The Edward L Kelly Leadership Center
Prince William County School Administration Center

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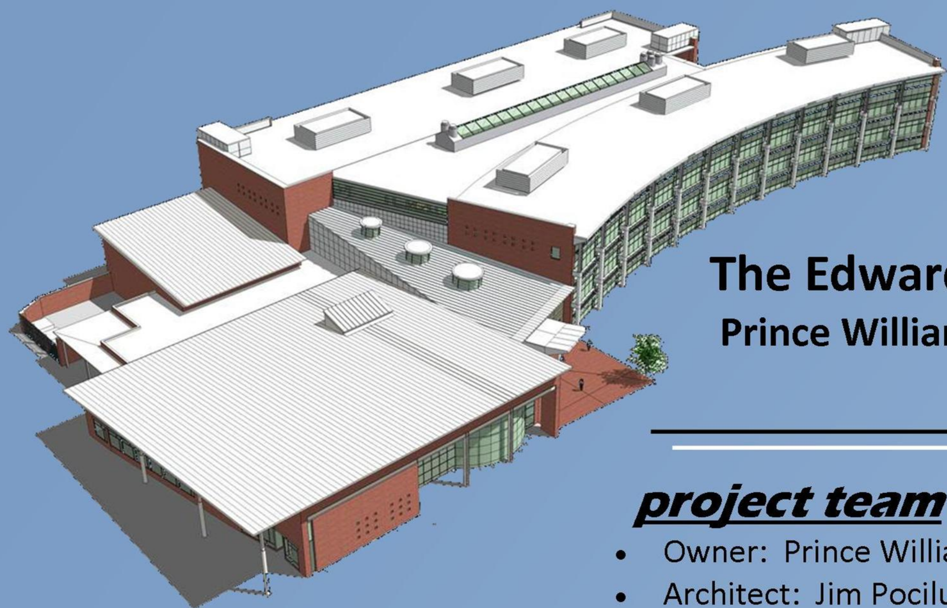
Thesis Consultant - Dr. Hanagan

Final Report
April 9, 2008

MOSELEYARCHITECTS
A PROFESSIONAL CORPORATION



The Edward L Kelly Leadership Center

Prince William County Schools Admin Building

Manassas, VA

project team

- Owner: Prince William County Schools
- Architect: Jim Pociluyko, Moseley Architects
- Structural Engineer: Jeff O'beirne, Moseley Architects
- Mechanical Engineer: Jim Miller, Moseley Architects
- Electrical Engineer: Russell Roundy, Moseley Architects
- Plumbing Engineer: Jeffry Mortensen, Moseley Architects
- Civil Engineer: Ross, France, Ratliff, Ltd.

general info

- 145,000 square feet
- School Board Room, Office space for over 500 employees, Training Rooms, Food Court
- 3 Stories above grade
- Height: 46' (56' to highest point)
- Construction Dates: Spring 2007 — Fall 2008
- Delivery Method: Design-Bid-Build

architecture

- 2 3-story sections, 1 1-story section
- Various building heights and roof types
- Curved, glass curtain wall
- Abundant Daylighting through curtain wall, full glass entry, ample windows, sunroofs

electrical / lighting

- Primary: 480/277 V , 3Φ, 4-wire
- Secondary: 280Y/120V, 3Φ, 4-wire
- 200 kW emergency generator
- Offices/Meeting Rooms: Indirect lighting and Daylighting

structural

- Steel Moment frame construction
- Steel Stud wall framing
- Steel Joist floor framing
- 4.5" non- composite concrete
- 4" Concrete slab on grade
- Spread column concrete footings (typically 7'-0"x7-0" – 10'-6"x10'-6")
- 2'-0" typical wall strip footing

mechanical

- Waterside hot water system, high-efficiency, condensing gas-fired boilers
- Chilled waterside system – dual-circuit, air-cooled chillers
- Variable air volume AHU for each floor
- Smoke control system in atrium

Ryan Pletz

Structural

Portfolio:
http://www.engr.psu.edu/ae/thesis/portfolios/2008/rsp143/

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Acknowledgments

I would like to gratefully thank my thesis advisor, Dr. Hanagan, and all the professors in the Architectural Engineering department, especially those in the structural option for their help throughout this research project. My professors have had an elemental role in my success throughout the past five years and I am very grateful to have such wonderful teachers and mentors.

Thank you to The Pennsylvania State University for such a fine establashment and the providing the tremendous program of Architectural Engineering.

I would particularly like to thank Moseley Architects for their sponsorship of this thesis project and for providing me with construction documents and all the resources I could ask for. I would especially thank Steve Jones, Paul Gagnon, Jeff O'Beirne, and John Swecker for their unreserved willingness to answer any questions I have had during this process and for taking the time with me to visit the building during the construction process. I was able to learn a great deal about the building from visiting the site.

Thank you to the Prince William County Schools for allowing me to study their building for my research project.

Thank you to the mentors and outside consultants for their willingness to support myself and the other researchers throughout the entire year.

Thank you to all of the students in the 2008 graduating class for their help throughout this year and the past 5 years. This report would not have been possible without the eagerness of everyone to help no matter the day of the week or the hour of the day.

Finally, I would like to express my gratitude to my family and friends for helping me to maintain a level head throughout the year when times were at the toughest. The inspiration and encouragement they have provided helped me get through the past year and my college career.

Executive Summary

The intent of the report is to investigate the proposal presented for the Edward L Kelly Leadership Center. The proposal includes an investigation into a re-designed structural system, a study into the possibility of architectural changes to the building, and the construction process impacts that arise from the changes.

The initial structural system of the building consists of non-composite steel beam and open web steel joists with non-composite steel deck and concrete floor slab. The proposal investigated the possibility of altering the gravity system to replace the open web steel joists with composite steel beams and composite steel deck and concrete floor slab. Also, multiple lateral system changes were examined which included braced steel frames, concrete or masonry shear walls, and the existing moment frame system. Wind and seismic forces were analyzed due to the architectural changes to the building. Foundation impacts were also looked at to test their adequacy. The goal of the new system was to increase the economy of the structural system. The composite system was found to decrease the floor system depth as well as the weight of the building and ultimately was most economical. The lateral system was, in the end, kept as the original moment frame system in the existing building. However, the number of frames was reduced from eleven to six in the north-south direction and from three frames to one frame in the east-west direction. The foundation of the building was increased slightly to accommodate the new architecture

For an architectural study, the proposal was to investigate the need to add additional stories to a portion of the building as a purely academic study. Two floors were added to one of the wings of the building adding an additional 36,000+ square feet for future expansion. In addition, the changes to the architecture of this wing also impacted the architectural experience in another part of the building. These impacts were studied and a solution was proposed to incorporate a green roof over a portion of the building. The new system provides potentially needed extra square footage for future expansion as well as an improved aesthetic appeal to the building.

For a construction management study, the cost and scheduling issues that resulted from the structural and architectural changes were analyzed. A takeoff of the changes that resulted from the new systems was compared to the existing system. The cost of the system new system was found to be $1.32 million compared to the original system which cost $0.779 million which is logical due to the additional two stories added. However, when analyzed as a per square foot or per floor basis, the construction cost is approximately the same cost at $260,554 per floor for the existing building and $236,063 per floor for the re-designed building. The new system provided a faster schedule as well as a cost savings. The green roof added an additional cost of $660,000-$750,000 to the building.

The overall investigation through this research was determined to be successful.

Building Information

Introduction

The Edward L Kelly Leadership center is an administration building for the Prince William County Public Schools. The building is an administrative building for the Prince William County Public Schools located in the northern Virginian city of Manassas (See Figures 5a and 5b for site location). Currently housed in separate facilities, the architectural goal of the building is to combine the several School Administration functions into one central facility. The facility is daylight-filled with a 3-story atrium with skylights and a clerestory entrance. The building program contains flexible office space for 500 employees as well as meeting and training rooms for the district.

The building is composed of essentially three distinct sections. The gross square footage of the building is approximately 150,000 square feet. There is a one-story section on the west of the building plan. It is here that the main School Board meeting rooms, meeting rooms, exercise, kitchen, and "public" spaces are located. This section of the building is approximately 25,000 square feet. On the northern portion of the building is a three-story, rectangular, 17,000 square foot section of the building where offices for district employees are located. The southern share consists of another three-story building that is radial in geometry and has a footprint of approximately 19000 square feet. An atrium and walkways separate the two three-story buildings by approximately 36 feet at its midpoint and represent another 20,000 square feet of the building. The two three-story buildings are approximately 60 feet in width and the rectangular and radial buildings are 265 feet and 295 feet, respectively.

The structural system is steel construction. Steel beams and girders are supported by steel W- or HSS-shape columns. Steel joists fill in the bays. The construction is non-composite concrete with steel decking. The lateral system consists of moment frames. Nearly every column-to-girder connection is fixed.

Overview/Architecture

The building is located in Manassas, Virginia and will serve as the Prince William County School Administrative Building. The program contains flexible office space for 500 County School Employees, as well as a School Board meeting room and other meeting/training rooms for school personnel. The size of the building is just less than 150,000 square feet. The design includes several parts including a one-story wing and two three-story wings. The building has a very open, flexible, and light filled atmosphere through the use of several curtain walls, a three-story atrium, and multiple skylights. The primary materials are steel, glass, and masonry.

