109	1160	206	17.76%
	5.3	w14x	90	954	w14x	109	1160	206	17.76%
	6.2	w14x	99	954	w14x	132	1160	206	17.76%
B	5	w14x	370	4150	w14x	500	5660	1510	26.68%
	6	w14x	370	4150	w14x	550	6270	2120	33.81%
	7	w14x	109	1160	w14x	211	2320	1160	50.00%
C	5	w14x	342	3840	w14x	283	3150	-690	-21.90%
	6	w14x	398	4470	w14x	370	4150	-320	-7.71%
	7	w14x	159	1740	w14x	211	2320	580	25.00%
D	5	w14x	370	4150	w14x	370	4150	0	0.00%
	6	w14x	342	3840	w14x	426	4790	950	19.83%
	7	w14x	68	544	w14x	120	1160	616	53.10%
E	5	w14x	455	5150	w36x	798	6517	1367	20.98%
	6	w14x	342	3840	w36x	798	6517	2677	41.08%
	7	w14x	68	544	w14x	132	1410	866	61.42%
F	5	w14x	370	4150	w36x	798	6517	2367	36.32%
	6	w14x	370	4150	w36x	798	6517	2367	36.32%
	7	w14x	61	486	w14x	132	1410	924	65.53%
G	5	w14x	370	4150	w14x	311	3460	-690	-19.94%
	6	w14x	370	4150	w14x	455	5150	1000	19.42%
	7	w14x	132	1410	w14x	159	1740	330	18.97%
H	5	w14x	342	3840	w14x	311	3460	-380	-10.98%
	6	w14x	342	3840	w14x	455	5150	1310	25.44%
	7	w14x	145	1580	w14x	176	1930	350	18.13%
J	5	w14x	283	3150	w14x	455	5150	2000	38.83%
	6	w14x	550	6270	w14x	500	5660	-610	-10.78%
	7	w14x	145	1580	w14x	176	1930	350	18.13%
K	5	w14x	342	3840	w14x	311	3460	-380	-10.98%
	6	w14x	398	4470	w14x	398	4470	0	0.00%
	7	w14x	109	1160	w14x	132	1410	250	17.73%
L	5	w14x	398	4470	w14x	605	6920	2450	35.40%
	6	w14x	398	4470	w14x	605	6920	2450	35.40%
	7	w14x	90	954	w14x	109	1160	206	17.76%
M	5	w14x	132	1410	w14x	176	1930	520	26.94%
	5.6	w14x	90	954	w14x	132	1410	456	32.34%
	6.2	w14x	109	1160	w14x	132	1410	250	17.73%

This table compares the size of 1st floor columns affected by the addition. Capacity determined via AISC Steel Construction Manual Table 4-1, for unbraced length of the basement level height = 17'

Gravity Conclusions & Economics Analysis

Economics Analysis

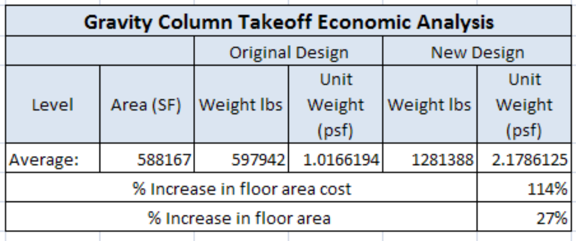
An economic analysis can be performed between the new and original systems by comparing the unit cost of the two design and by determining the percent increase in cost. Cost is expressed in tonage of steel required; therefore the unit cost is the ratio of steel weight to square footage. Although a full cost breakdown of the $150 million MHSC project would be more accurate, this approach provides a rough estimate in determining the efficiency and effectiveness of the new system. The table on the right lists square footages and unit weights per each level, the resulting % increase in price, and the totals for theses items.



Gravity Beam Takeoff Economic Analysis

		Original Design		New Design		
Level	Area (SF)	Weight lbs	Unit Weight (psf)	Weight lbs	Unit Weight (psf)	% Increase
1st	57000	244481	4.28914	273171	4.792474	12%
2nd	51800	200589	3.872375	226127	4.365386	13%
3rd	45000	162914	3.620311	184792	4.106489	13%
4th	57000	467444	8.200772	241313	4.233561	-48%
5th	28000	86331	3.08325	108050	3.858929	25%
6th	28000	86046	3.073071	107766	3.848786	25%
7th	28000	86046	3.073071	107766	3.848786	25%
8th	28000	86046	3.073071	107766	3.848786	25%
9th	28000	86046	3.073071	107766	3.848786	25%
10th	28000	86046	3.073071	107766	3.848786	25%
11th	28000	86046	3.073071	107766	3.848786	25%
12th	25197		0	95864	3.80458	
13th	25197		0	89512	3.552486	
14th	25197		0	89512	3.552486	
15th	25197		0	89512	3.552486	
16th	25197		0	89512	3.552486	
Pent.	25197	99056	3.931262	125470	4.979561	27%
Mez.	7500	39685	5.291333	39685	5.291333	0%
Roof	22685	63879	2.815914	50798	2.239277	-20%
Average:	588167	1880655	4.069079	2349914	3.995318	-2%
% Increase in floor area cost					-2%	
% Increase in floor area					27%	

The drastic increase in the gravity frame costs can be associated with the fact that when the lateral system was changed from braced to lateral, a significant number of high weight beams were moved from the lateral to the gravity takeoff. With that said, their is still a decrease in total percent change in cost, reinforcing the statement within the lateral section that they new design is more efficient.



Gravity Column Takeoff Economic Analysis

		Original Design		New Design	
Level	Area (SF)	Weight lbs	Unit Weight (psf)	Weight lbs	Unit Weight (psf)
Average:	588167	597942	1.0166194	1281388	2.1786125
% Increase in floor area cost					114%
% Increase in floor area					27%

The increase in column cost is forseeable since there is simply no opportunity to save weight in this area: The gravity columns simply must be added. When this is taken to consideration with the savings gained by the lateral system and gravity beams, it becomes clear that the overall design is still an efficient, cost saving approach to add square footage to the building program.

Gravity Conclusions

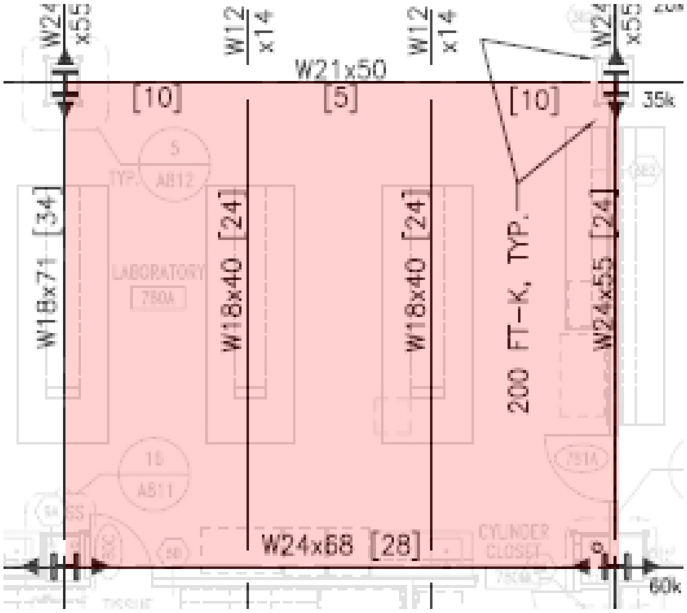
After analysing the hand calculations, it is clear that there is a slight inconsistancy between the RAM model and the "existing" model, most likely due to the heavier floor of the existing system which was specifically designed for vibration control: however, the second column comparison highlights the important fact that this error is consistant, thus verifying the design results and allowing the comparison of the "original" and "new"models.