Construction

The project delivery method for this building is Design-Bid-Build. The building was designed by Moseley Architects and was put out for public bid in September, 2006. Bids were due late October, 2006. The contract was awarded to V.F. Pavone via Lump Sum Contract. Construction began in late Winter,

2007. The contractual substantial completion date for the project is set for October 1, 2008. The cost of the project is $32,639,800.

Structural

The main structural system in the building is steel space moment frame. Nearly all connections are moment-resisting connections. All columns in the structural system are steel. In the one-story building, some typical interior columns include W12x79 and W10x68. Exterior columns are often HSS shapes. Typical shapes include HSS8x6x1/4 in the one-story wing and W14x68 and W14x82 for the interior and HSS12.75x0.375 for the exterior in the three-story wing. Built up W21 shapes with HSS2½ (TOP) are typically used for beams while W24 are used for girders. The size of the bays are generally 24' wide and span 30'. Steel joists are used to span inside the bays. 28K8 joists are the most common joist in the framing. Typical spacing is approximately 4' on center. The one-story "floor" (mezzanine) joists are 26K9 spanning 30' in one part of this platform and 24K3/26K4 spanning 16'/19' respectively. Roof joists in the one-story portion are typically 28K10. Foundations consist of spread footings and strip wall footings at (-2'-0") from grade on soil with a bearing capacity of 3000 psf. Typical column spread footings range in size from 4'-0"x4'-0" to 11'-0"x11'-0". The strip wall footings are typically 2'-0" wide and 1'-0" deep. The slab-on-grade is 4" deep.

Mechanical

The air distribution system utilizes variable air volume controlled locally or remotely by a direct digital system. The Heating, Ventilation, Air Conditioning system utilizes a waterside/airside system which uses chilled water and hot water for cooling and heating, respectively. The hot water system uses high-efficiency, condensing-type, gas-fired boilers with centrifugal pumps. The hot water serves the preheat coil at the AHU and reheat coils at the VAV box terminals. The chilled water system uses two dual-circuit, air-cooled chillers with centrifugal pumps. The chilled water is provided to the cooling coils in the AHU. The building has six Roof Top Units (RTU); three dedicated to each three-story wing. These units supply 5600-6800 CFM each with 2800-4150 CFM outdoor air and 2800-4150 CFM return air.. Four AHU serve the atrium (two), the boardroom (one-dedicated), and the remaining meeting rooms (one). These AHU make operate with return/exhaust heat recovery systems.

Electrical

The primary electrical service is provided through 480/277V, 3Φ, 4-wire underground service. The switchboard has 4000 amp bussing with 3000A main circuit breaker. The voltage is dropped to 277V for lighting and dropped to 120V for receptacles. There is a 200kW emergency electrical generator attached to the system.

Lighting

Much of the lighting is provided through ample daylighting. The fully glass entrance and atrium, as well as vast curtain walls and skylighting provides a great deal of lighting for the building. In addition, the offices are fitted with indirect linear fluorescent lighting consisting of (2) 28 watt 32T5 pendant lamps per luminaire. The conference rooms, meeting spaces, and premium areas are typically fitted with direct lighting. A typical premium space will have 9" 26 watt compact fluorescent recessed round downlights with open reflectors.

Life Safety

The atrium is equipped with a smoke control system for life safety purposes directed by code requirements. In the event where smoke control becomes an issue, air vacates through exhaust grilles located on either side of the atrium. There are six smoke purge fans that are capable of exhausting 174720 CFM (29120 CFM per fan). Make-up air is provided through automatic door openers at each entrance. These systems are all controlled the fire control panel and power provided by the emergency generator.

Building Traffic

The building is very flexible through its design and has many open, flexible areas. There is a very open atrium which serves as the main entryway and contains an open stairway and two elevator shafts for access to upper levels of the atrium. The building spaces are accessed through the atrium corridors.

Structural System

Floor and Roof Framing

Three-story wing:
W21 shapes are typically used for beamswhile W24 are used for girders. The sizes of the bays are generally 24' wide and span approximately 30'. Steel joists are used to span inside the bays. 28K8 joists are the most common joist in the framing. Typical spacing is approximately 4' on center. Joists also frame the roof, where, to account for the heavy and asymmetric loads of mechanical equipment, KCS joists are commonly found. Roof beams are typically W18x35 and girders W21x44.

One-story wing:
This part of the building contains an elevated area that serves as an equipment platform. It covers a good portion of the footprint of this section. The "floor joists" are 26K9 spanning 30' in one part of this platform and 24K3/26K4 spanning 16'/19' respectively. Roof joists in the one-story portion are typically slightly larger than the 3-story building (28K10) since they span a much longer distance of around 47'. The structural plans show an area where the joists become increasingly closer to each other. This is due to the higher roof causing snow to drift onto the lower roof in addition to windward drift. A few special joists (KSP) are used in certain areas of the one-story roof framing to account for unique loading. This is generally where there are folding partitions, causing heavy concentrated loading at points, in meeting rooms such as the School Board Meeting room.

Figures 3 and 4 following this summary are representative of the floor and roof framing for the 3-story wing (Figure 3) and the one-story wing (roof framing, Figure 4)

Lateral System

The lateral forces, such as wind and seismic forces, in the building are resisted entirely through moment frames. The engineer chose to implement a moment frame to resist these horizontal forces. The particular frame is a space moment frame, meaning that all of the steel frames are used in the moment frame system. Figures 1 and 2 below show typical details of moment connections used throughout the building. Two distinct types of fixed connections are used. The first (Figure 1) is a fixed connection of the girder to the column flange. This connection is made through welds of the girder flange to the column flange. A shear plate connects the girder web to the flange. The second (Figure 2) is a fixed connection of the girder to the column web. This connection is made with a plate welded to the column web and bolted to the girder flange. A shear plate connects the web of the column and girder.

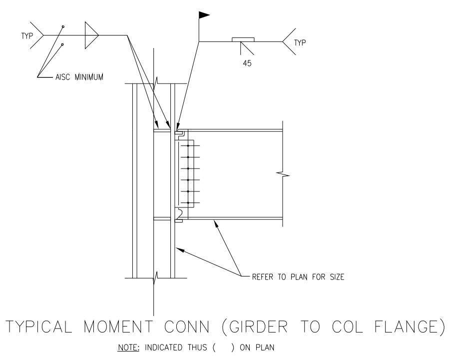


Figure 1. Moment Connection – Girder to Column Flange

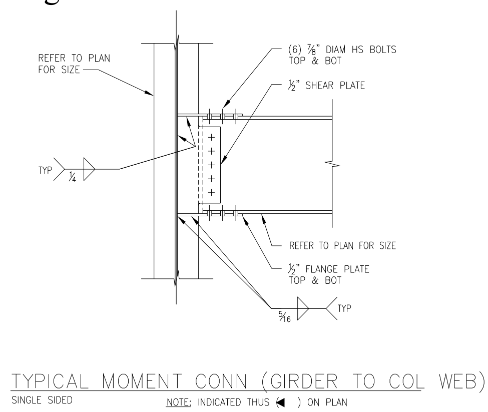


Figure 2. Moment Connection – Girder to Column Web

Foundations

A shallow foundation type is used for this building. Foundations consist of spread footings and strip wall footings. The geotechnical engineer for the project indicated that the allowable bearing capacity of the soil is 3000 PSF. The top of the footings are set at (-2'-0") from grade. Reinforcement for spread footings range from (4)#5 BOT bars for the 3'-0"x3'-0" footings to (11)#7 TOP & BOT for the 11'-0"x11'-0" footings. Exterior column spread footings are typically 4'-0"x4'-0" to 6'-0"x6'-0" in the one-story portion and 7'-0"x7'-0" in the three-story portion. Interior column footings in the one-story portion are typically 6'-0"x6'-0" to 8'-0"x8'-0". The three-story interior column footings are 9'-0"x9'-0" to 11'-0"x11'-0". The strip wall footings are typically 2'-0" wide and 1'-0" thick. Reinforcement for strip footings are (3) continuous #5 bars. The strength of the concrete used for foundations is 3000 psi. The concrete strength for the 4" slab on grade is 3500 psi and contains 6x6-W1.4xW1.4 WWF at mid-depth.

Columns

All columns in the structural system are steel. In the one-story building, some typical interior columns include W12x79 and W10x68. Exterior columns are often rectangular HSS shapes. Typical shapes include HSS8x6x1/4 in the one-story building. In the three-story building, columns are, again, typically W-shapes for the interior and HSS shapes for the exterior. Typical shapes include W14x68 and W14x82 for the interior and circular HSS12.75x0.375 for the exterior.

The following figures represent the typical structural plans of the building.

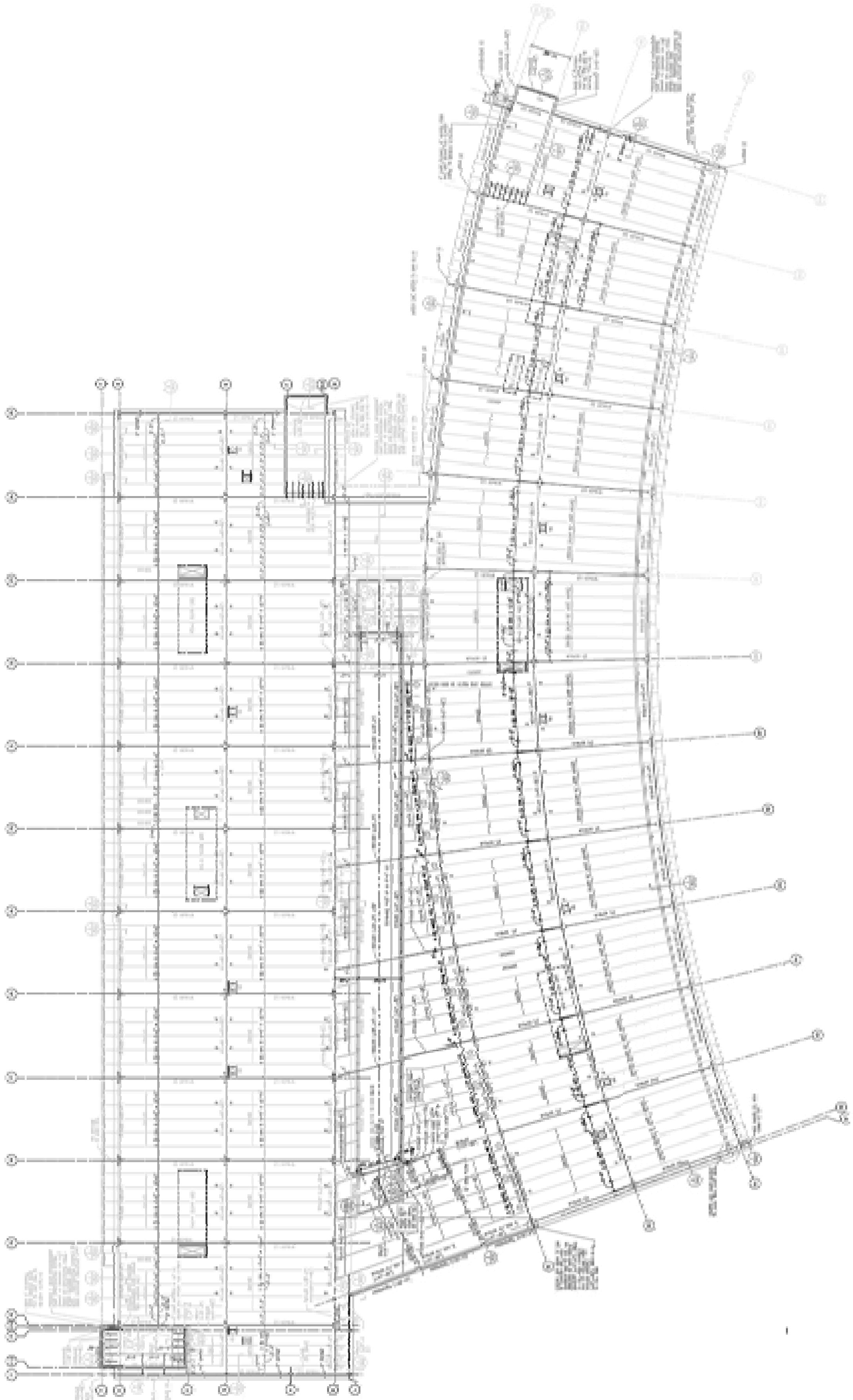


Figure 3. Structural Roof Plan (3-story wings only)

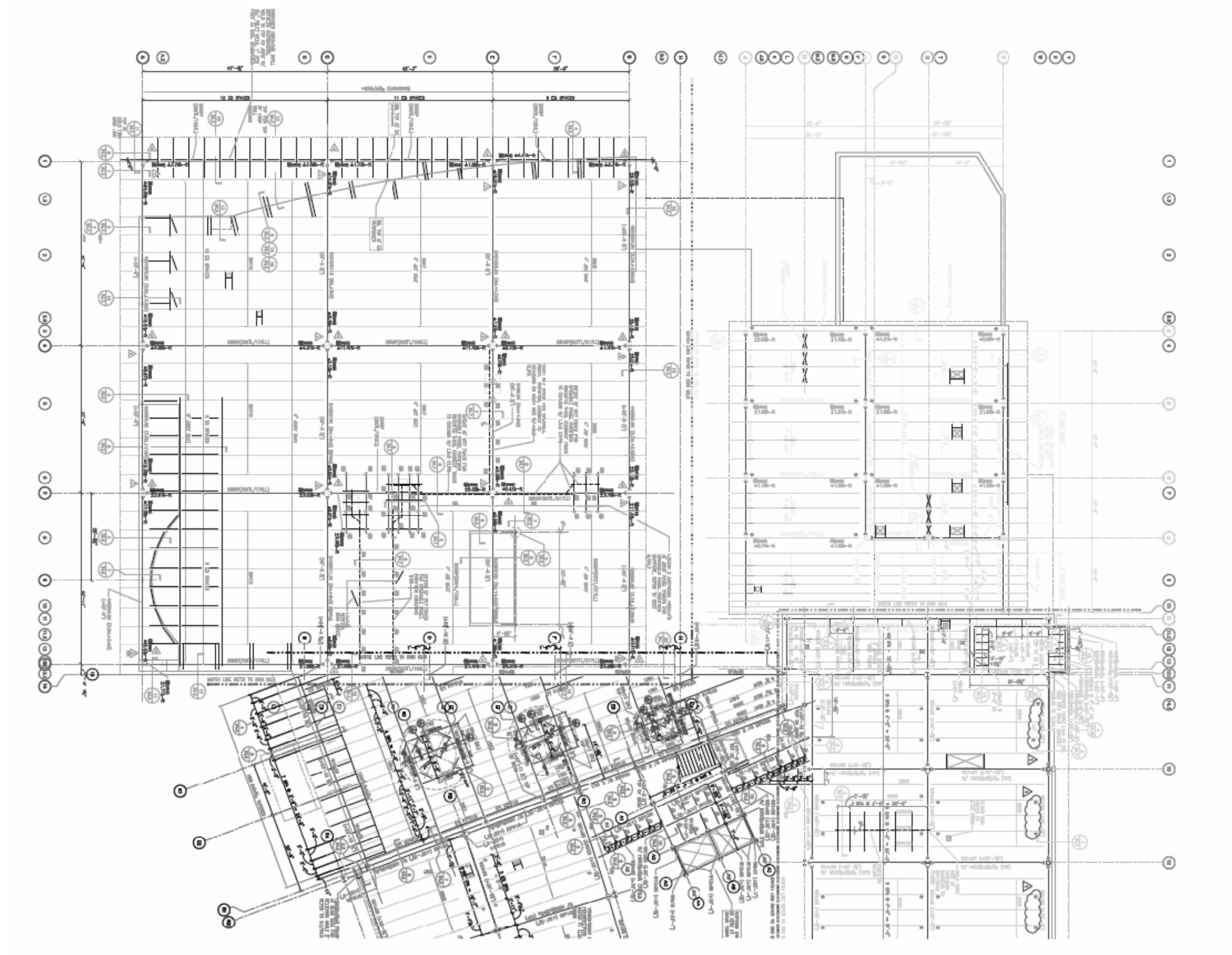


Figure 4. Roof Plan of One-story wing

Codes and Loading

The Virginia Uniform Statewide Building Code (VUSBC), 2000 edition was used for the design of the Edward L Kelly Leadership Center. This code absorbs much of its code from the International Building Code (IBC). IBC2000 will be used when referencing the original design of this building. In addition to IBC, the following codes and specifications were also implemented into the design:

ASCE 7-98, Minimum Design Loads for Buildings and Other Structures
ACI 530-99, Building Code Requirements for Masonry Structures With Commentary
AISC Specification for Structural Steel Buildings, Allowable Stress Design and Plastic Design
AISC Code of Standard Practice for Steel Buildings and Bridges
Steel Deck Institute Design Manual for Composite Desks, Form Decks, and Roof Decks
AISI Specification for the Design of Cold Formed Steel Structural Members

Live Loads	IBC 2006	Snow Load
Meeting Rooms	50 + 20 PSF	
Office Space	50 + 20 PSF	
1st Floor Corridors	100 PSF	
Corridors above 1st Floor	80 PSF	
Stairwell	100 PSF	
Mechanical Rooms	150 PSF	
Storage	125 PSF	
Flat Roof		21 PSF
Sloped Roof		21 PSF

Floor - Superimposed Dead Loads	
Mechanical	4 PSF
Electrical / Lighting	3 PSF
Sprinklers	3 PSF
Drop Ceiling	5 PSF
Total	15 PSF

Roof - Superimposed Dead Loads	
Roofing / Insulation	5 PSF
Mechanical	4 PSF
Electrical / Lighting	3 PSF
Sprinklers	3 PSF
Drop Ceiling	5 PSF
Total	20 PSF

Proposal

Problem Statement

Based on the analyses performed thus far on the Edward L Kelly Leadership center, the structural system is satisfactory in its ability to resist the required loading conditions of gravity, wind, and seismic. However, it is my hypothesis that there is a great amount of redundancy that creates a less efficient structural system. Specifically, there seems to be an excessive use of fixed connections of the steel beams. On the architectural front, discussions with the architects of the project allude to the necessity for future expansion of the building to accommodate the growing school system. These issues will be investigated with anticipation of creating a more efficient design.