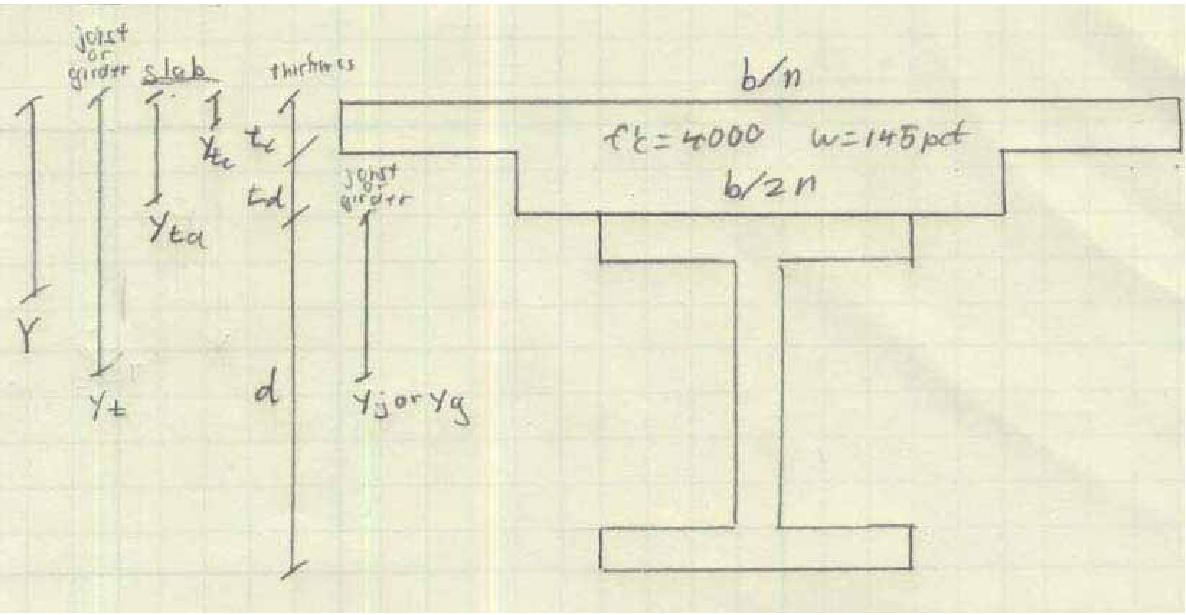
The final gravity economics analysis can be combined with the existing lateral analysis to reach the final conclusion about the structural redesign: that the addition of square footage to an existing midrise building is an efficient means to increasing program capacity, and that the simplification from moment to braced frames as a an effective means to obtain cost savings.

Vibration Check

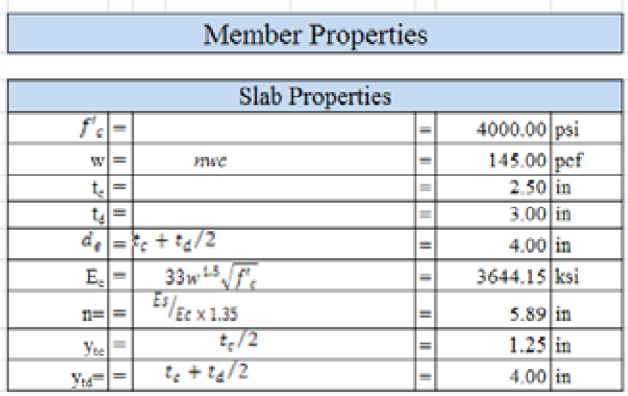
The final step within the Structural Breadth section is to perform a vibration analysis of the existing floor system. The MHSC contains many laboratory spaces with sensitive equipment, which may not function properly when exposed to vibrations caused by walking excitation. Vibration control of the floor system can dramatically affect the floor sizing requirements, including slab thickness and weights, and beam and girder sizing. An analysis of floor system – based off the Steel Design Guide Series 11: Floor Vibration Due to Human Activity – will shed light on the "existing" floor systems design intent, and determine the suitability of the floor system for various laboratory applications.

A typical bay, depicted below within the context of the floor plan and enlarged on the next page was used for the analysis, which appears on the following pages. This bay was chosen since most of the laboratory equipment is located here. Next to the enlarged plan is a section of the beam/joist, with the member properties used during the analysis labeled.


W24
x55
W12
x14
W21x50
W12
x14
W24
x55
[10]
[5]
[10]
35k
W18x71 [34]
W18x40 [24]
W18x40 [24]
200 FT–K, TYP.
W24x55 [24]
LABORATORY
760A
W24x68 [28]
CYLINDER
CLOSET
60k


joist
or
girder
slab
thickness
b/n
f'c=4000
w=145pcf
b/2n
Ytc
tc
td
Yta
joist
or
girder
Y
Yt
d
Yj or Yg

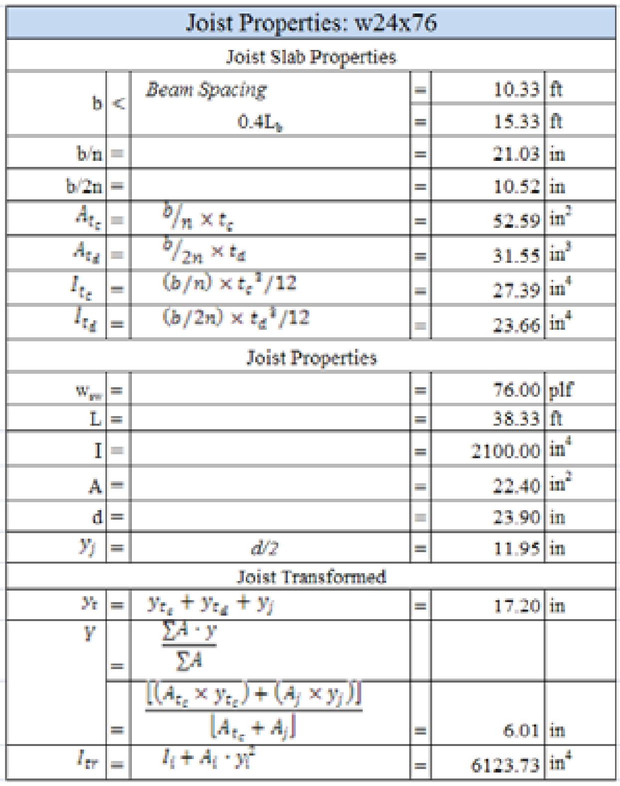
The design process is presented in order of the calculations made, starting with "Member Properties" which determines the "Panel Natural Frequency." This is followed by the "Mid Span Flexibilities" and ends with "Expected Maximum Velocity due to Walking." Finally a chart comparing the calculated vibration velocity values to the allowed values concludes for which types of facility or equipment use the floor system is suitable for. Design assumptions are noted as they appear in the calculations.



Member Properties

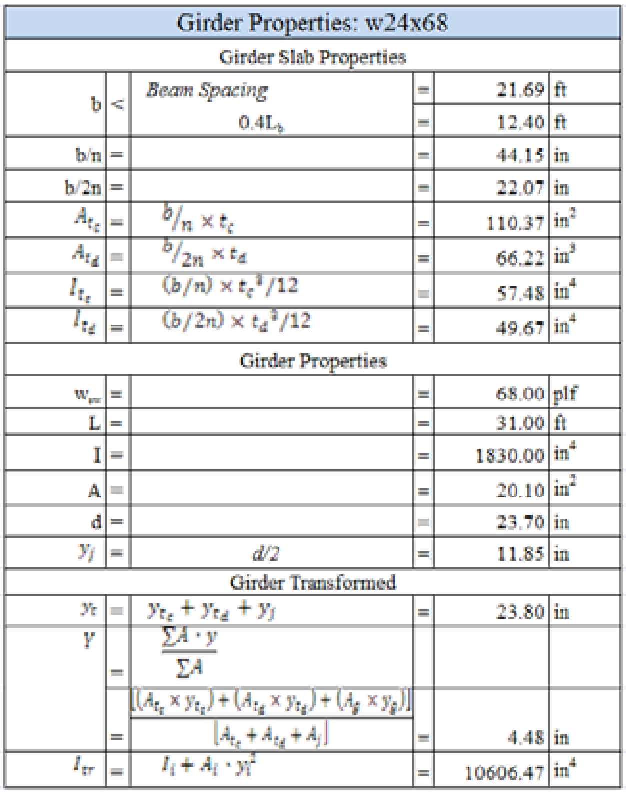
Slab Properties

f'_c	=		=	4000.00	psi
w	=	nwc	=	145.00	pcf
t_c	=		=	2.50	in
t_d	=		=	3.00	in
d_e	=	$t_c + t_d/2$	=	4.00	in
E_c	=	$33w^{1.5}\sqrt{f'_c}$	=	3644.15	ksi
n=	=	$^{Es}/_{Ec \times 1.35}$	=	5.89	in
y_{tc}	=	$t_c/2$	=	1.25	in
y_{td}=	=	$t_c + t_d/2$	=	4.00	in