Proposed Solution

While the current design utilizes a non-composite steel framing system, an alternative framing system will be investigated. The alternative will be remain as steel framing, but will consist of concrete on composite steel deck. In addition, rather than steel joists as fillers between main beams, composite steel beams will be investigated to fill the bays. While steel joists offer advantages such as low weight and open webs to accommodate mechanical systems, steel beams will more than likely offer a more shallow system, combat vibration issues, and can be spaced at greater distances than allowed by joists.

While the current lateral system is composed purely of moment frames (see structural floor plans in previous Figures 4 and 5), a new lateral system will be investigated. The current architectural program consists of a very open floor plan. The exterior walls consist either of glass curtain walls or storefront windows. This is the biggest obstacle when considering alterations to the lateral system as it limits the areas where steel bracing or shear walls can be used. The existing moment framing will also be investigated, but in a much more limited sense compared to the current system.

The most up-to-date codes, such as ASCE7-05, IBC 2006, and all applicable codes will be utilized in the structural re-design process. Existing RAM Structural System models will be used and adapted, as needed, for the new framing system. Changes to the lateral system will be investigated through use of this model with braced frames or a reduction in fixed connections. Hand calculations will supplement computer output and used to verify results.

Breadth Topics

Breadth Study 1: Architecture

The architect has indicated that an expansion to the building may be necessary to make room for future employees. Therefore, to accommodate for future expansion, an architectural breadth study will be conducted. A look at multiple configurations will be considered. Based on the site plan, expanding the building is possible horizontally. In addition, a vertical expansion is possible with the addition of floors to the main three-story wings.

Breadth Study 2: Construction

A second breadth study will be conducted on the construction process. Because the architectural plans will be expanded and the structural system will be revised, scheduling will become an issue. A cost analysis will be conducted on the new floor system and compared to the previous system. An in-depth scheduling investigation will be conducted and solutions will be compiled to fully compare the existing building with the new design. RSMeans Building Construction Data will be used to generate costs per square foot estimates. Scheduling times will also be estimated using appropriate RSMeans reference texts.

Solution Method

The new steel structural system will be analyzed based upon the specifications of the AISC Steel Construction Manual, 13th edition. Gravity and lateral loading will be determined with ASCE7-05. Using the computer program, RAM Structural System, a model of the building will be input. In addition to the use of computer models, hand calculations will supplement overall results and be used to confirm the output. The appropriate changes to the current building, such as the change to composite decking and beams and the elimination of steel joists within the bays will be made within the model. The overall changes in building weight with regard to its impact on foundations will be investigated and, if problematic, changes to the foundation will be considered. Investigations into several changes to the lateral elements will be conducted within RAM. A reduction in the number of moment connections will be made initially. Later, the addition of braced frames and shear walls will be analyzed. Wind and seismic loading will be re-analyzed with the new architectural system. The new loading will be distributed to the newly proposed lateral system. Lastly, scheduling and costs will be investigated using up to date versions of RSMeans Building Construction Data. All changes will be compared to the original design.

Architectural Breadth

Current Architecture

The current building contains an area of approximately 150,000 square feet. This area is divided between several different areas, or “wings,” that make up the whole building. The one-story wing contains 23,700 square feet. This part of the building contains most of the “public” spaces. It contains spaces such as the school board meeting room, large group meeting rooms, the kitchen and serving area, and the fitness room. Connected to this wing are two additional wings. These wings act as the “private” space of the building. It contains both open and private office space, file storage, as well as smaller conference and meeting rooms. Each of these wings is three-stories in height and both are adjoined with a common atrium. The atrium is fully open and light-filled with a large skylight and windows that create a transparency between the two three-story wings. Both of these wings serve a common purpose of a combination of open workspace and private office. The first of these wings, the southernmost wing, is radial in form. This wing will be referenced as the “radial wing.” The northernmost has rectangular form and will be referenced as the “rectangular wing.” In the radial wing, each floor contains 18,784 square feet of area (56,352 SF total). This is broken down into 7,000 square feet for open office (60 workstations), 1,857 square feet for private office (11 offices), and about 10,000 square feet for workrooms, meeting rooms, conference rooms, etc. The rectangular wing has 18,885 square feet allocated to each floor with 6,500 square feet designated as open office (58 workstations), 2,400 square feet as private office (18 offices), and approximately 10,000 square feet as workrooms, meeting rooms, and conference rooms. Figure 5 below shows the different areas of the building under consideration.

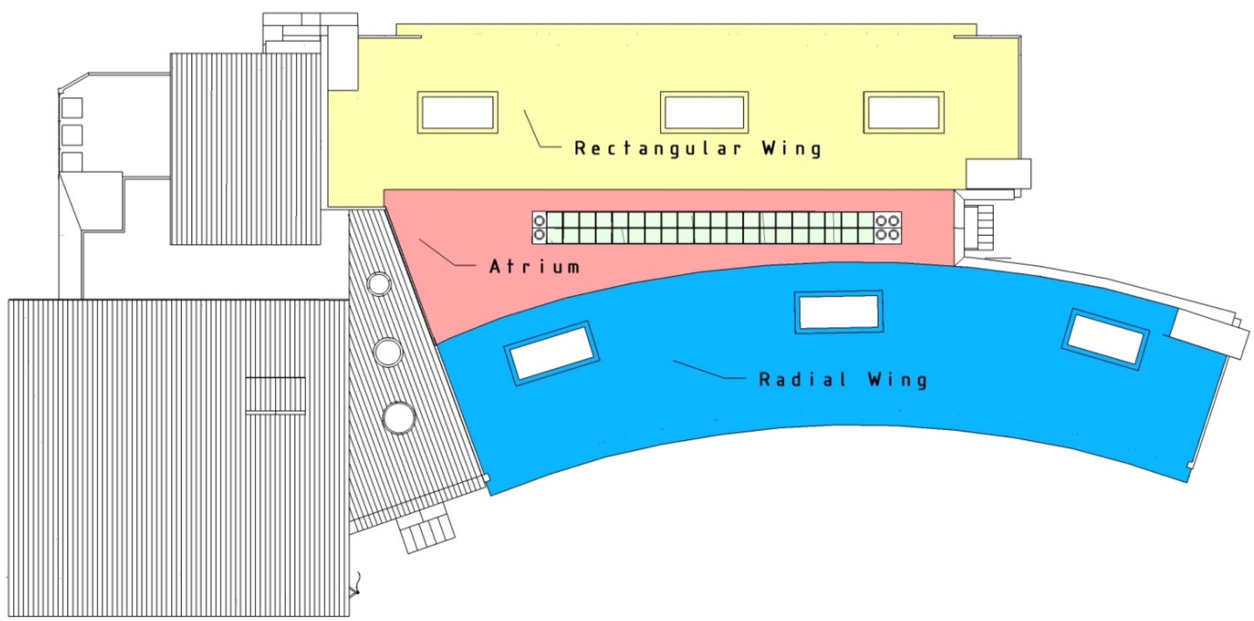


Figure 5. The three areas of the building under consideration for alterations.

The following two figures show the location of the building (Figure 5a) and the site plan (Figure 5b).

Figure 5a. Site Location, Manassas, Virginia

Figure 5b. Site Plan with Surrounding Context, Dumfries Road, Manassas, Virginia

Architectural Problem

Schematic drawings from the architectural firm indicate that future expansion on the building site was considered in the design of the building. School systems are always growing and constantly changing in size. The school district currently has an enrollment of 72,654 pupils. According to the school district, the enrollment has grown 2.5% to 3.5% per year on average for the past 5 years. There are 5000 teachers for the 2007-2008 school year and this number typically grows proportionately to the percent change in pupils. Therefore, to accommodate future expansion of the school system, the administration building will need to grow proportionately. Though this may be, in actuality, a desire and requirement of the owner at a future time, this will only be investigated as an academic study.

The investigation involved research into potential site layouts for future expansion. The architect originally indicated a desire to expand the building horizontally (See Figure 6). This was considered as a possibility and, in addition to this original proposal, an expansion of a portion of the building vertically was considered. While it is certainly possible to add to the building on the horizontal plane, after fully investigating each of these options, the final proposal is to expand the rectangular wing of the building vertically. This will include the addition of two stories, each 15’-4”, mimicking the existing floor-to-floor heights. This plan was determined to be more feasible due to several factors. The first factor is the benefit of having a single construction sequence. One of the other benefits over the alternative is the ability to maintain the site as in without infringing on, and possibly crowding, the existing site. Along with this, should parking become a concern, this proposal will leave ample room for the addition of any needed parking expansions. Of course, a substantial amount of first cost will need to be allocated to achieve this goal. This is investigated later with a construction management analysis.

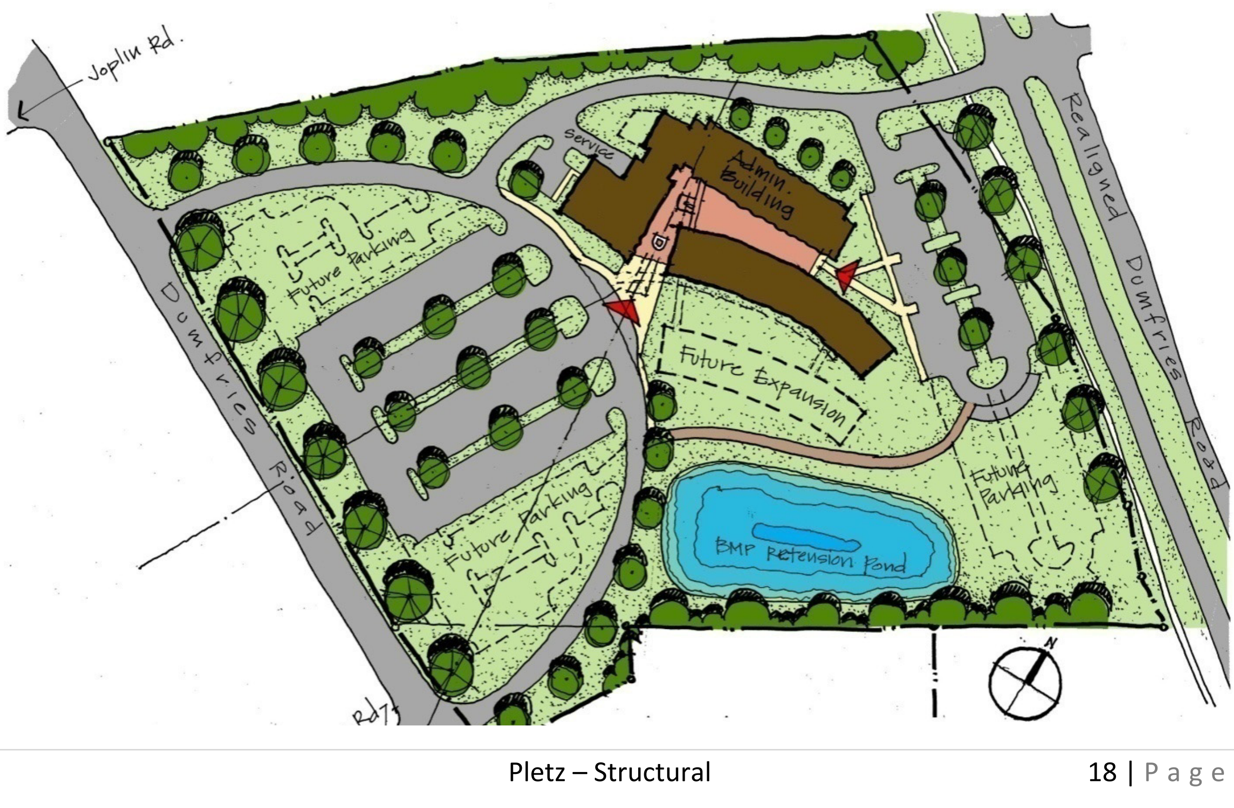


Figure 6. Building Site Plan with Outline of Originally Proposed Future Expansion.

Additional figures are shown below that show the building along with the site context. The first image (Figure 7) is an aerial view of the building looking at the north façade. The image shows the one-story wing on the left which connects to the three-story wings on the right.

The second image (Figure 8) shows the south of the building in an aerial view. This rectangular wing of the building is the primary focus of this study. In addition, however, the radial wing shown in Figure 7 will also be studied for the feasibility of adding a garden roof.