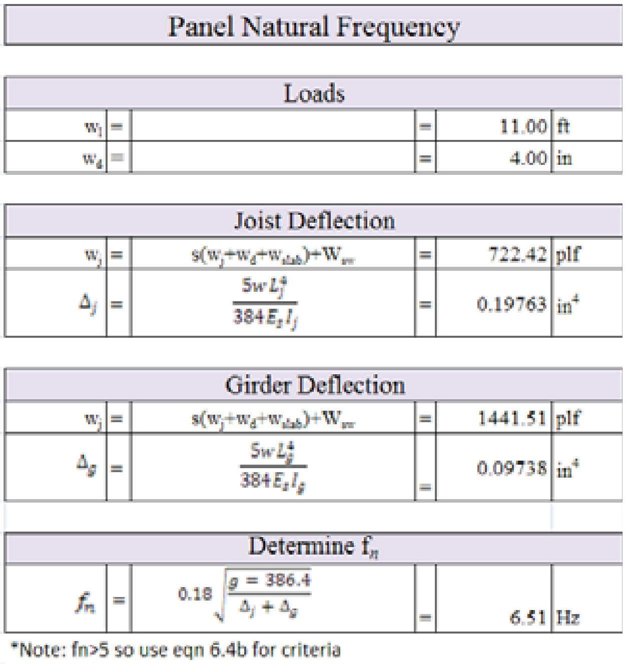
Joist Properties: w24x76

Joist Slab Properties					
b	<	*Beam Spacing*	=	10.33	ft
		$0.4L_b$	=	15.33	ft
b/n	=		=	21.03	in
b/2n	=		=	10.52	in
A_{t_c}	=	$^b/_n \times t_c$	=	52.59	in^2
A_{t_d}	=	$^b/_{2n} \times t_d$	=	31.55	in^3
I_{t_c}	=	$(b/n) \times t_c^3/12$	=	27.39	in^4
I_{t_d}	=	$(b/2n) \times t_d^3/12$	=	23.66	in^4
Joist Properties					
w_{sw}	=		=	76.00	plf
L	=		=	38.33	ft
I	=		=	2100.00	in^4
A	=		=	22.40	in^2
d	=		=	23.90	in
y_j	=	d/2	=	11.95	in
Joist Transformed					
y_t	=	$y_{t_c} + y_{t_d} + y_j$	=	17.20	in
y	=	$\frac{\sum A \cdot y}{\sum A}$			
	=	$\frac{[(A_{t_c} \times y_{t_c}) + (A_j \times y_j)]}{[A_{t_c} + A_j]}$	=	6.01	in
I_{tr}	=	$I_i + A_i \cdot y_i^2$	=	6123.73	in^4



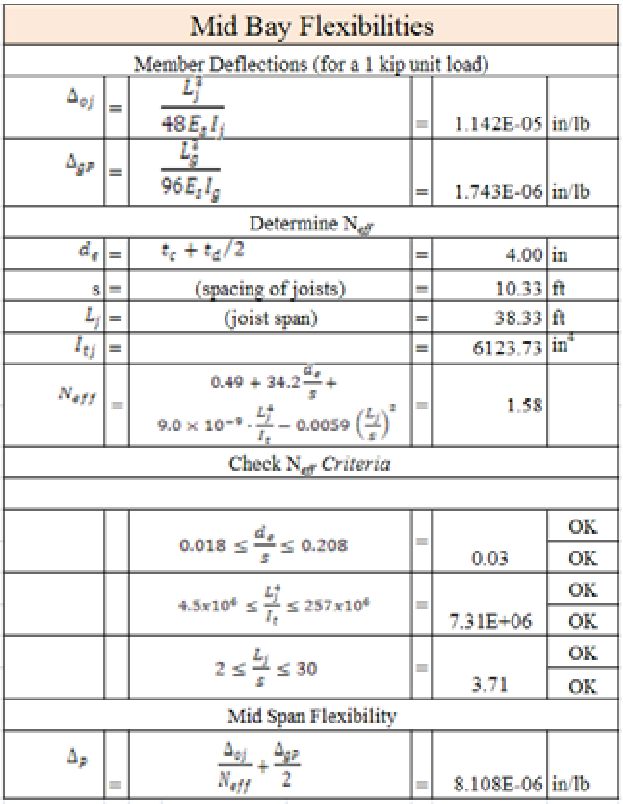
Girder Properties: w24x68

Girder Slab Properties					
b	<	*Beam Spacing*	=	21.69	ft
		$0.4L_b$	=	12.40	ft
b/n	=		=	44.15	in
b/2n	=		=	22.07	in
A_{t_c}	=	$^b/_n \times t_c$	=	110.37	in^2
A_{t_d}	=	$^b/_{2n} \times t_d$	=	66.22	in^3
I_{t_c}	=	$(b/n) \times t_c^3/12$	=	57.48	in^4
I_{t_d}	=	$(b/2n) \times t_d^3/12$	=	49.67	in^4
Girder Properties					
w_{sw}	=		=	68.00	plf
L	=		=	31.00	ft
I	=		=	1830.00	in^4
A	=		=	20.10	in^2
d	=		=	23.70	in
y_j	=	d/2	=	11.85	in
Girder Transformed					
y_t	=	$y_{t_c} + y_{t_d} + y_j$	=	23.80	in
y	=	$\frac{\sum A \cdot y}{\sum A}$			
	=	$\frac{[(A_{t_c} \times y_{t_c}) + (A_{t_d} \times y_{t_d}) + (A_g \times y_g)]}{[A_{t_c} + A_{t_d} + A_j]}$	=	4.48	in
I_{tr}	=	$I_i + A_i \cdot y_i^2$	=	10606.47	in^4

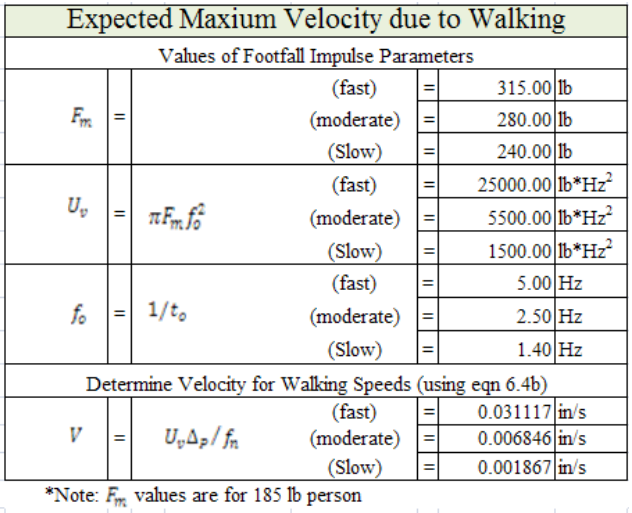


Panel Natural Frequency					
Loads					
w_l	=		=	11.00	ft
w_d	=		=	4.00	in
Joist Deflection					
w_j	=	$s(w_l+w_d+w_{slab})+W_{sw}$	=	722.42	plf
Δ_j	=	$\frac{5wL_j^4}{384E_sI_j}$	=	0.19763	in^4
Girder Deflection					
w_j	=	$s(w_l+w_d+w_{slab})+W_{sw}$	=	1441.51	plf
Δ_g	=	$\frac{5wL_g^4}{384E_sI_g}$	=	0.09738	in^4
Determine f_n					
f_n	=	$0.18\sqrt{\frac{g=386.4}{\Delta_j+\Delta_g}}$	=	6.51	Hz