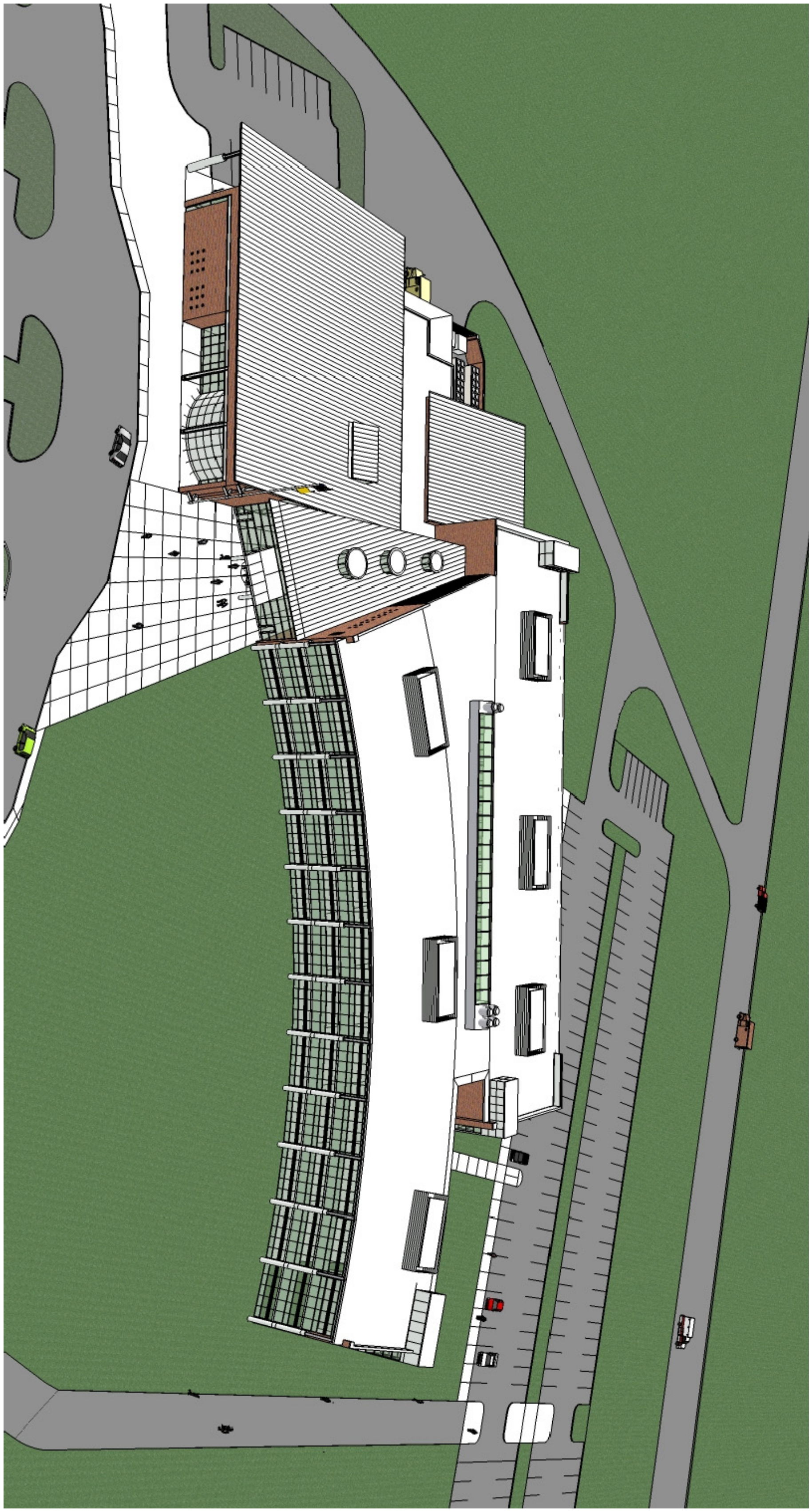


Figure 7. 3D Aerial View at North

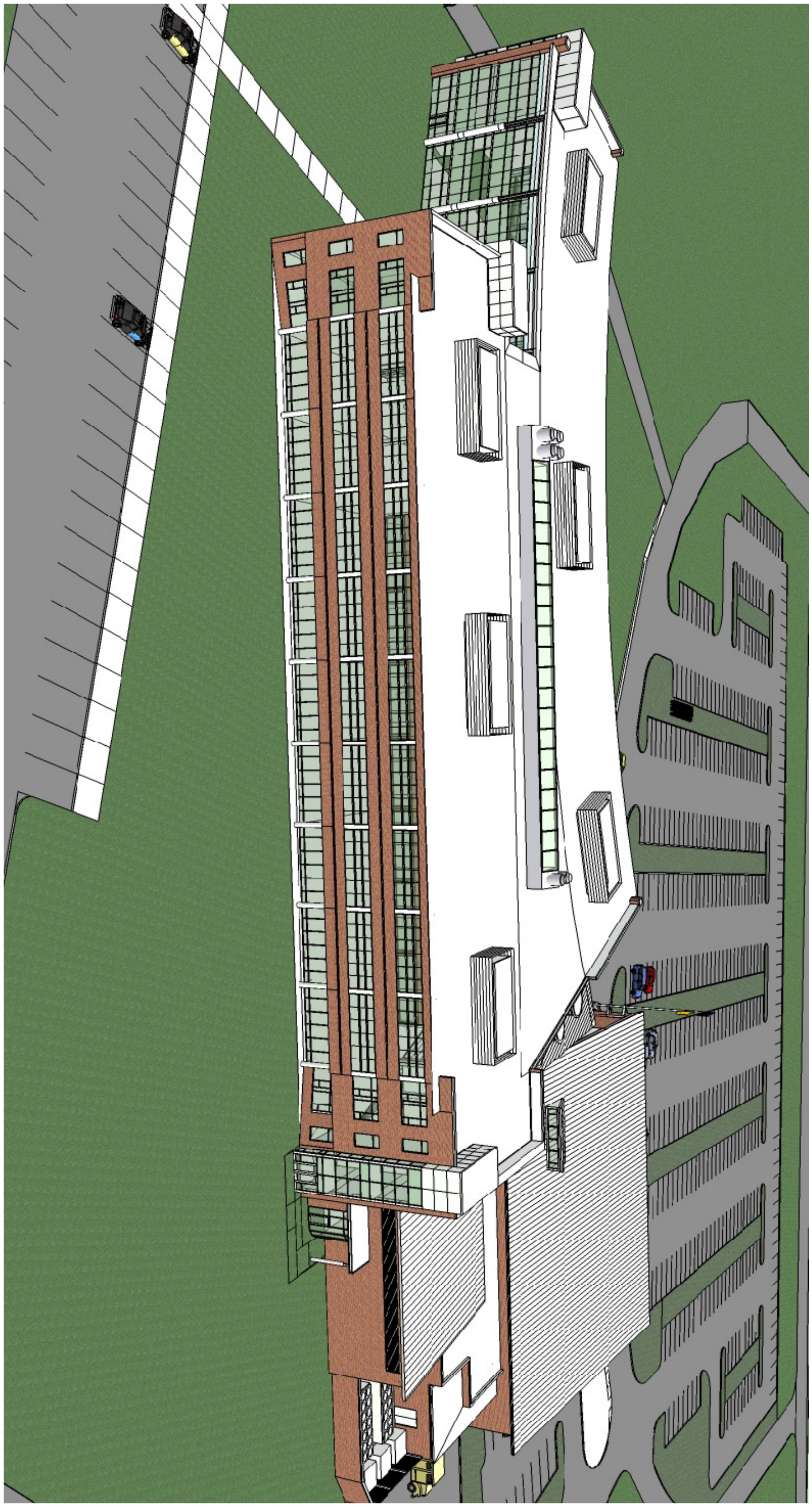


Figure 8. 3D Aerial View at South

The following two figures are representations of the 2 additional stories added to the 2-story rectangular wing. The above figure (Figure 9) is an elevation of the new wing and the figure below (Figure 10) is a 3D model of the back of the wing.

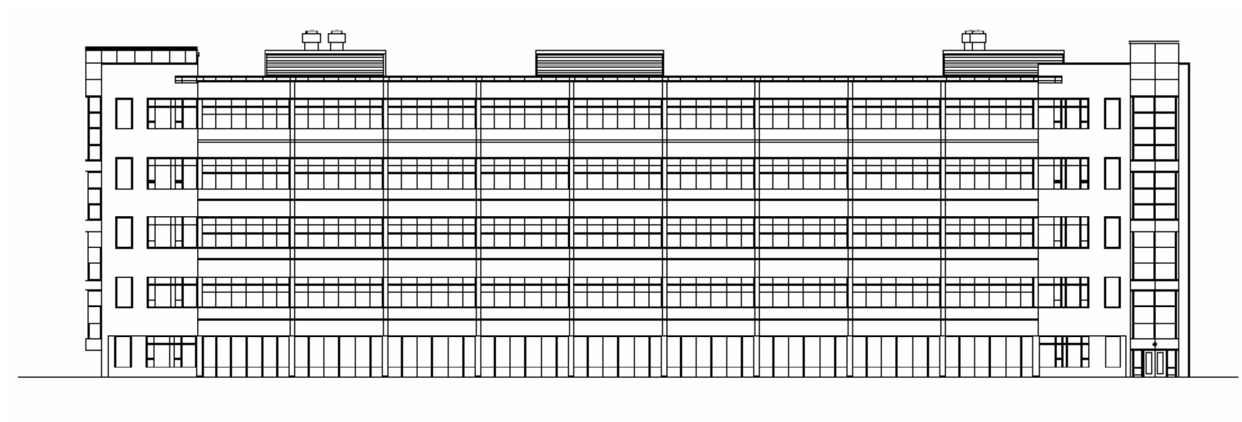


Figure 9. South Elevation of Proposed 5-story Rectangular Wing

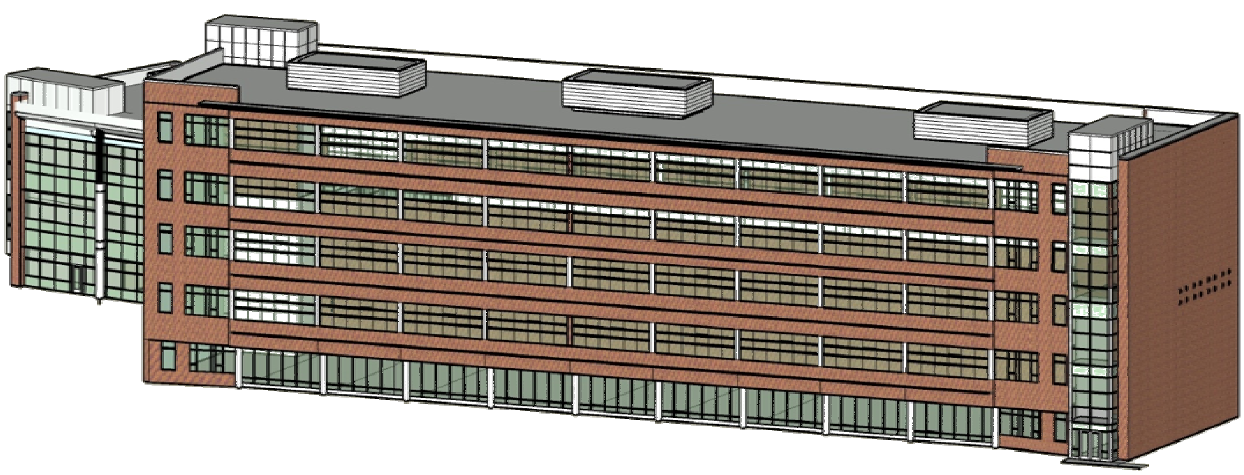


Figure 10. 3D Model of Proposed 5-story Rectangular Wing with Radial Wing in Back

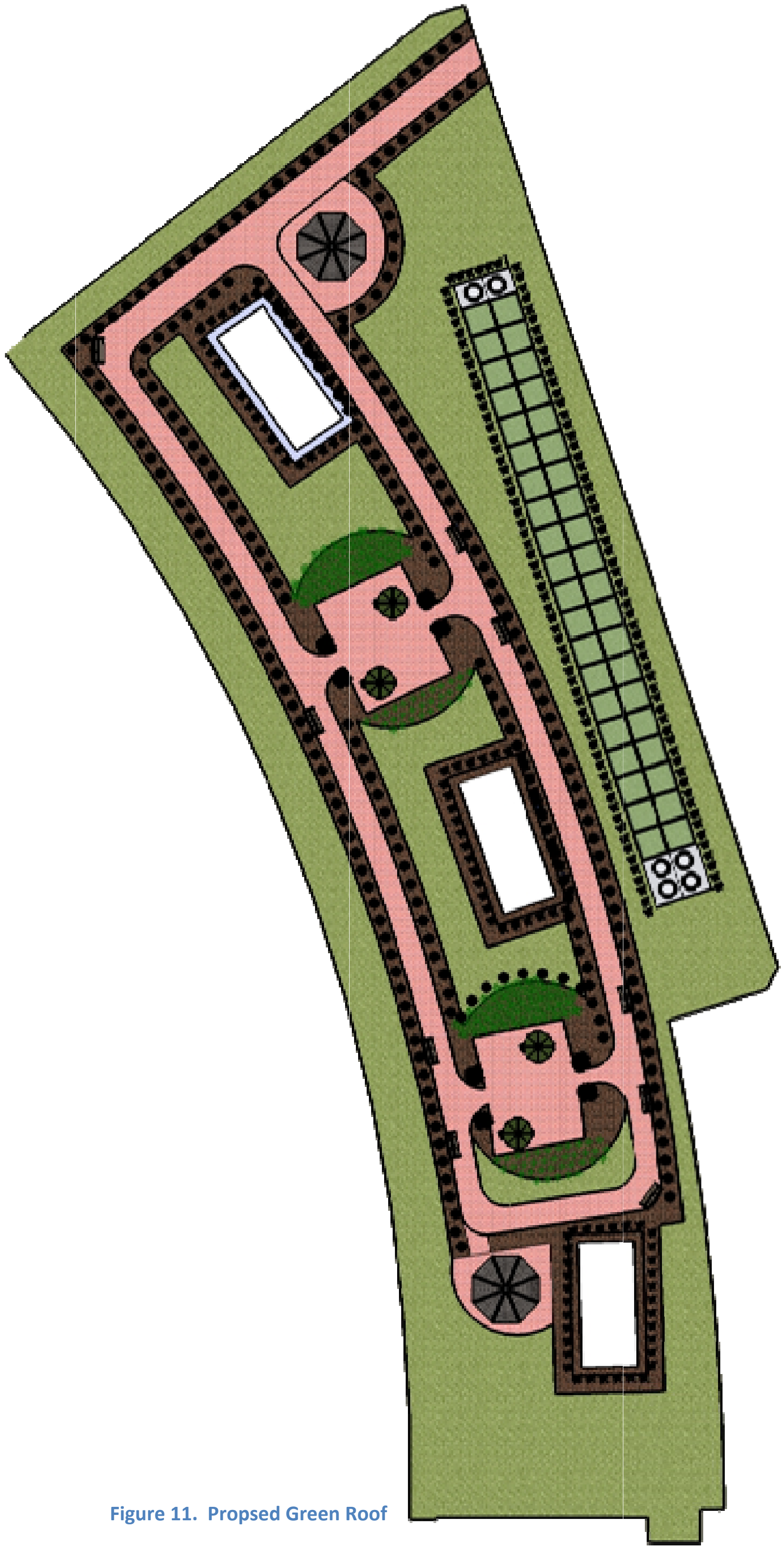


Figure 11. Propsed Green Roof

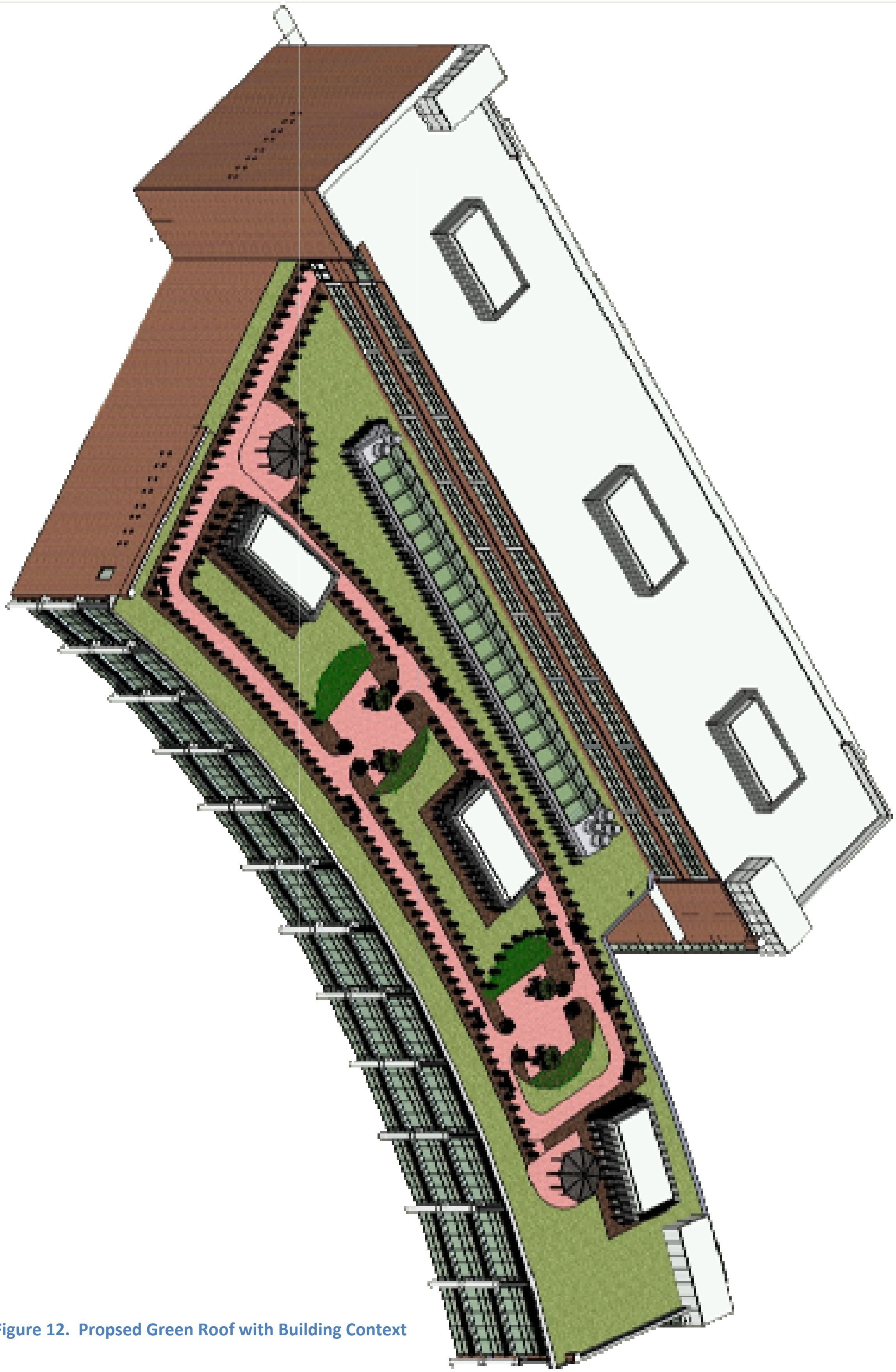


Figure 12. Propsed Green Roof with Building Context

A problem that occurs when the previously three-story rectangular wing is increased to five stories is the differing heights with the adjacent atrium and radial wing. The architectural experience is less appealing from the rectangular wing at these levels when viewing the outside onto the top of the other wing and atrium consisting of roofing materials and mechanical units. To achieve a more desirable aesthetic, it is proposed to include an expansive landscaped roof covering the atrium and radial wing. The proposal includes an approximate 30,000 square feet of landscaped area. This can be broken down into 5360 square feet of hardscape (18%), 1715 (6%) square feet of existing mechanical area and 22925 square feet of softscape (76%).

Figure 11 shows the model of the proposed green roof and Figure 12 shows the green roof in the context of the complete building. This roof will not only increase the appeal of the view, but also provide an enjoyable atmosphere for workers to each lunch or relax in a refreshing environment. In addition, the roof will provide yearly energy savings due to the higher insulation values.

This green roof was designed to be an intensive garden roof. This means that plants up to 5 feet tall can be planted in the garden. The typical construction of the green roof starts with a roofing membrane protection material. This material was chosen to by Hydroflex RB at 3/8 inches thick due to the intensive nature of the plants (See Appendix A for details of these garden roof materials). Next, insulation is placed on top of the membrane. The insulation chosen was 3 inches of Dow STYROFOAM. For drainage and water retention, a moisture mat is required. Moisture Mat SSM45 was chosen at 3/16 inches. Next, a “container” for the soil is created with Floradrain FD60 material filled with mineral soil. This material is 2 ¼ inches thick. The substrate soil chosen is an intensive soil composed of 55% mineral soil and 45% organic soil at 6 inches. The vegetation, as mentioned previously, is allowed up to 5 feet tall. Where vegetation-free zones are required, conventional pea gravel is required at 6 inches thick. In “traffic” areas, concrete pavers were specified. This entire construction for this set of materials costs $22-25 per square foot. See the Construction Management section for further details of the cost of the garden roof construction. Also, see the Structural section for details about the weights of these materials. The following figure (Figure 13) shows an illustration of the section with the materials labeled.

The following figures are representative of typical required sections required in the construction of the

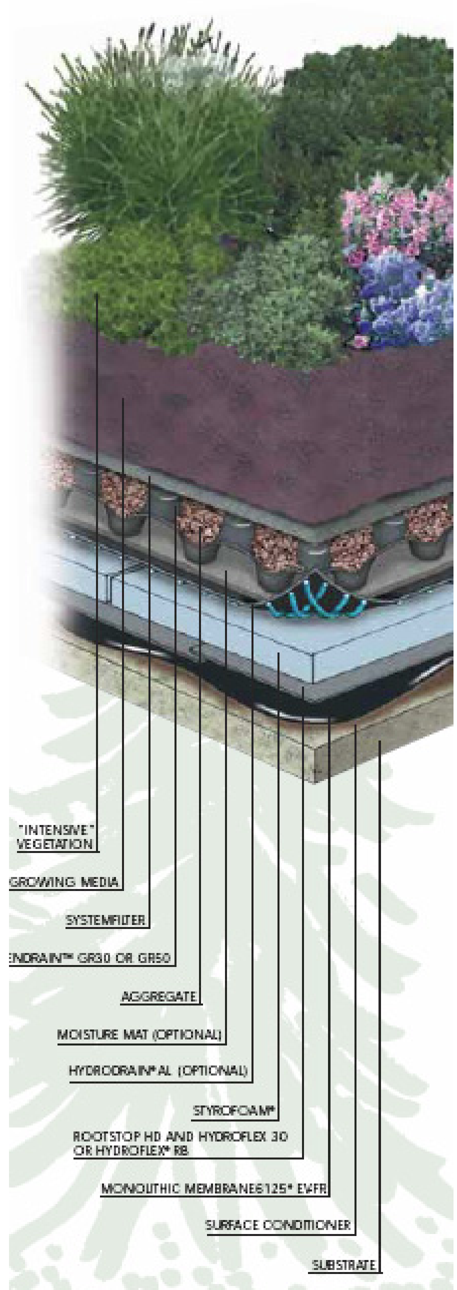


**Figure 13. Green Roof Illustration.
Source: American Hydrotech, Inc.**

garden roof. The first figure (Figure 14) shows a typical detail of the termination of an intensive garden roof. This would be located, for example, at the edge of the building. No vegetation is permitted in the "vegetation-free zone" which exists 1'-6" from the edge of the building.