*Note: fn>5 so use eqn 6.4b for criteria

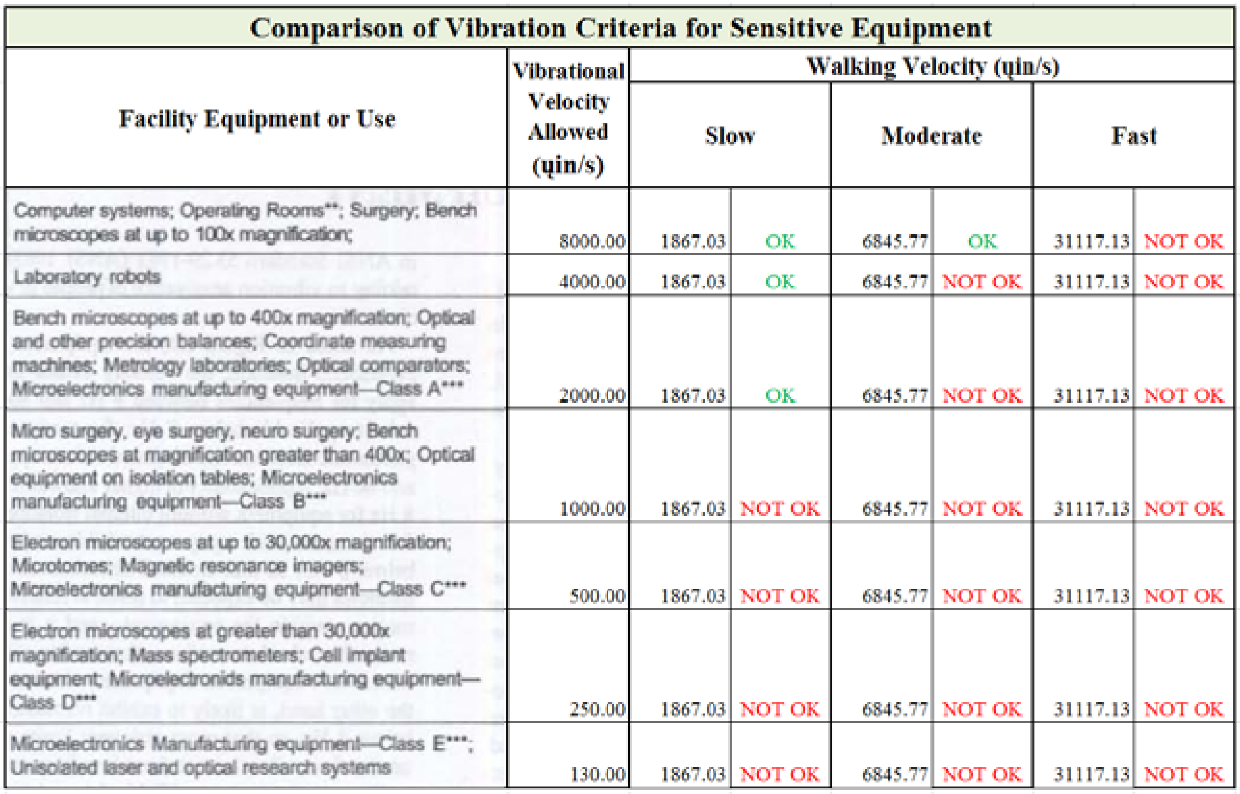


Mid Bay Flexibilities					
Member Deflections (for a 1 kip unit load)					
Δ_{oj}	=	$\frac{L_j^3}{48E_sI_j}$	=	1.142E-05	in/lb
Δ_{gP}	=	$\frac{L_g^3}{96E_sI_g}$	=	1.743E-06	in/lb
Determine N_{eff}					
d_e	=	$t_c+t_d/2$	=	4.00	in
s	=	(spacing of joists)	=	10.33	ft
L_j	=	(joist span)	=	38.33	ft
I_{tj}	=		=	6123.73	in^4
N_{eff}	=	$0.49+34.2\frac{d_e}{s}+9.0\times10^{-9}\cdot\frac{L_j^4}{I_t}-0.0059\left(\frac{L_j}{s}\right)^2$	=	1.58	
Check N_{eff} *Criteria*					
		$0.018\leq\frac{d_e}{s}\leq0.208$	=	0.03	OK OK
		$4.5x10^6\leq\frac{L_j^4}{I_t}\leq257x10^6$	=	7.31E+06	OK OK
		$2\leq\frac{L_j}{s}\leq30$	=	3.71	OK OK
Mid Span Flexibility					
Δ_p	=	$\frac{\Delta_{oj}}{N_{eff}}+\frac{\Delta_{gP}}{2}$	=	8.108E-06	in/lb



Expected Maxium Velocity due to Walking						
Values of Footfall Impulse Parameters						
F_m	=		(fast)	=	315.00	lb
			(moderate)	=	280.00	lb
			(Slow)	=	240.00	lb
U_v	=	$\pi F_m f_o^2$	(fast)	=	25000.00	$lb*Hz^2$
			(moderate)	=	5500.00	$lb*Hz^2$
			(Slow)	=	1500.00	$lb*Hz^2$
f_o	=	$1/t_o$	(fast)	=	5.00	Hz
			(moderate)	=	2.50	Hz
			(Slow)	=	1.40	Hz
Determine Velocity for Walking Speeds (using eqn 6.4b)						
V	=	$U_v\Delta_p/f_n$	(fast)	=	0.031117	in/s
			(moderate)	=	0.006846	in/s
			(Slow)	=	0.001867	in/s

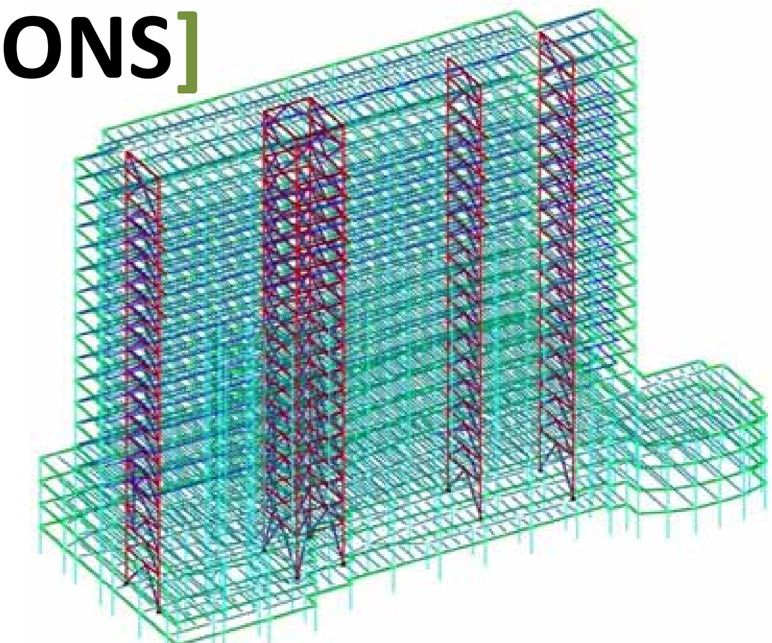
*Note: F_m values are for 185 lb person

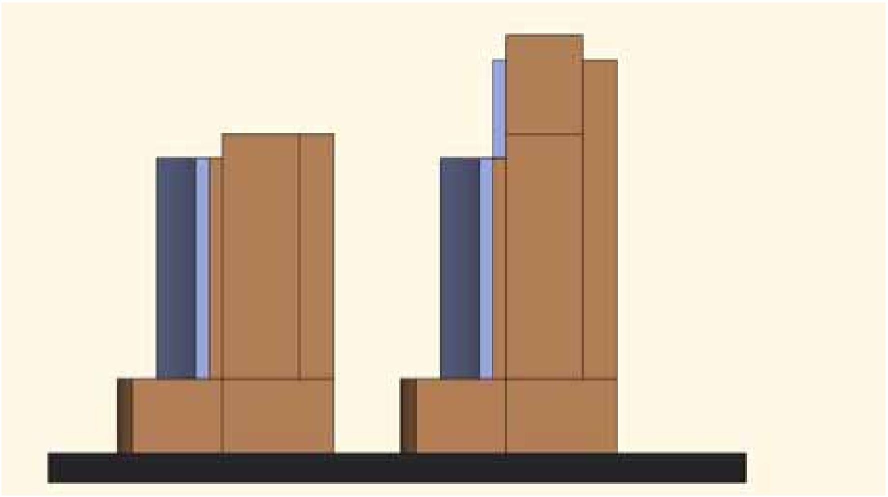


Comparison of Vibration Criteria for Sensitive Equipment							
Facility Equipment or Use	Vibrational Velocity Allowed (μin/s)	Walking Velocity (μin/s)					
		Slow		Moderate		Fast	
Computer systems; Operating Rooms**; Surgery; Bench microscopes at up to 100x magnification;	8000.00	1867.03	OK	6845.77	OK	31117.13	NOT OK
Laboratory robots	4000.00	1867.03	OK	6845.77	NOT OK	31117.13	NOT OK
Bench microscopes at up to 400x magnification; Optical and other precision balances; Coordinate measuring machines; Metrology laboratories; Optical comparators; Microelectronics manufacturing equipment—Class A***	2000.00	1867.03	OK	6845.77	NOT OK	31117.13	NOT OK
Micro surgery, eye surgery, neuro surgery; Bench microscopes at magnification greater than 400x; Optical equipment on isolation tables; Microelectronics manufacturing equipment—Class B***	1000.00	1867.03	NOT OK	6845.77	NOT OK	31117.13	NOT OK
Electron microscopes at up to 30,000x magnification; Microtomes; Magnetic resonance imagers; Microelectronics manufacturing equipment—Class C***	500.00	1867.03	NOT OK	6845.77	NOT OK	31117.13	NOT OK
Electron microscopes at greater than 30,000x magnification; Mass spectrometers; Cell implant equipment; Microelectronids manufacturing equipment—Class D***	250.00	1867.03	NOT OK	6845.77	NOT OK	31117.13	NOT OK
Microelectronics Manufacturing equipment—Class E***; Unisolated laser and optical research systems	130.00	1867.03	NOT OK	6845.77	NOT OK	31117.13	NOT OK

This chart comparison shows that the "existing" floor system is suitable only for basic laboratory equipment and procedures, such as bench microscopes with 100x magnification, surgery and operating procedures. If walking speeds are minimized to a slow pace, microscopes with 400x magnification could be used. This demonstrates that the Temple University Multipurpose Health Science Center's floor system is adequate for less sensitive equipment and procedure requirements, which may meet the university's needs; however, a redesign would be required to accommodate equipment with more stringent vibration requirements.

[CONCLUSIONS]





The Multipurpose Health Science Center (MHSC) is a new, state of the art medical facility within the heart of Philadelphia. The great urban location comes with a primary shortcoming: lack of space, which has prompted the owner to express need for a significant program expansion including the addition of green space; thereby requesting an in-depth architectural, green roof, and structural investigation.

Architectural Breath

By analyzing the existing architectural conditions, a building mass, typical floor plans and framing layouts were created with several design considerations guiding the process, including attention to the scale of the surrounding site and neighborhood; consideration of the original design's massing, programming, cladding, and preexisting design philosophy; optimization of the structural system; and finally a life safety check.

A stepped back massing complimented the scale of the neighborhood and provided to opportunity to fulfill the design philosophy by expressing internal functions externally. The program requirements were successfully met by the addition 126,000 sf program space, incorporating a layout efficiency of 83%.

The structural system was optimized by continuing up the existing structural system vertically and special considerations such as vibration issues and mechanical equipment space allocation were taken into account.

Finally, an egress check insured that the adequate size and number of exits were present to insure the safety of the building's 6,000 potential occupants.

Green Roof Breath

The green roof is a dual intensive and extensive system with a 1400sf accessible garden above the library, with an extensive green roof system covering the remaining 4800 sf of roof. The intensive system is modeled after "Roofscapes, Inc." "Savannah" style roof, which is 6" thick and can sustain supports sedums, meadow grasses and meadow perennials. The shallower "Flower Carpet" system is 3-5" deep and supports sedums and small herbs.

A cost estimate was made comparing the typical built-up roof system with the intensive and extensive roof systems. It was shown that the $49,871 cost for the intensive roof is only 0.03% of the total $150 million budget, making it clear that the green roof is a very feasible addition to the budget, especially when long term cost reductions in storm water treatment and energy consumption are considered.

Structural Depth

The structural system had to not only meet the strength and serviceability requirements of the new design, but to do so as efficiently as possible. Extensive computer modeling of the lateral and gravity system was used to meet these design criteria, which was verified by even more extensive manual checks.

The existing lateral system consisted of E-W braced frames and N-S moment frames. The E-W frames were simply continued upward with chevron bracing while the N-S frame moment frames were replaced by two braced frames in an attempt at creating a more efficient system. After dozens of design iterations in RAM, a check using STAAD, and multiple hand calculations, a finalized design was made which met the 8.8" drift limit and resisted the controlling load combination $1.2D + 1.6W + 0.5L_{r.}$ Torsion did play a significant role in the building design due to the sizing and geometry of the N-S frame columns. Afterwards, frame foundations were sized based off of the bedrock bearing capacity, resulting in foundations that were 15% to 50% larger than the original designs, which is a reasonable size increase, considering the dramatically increased gravity and wind loads.

The economic analysis of the lateral system was based off the unit weight ratio, which is the ratio of steel weight to square footage. The unit weight ratio of the original and new designs was then compared to obtain a percentage price change. A 6% decrease indicated that the design was successful in reducing relative cost.

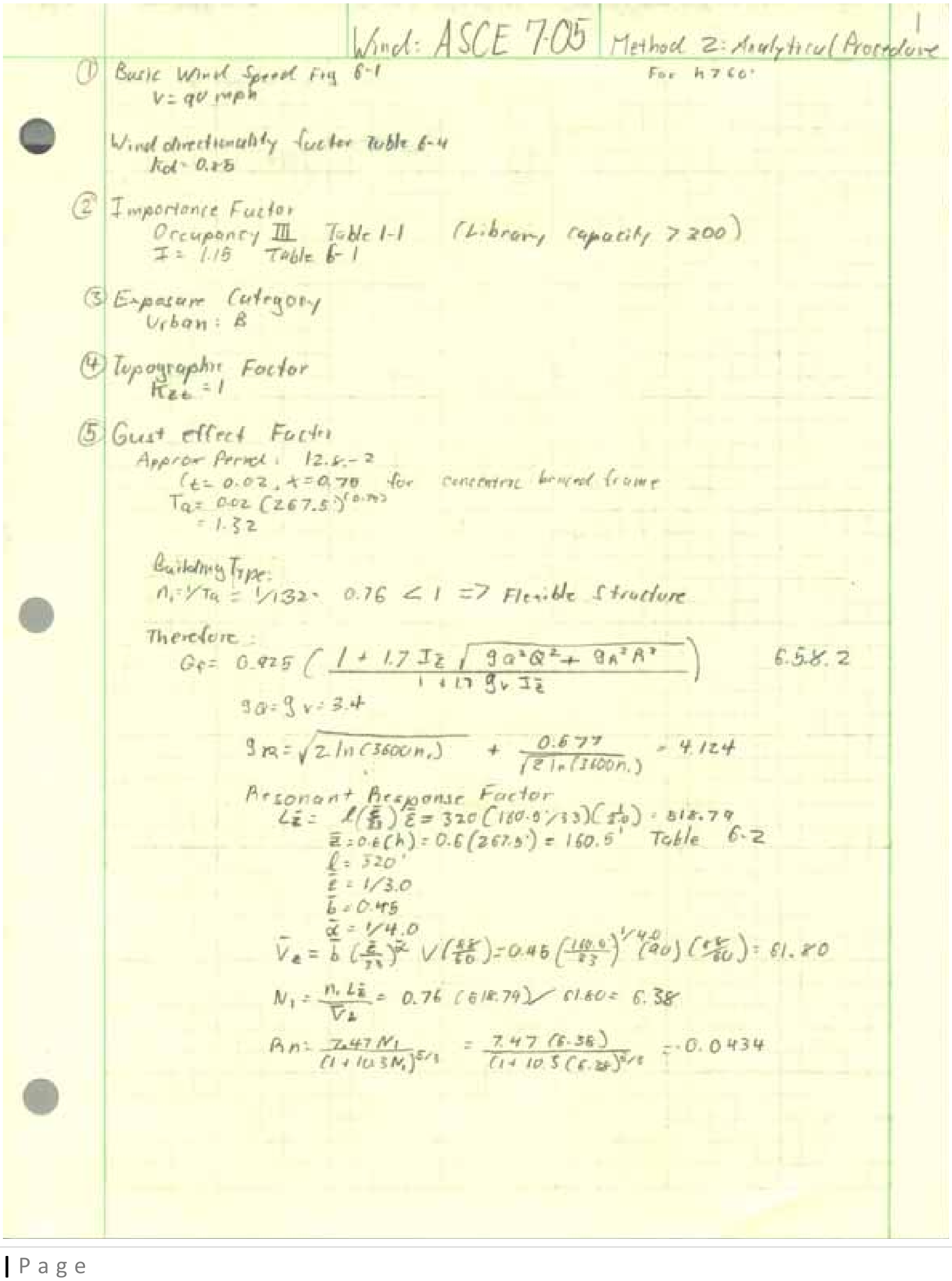
The gravity analysis included in-depth hand checks which were used to compare the original design to the new one, as well as check the accuracy of the modeling program by comparing the original RAM model design to the "existing original design. Another cost analysis following this section helped provided further evidence that the building design was an effective way to increase the program capacity and that the structural design was an efficient, cost saving design.

Lastly, a vibration analysis for sensitive equipment was made following the Steel Design Guide Series 11 criteria. The results of the analysis demonstrated that the floor system is adequate for less sensitive equipment and procedure requirements, which may meet the university's needs; however, a redesign would be required to accommodate more sensitive equipment.

Overall Conclusion

The goals set out in the design impetus were successfully achieved: The final architectural design was an efficient and thoughtful design, which met the programming requirements and provided a safe environment for the building occupants. A compelling case was made supporting the incorporation of green roofs into the building design by comparing the costs of various systems and describing the long term benefits of green roofs. Most importantly, a new structural design was created that not only met the stringent strength and serviceability requirements of the MHSC's laboratories and classroom, but was a cost effective alternative to the original design.

[APPENDIX]



Wind: ASCE 7-05 Method 2: Analytical Procedure 1

For $h > 60'$

① Basic Wind Speed Fig 6-1

$V = 90$ mph

Wind directionality factor Table 6-4

$K_d = 0.85$

② Importance Factor

Occupancy III Table 1-1 (Library, capacity > 200)

$I = 1.15$ Table 6-1

③ Exposure Category

Urban: B

④ Topographic Factor

$K_{zt} = 1$

⑤ Gust effect Factor

Approx Period: 12.8.-2

$C_t = 0.02$, $x = 0.75$ for concentric braced frame

$T_a = 0.02(267.5')^{(0.75)}$

$= 1.32$

Building Type:

$n_1 = 1/T_a = 1/1.32 = 0.76 < 1 \Rightarrow$ Flexible Structure

Therefore:

$$G_f = 0.925\left(\frac{1 + 1.7 I_{\bar z}\sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1 + 1.7 g_v I_{\bar z}}\right) \quad 6.5.8.2$$

$g_Q = g_v = 3.4$

$$g_R = \sqrt{2\ln(3600 n_1)} + \frac{0.577}{\sqrt{2\ln(3600 n_1)}} = 4.124$$

Resonant Response Factor

$L_{\bar z} = \ell\left(\frac{\bar z}{33}\right)^{\bar\epsilon} = 320(160.5'/33)\left(\frac{1}{3.0}\right) = 518.79$

$\bar z = 0.6(h) = 0.6(267.5') = 160.5'$ Table 6-2

$\ell = 320'$

$\bar\epsilon = 1/3.0$

$\bar b = 0.45$

$\bar\alpha = 1/4.0$

$$\bar V_{\bar z} = \bar b\left(\frac{\bar z}{33}\right)^{\bar\alpha} V\left(\frac{88}{60}\right) = 0.45\left(\frac{160.5}{33}\right)^{1/4.0}(90)\left(\frac{88}{60}\right) = 61.80$$

$$N_1 = \frac{n_1 L_{\bar z}}{\bar V_{\bar z}} = 0.76(518.79)/61.80 = 6.38$$

$$R_n = \frac{7.47 N_1}{(1 + 10.3 N_1)^{5/3}} = \frac{7.47(6.38)}{(1 + 10.3(6.38))^{5/3}} = 0.0434$$

2

R_h: $n = 4.6\, n_1 h / \bar{V}_z = 4.6\,(0.76)(267.5) / 61.80 = 15.132$

$$R_h = \frac{1}{n} - \frac{1}{2n^2}\left(1 - e^{-2n}\right)$$

$$= \frac{1}{15.132} - \frac{1}{2(15.132)^2}\left(1 - e^{-2(15.132)}\right) = 0.0639$$

R_B: $n = 4.6\, n_1 B / \bar{V}_z = 4.6(0.76)(392) / 61.80 = 19.912$

for most of building

$$R_B = \frac{1}{19.912} - \frac{1}{2(19.912)^2}\left(1 - e^{-(2)(19.912)}\right) = 0.0495$$

R_L: $n = 15.4\, n_1 L / \bar{V}_z = 15.4\,(0.76)(85') / 61.80 = 16.098$

for most of building

$$R_L = \frac{1}{16.098} - \frac{1}{2(16.098)^2}\left(1 - e^{-2(16.098)}\right) = 0.060$$

$$R = \sqrt{\frac{1}{\beta} R_n R_h R_b (0.53 + 0.47 R_L)}$$

$$= \sqrt{\frac{1}{0.05}(0.0434)(0.0639)(0.0495)(0.53 + 0.47(0.060))}$$

$$= 0.039$$

Background Response

$$Q = \sqrt{\frac{1}{1 + 0.63\left(\frac{B+h}{L_z}\right)^{0.63}}} = \sqrt{\frac{1}{1 + 0.63\left(\frac{352 + 267.5}{518.79}\right)^{0.63}}} = 0.766$$

Intensity of Turbulence

$$I_z = c\left(\frac{33}{z}\right)^{1/6} = \underset{\text{Table 6-2}}{0.30}\left(\frac{33}{160.5}\right)^{1/6} = 0.230$$

$$G_f = 0.925\left(\frac{1 + 1.7(0.230)\sqrt{(3.4)^2(0.766)^2 + (3.4)^2(0.039)^2}}{1 + 1.7(3.4)(0.230)}\right)$$

$$= 0.802$$

⑥ Enclosure Factor : Enclosed

⑦ Internal Pressure → don't care

⑧ External Pressure Coefficient Fig 6-6

Windward wall	$C_p = 0.8$	q_z	$L/B = 85/392 = 0.241$
leeward wall	$C_p = -0.5$	q_h	
Sidewall	$C_p = -0.7$	q_h	
Roof	$C_p = -0.18$		

⑨ Velocity Pressure

$$q_z = 0.00256\, K_z (K_{zt})(K_d)\, V^2 I$$

$$= 0.00256\, K_z (1)(0.85)(90)^2(1.15) = 20.269 K_z$$

K_z : Table 6-3 Exp B case 2

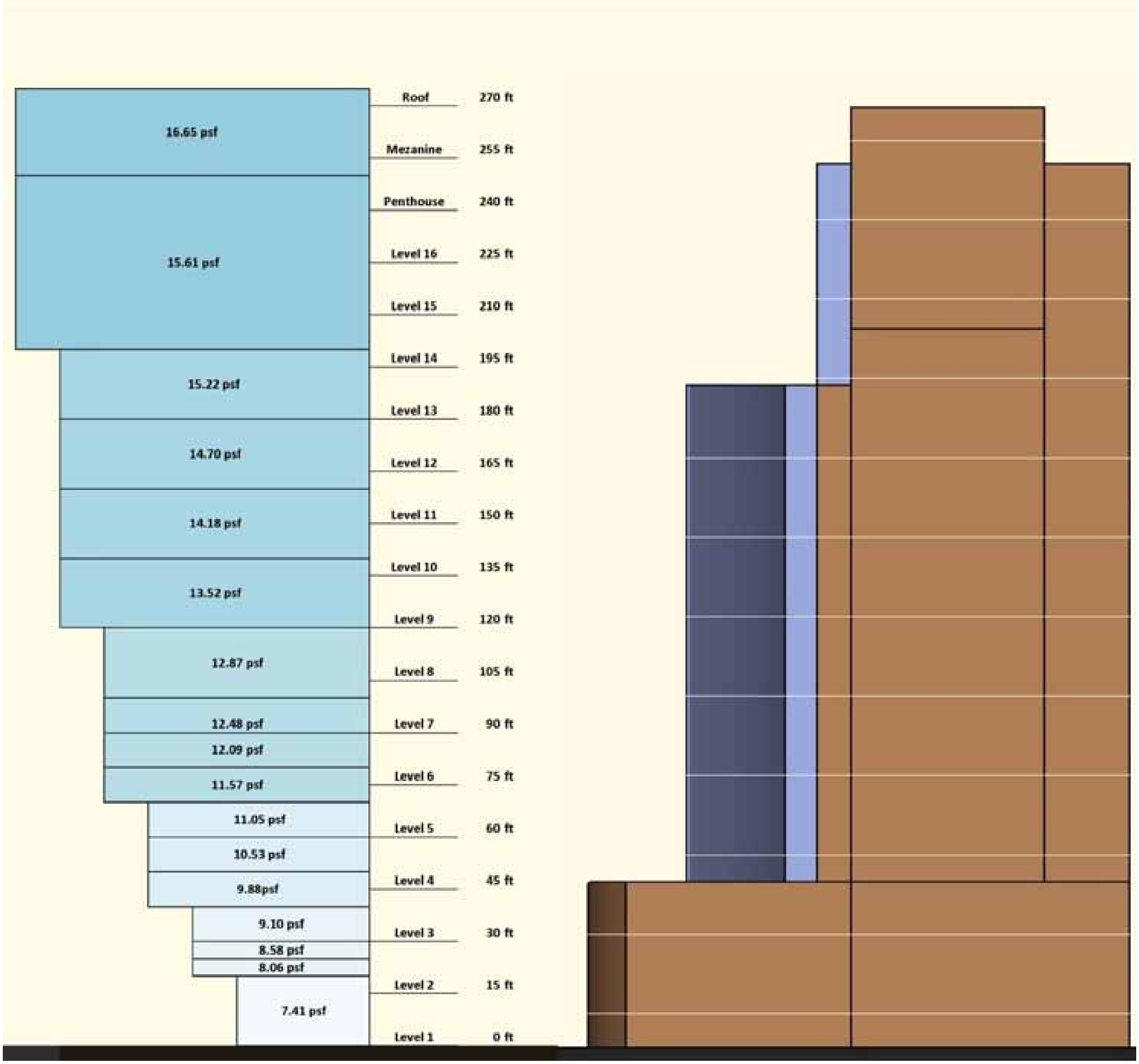
(10) MWFRS for Flexible Buildings

Windward

$$p = q\,G_f C_p \cancel{\pm q_i\,GC_{pi}}$$
$$= q\,(0.802)(0.8)$$
$$= 0.64q$$

Leeward

$$p = q_h G_f C_p \pm q_i\,GC_{pi} \qquad q_i = q_h = (20.269)(1.28) = 25.944$$
$$= 25.944(0.802)(0.5) = 10.40$$



Above: Wind loading diagram

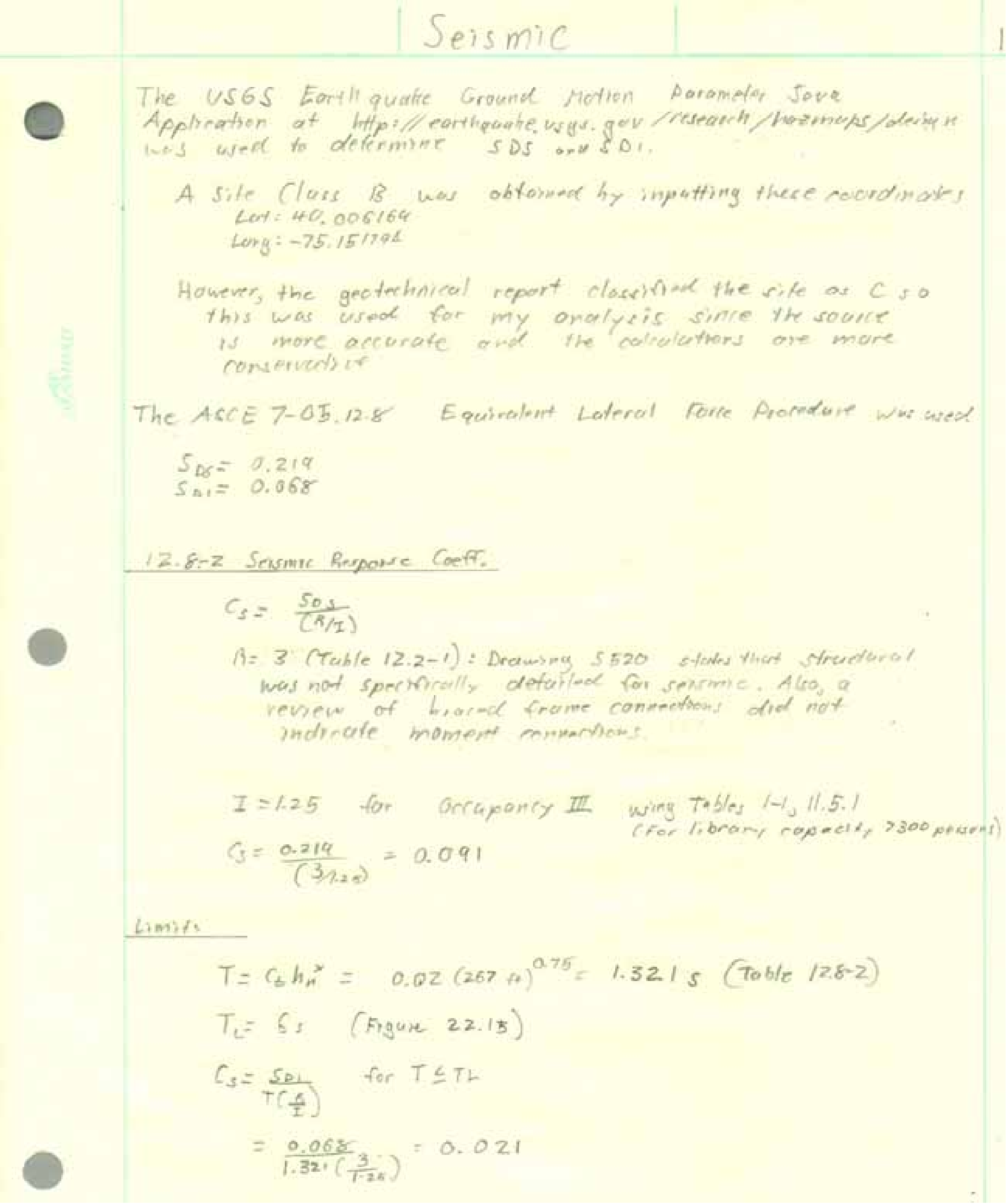
Detailed Calculations - E-W

				Windward	Leeward	Windward kips	Leeward kips	Total Kips
Floor	Height	Elevation	Bldtg Width	PLF vert W	PLF vert L	Bldg Load	Bldg Load	
Level 1	15.33	0	399.8333	54.36	76.27	21.73	30.49	52.23
Level 2	15.33	15.33	399.8333	120.35	159.38	48.12	63.73	111.85
Level 3	15.33	30.66	399.8333	145.03	159.47	57.99	63.76	121.75
Level 4	14.67	45.99	399.8333	158.65	156.00	63.43	62.37	125.81
Level 5	14.67	60.66	352	203.04	187.11	71.47	65.86	137.33
Level 6	14.67	75.33	352	177.48	152.62	62.47	53.72	116.19
Level 7	14.67	90	352	185.97	152.53	65.46	53.69	119.15
Level 8	14.67	104.67	352	196.63	152.53	69.21	53.69	122.91
Level 9	14.67	119.34	352	202.70	152.53	71.35	53.69	125.04
Level 10	14.67	134.01	352	208.60	152.53	73.43	53.69	127.12
Level 11	14.67	148.68	352	215.53	152.53	75.87	53.69	129.56
Level 12	14.67	163.35	352	213.45	152.53	75.13	53.69	128.83
Level 13	14.67	178.02	352	218.30	152.53	76.84	53.69	130.53
Level 14	14.67	192.69	352	223.16	152.53	78.55	53.69	132.24
Level 15	14.67	207.36	352	244.14	152.53	85.94	53.69	139.63
Level 16	14.67	222.03	352	244.14	152.53	85.94	53.69	139.63
Level 17 PentFlr	17.75	236.7	352	269.80	168.57	94.97	59.34	154.31
Level 18 PentMez	12.96	254.45	352	255.58	159.68	89.97	56.21	146.17
Level 19 Roof	0	267.41	352	107.85	67.38	37.96	23.72	61.68
Total						1305.84	1016.12	2321.96

Detailed Calculations - N-S

				Windward	Leeward	Windward kips	Leeward kips	Total Kips
Floor	Height	Elevation	Bldtg Width	PLF vert W	PLF vert L	Bldg Load	Bldg Load	
Level 1	15.33	0	160.1667	54.36	76.27	8.71	12.22	20.92
Level 2	15.33	15.33	160.1667	120.35	159.38	19.28	25.53	44.80
Level 3	15.33	30.66	160.1667	145.03	159.47	23.23	25.54	48.77
Level 4	14.67	45.99	160.1667	158.65	156.00	25.41	24.99	50.40
Level 5	14.67	60.66	128.6667	203.04	187.11	26.12	24.08	50.20
Level 6	14.67	75.33	128.6667	177.48	152.62	22.84	19.64	42.47
Level 7	14.67	90	128.6667	185.97	152.53	23.93	19.63	43.55
Level 8	14.67	104.67	128.6667	196.63	152.53	25.30	19.63	44.93
Level 9	14.67	119.34	128.6667	202.70	152.53	26.08	19.63	45.71
Level 10	14.67	134.01	128.6667	208.60	152.53	26.84	19.63	46.47
Level 11	14.67	148.68	128.6667	215.53	152.53	27.73	19.63	47.36
Level 12	14.67	163.35	77.9167	213.45	152.53	16.63	11.88	28.52
Level 13	14.67	178.02	77.9167	218.30	152.53	17.01	11.88	28.89
Level 14	14.67	192.69	77.9167	223.16	152.53	17.39	11.88	29.27
Level 15	14.67	207.36	77.9167	244.14	152.53	19.02	11.88	30.91
Level 16	14.67	222.03	77.9167	244.14	152.53	19.02	11.88	30.91
Level 17 PentFlr	17.75	236.7	77.9167	269.80	168.57	21.02	13.13	34.16
Level 18 PentMez	12.96	254.45	77.9167	255.58	159.68	19.91	12.44	32.36
Level 19 Roof	0	267.41	77.9167	107.85	67.38	8.40	5.25	13.65
Total						393.87	320.36	714.24

Above: Detailed calculations for story shear



Seismic

The USGS Earthquake Ground Motion Parameter Java Application at http://earthquake.usgs.gov/research/hazmaps/design was used to determine SDS and SD1.

A Site Class B was obtained by inputting these coordinates
Lat: 40.006164
Long: -75.151796

However, the geotechnical report classified the site as C so this was used for my analysis since the source is more accurate and the calculations are more conservative

The ASCE 7-05 12.8 Equivalent Lateral Force Procedure was used

$S_{DS} = 0.219$
$S_{D1} = 0.068$

12.8-2 Seismic Response Coeff.

$$C_s = \frac{S_{DS}}{(R/I)}$$

R = 3 (Table 12.2-1): Drawing S520 states that structural was not specifically detailed for seismic. Also, a review of braced frame connections did not indicate moment connections.

I = 1.25 for Occupancy III using Tables 1-1, 11.5.1
(For library capacity >300 persons)

$$C_s = \frac{0.219}{(3/1.25)} = 0.091$$

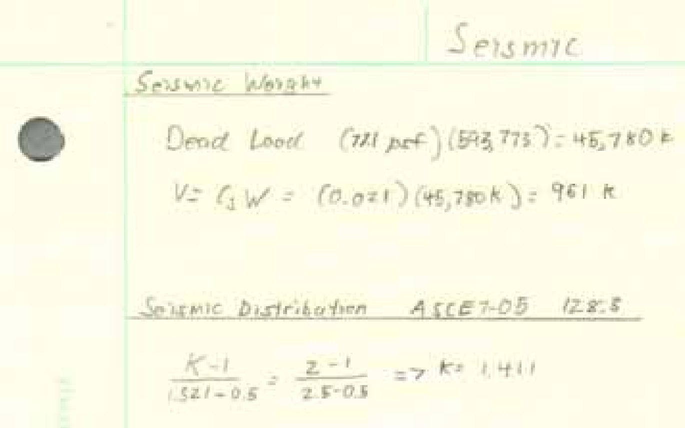
Limits

$$T = C_t h_n^x = 0.02\,(267\text{ ft})^{0.75} = 1.321\text{ s} \quad \text{(Table 12.8-2)}$$

$$T_L = 6\text{ s} \quad \text{(Figure 22.15)}$$

$$C_s = \frac{S_{D1}}{T\left(\frac{R}{I}\right)} \quad \text{for } T \le T_L$$

$$= \frac{0.068}{1.321\left(\frac{3}{1.25}\right)} = 0.021$$



Seismic

Seismic Weight

Dead Load (77.1 psf)(593,773) = 45,780 k

$V = C_s W = (0.021)(45,780k) = 961 k$

Seismic Distribution ASCE7-05 12.8.5

$\frac{K-1}{1.521-0.5} = \frac{2-1}{2.5-0.5} \Rightarrow K = 1.411$

Vertical Seismic Distribution										
Level	Area (SF)	w_x (K)=	A * DL(77.1psf) * C_s(.021)	H_x	K	$w_x * H_x^K$	Cvx =	$\frac{(w_x * H_x^K)}{(\sum wx * Hx^K)}$	F_x (K)=	C_{vx} * V(955k)
1st	57000		92.29	0.00	1.411	0.00		0.00		0.00
2nd	51800		83.87	15.33	1.411	3948.24		0.00		4.56
3rd	45000		72.86	30.66	1.411	9120.94		0.01		10.53
4th	57000		92.29	45.99	1.411	20472.30		0.02		23.62
5th	28000		45.33	60.66	1.411	14863.02		0.02		17.15
6th	28000		45.33	75.33	1.411	20175.93		0.02		23.28
7th	28000		45.33	90.00	1.411	25933.90		0.03		29.93
8th	28000		45.33	104.67	1.411	32092.30		0.04		37.03
9th	28000		45.33	119.34	1.411	38616.85		0.05		44.56
10th	28000		45.33	134.01	1.411	45480.19		0.06		52.48
11th	28000		45.33	148.68	1.411	52659.90		0.06		60.77
12th	25197		40.80	163.35	1.411	54116.99		0.07		62.45
13th	25197		40.80	178.02	1.411	61098.98		0.07		70.51
14th	25197		40.80	192.69	1.411	68321.72		0.08		78.84
15th	25197		40.80	207.36	1.411	75774.22		0.09		87.44
16th	25197		40.80	222.03	1.411	83446.74		0.10		96.30
Pent.	25197		40.80	236.70	1.411	91330.59		0.11		105.39
Mez.	7500		12.14	254.45	1.411	30105.09		0.04		34.74
Roof	22685		36.73	267.41	1.411	97669.76		0.12		124.00
Totals:	588167		952.30			825227.66				963.59
Note:	Cs=0.02									