The next figure (Figure 15) shows typical details for the transition of the garden roof. For example, this would be located where the hardscape (pavers) transition into the softscape (vegetation). Soil stops are required between the hardscape and softscape as shown in these sections. Soil stops can be constructed from concrete or timber.

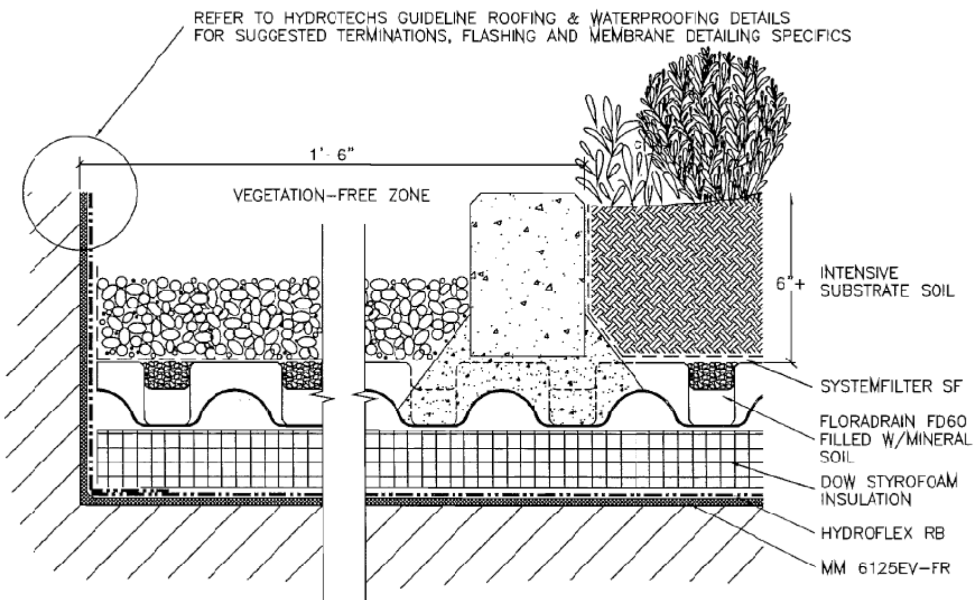


Figure 14. Typical Detail of Intensive Green Roof Termination Area

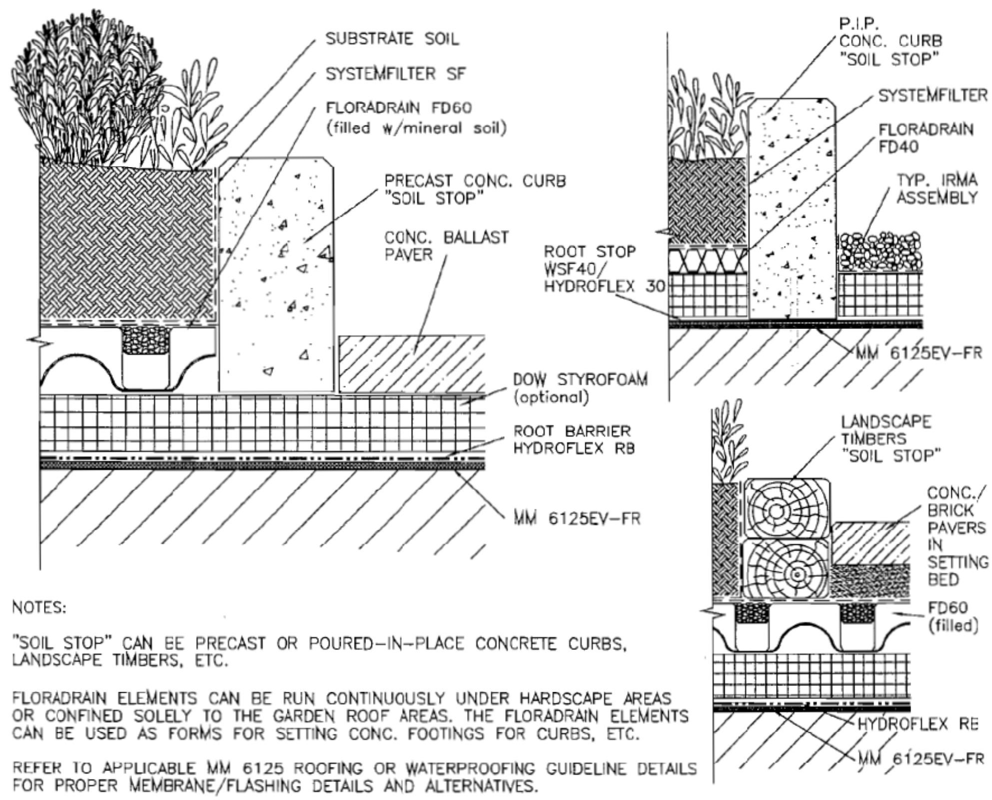


Figure 15. Typical Softscape to Hardscape Transition Detail

The following figures are the architectural floor plans for the rectangular wing of the building. Figure 16 is the first floor architectural plan, Figure 17 is the second floor, and figure 18 is the third floor plan. While all of the plans exhibit the same general program, each floor has its own unique features. For example, in the lower left of the third floor are open office workstations. On the second floor, there are meeting and work rooms, and on the first floor there is a conference room and copy room. No two parts of the building have a standard floor plan. This makes for a complicated situation when investigated lateral systems because one area may make for a great area to place a lateral member on one floor plan where on the other two floor plans obstructions like an open corridor occur. This will be discussed further in the Structural section of this study.

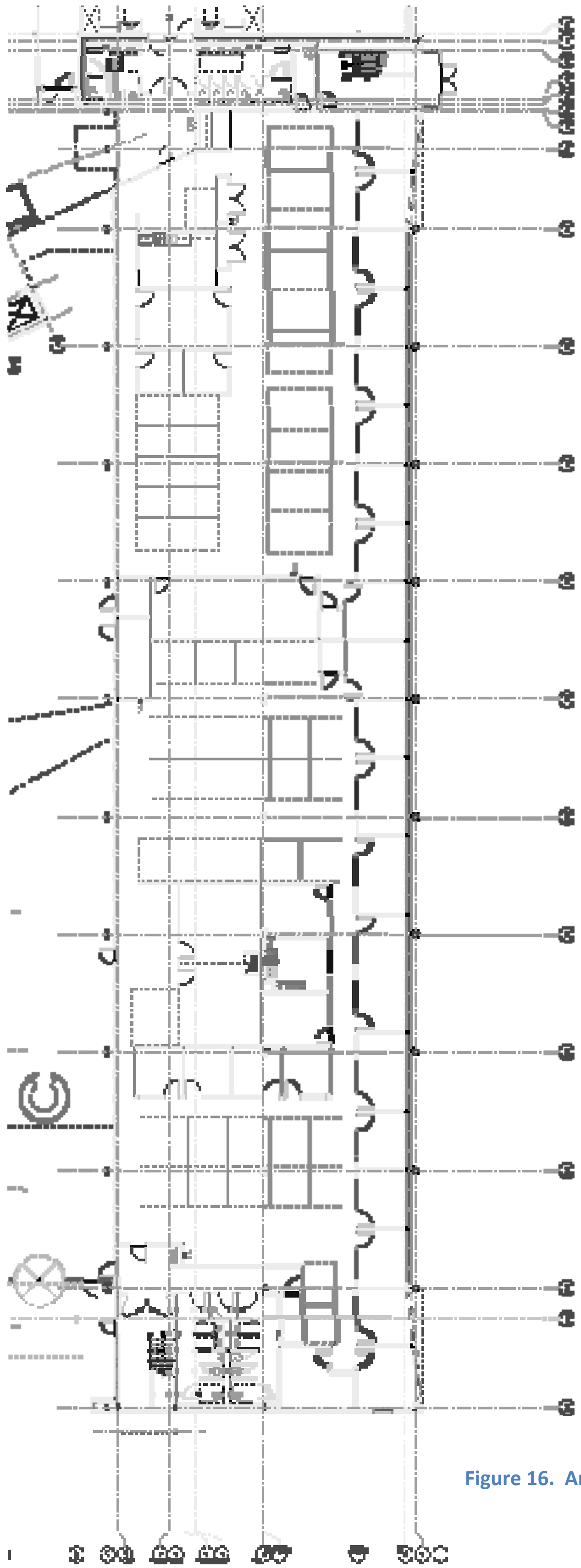


Figure 16. Architectural Floor Plan – First Floor

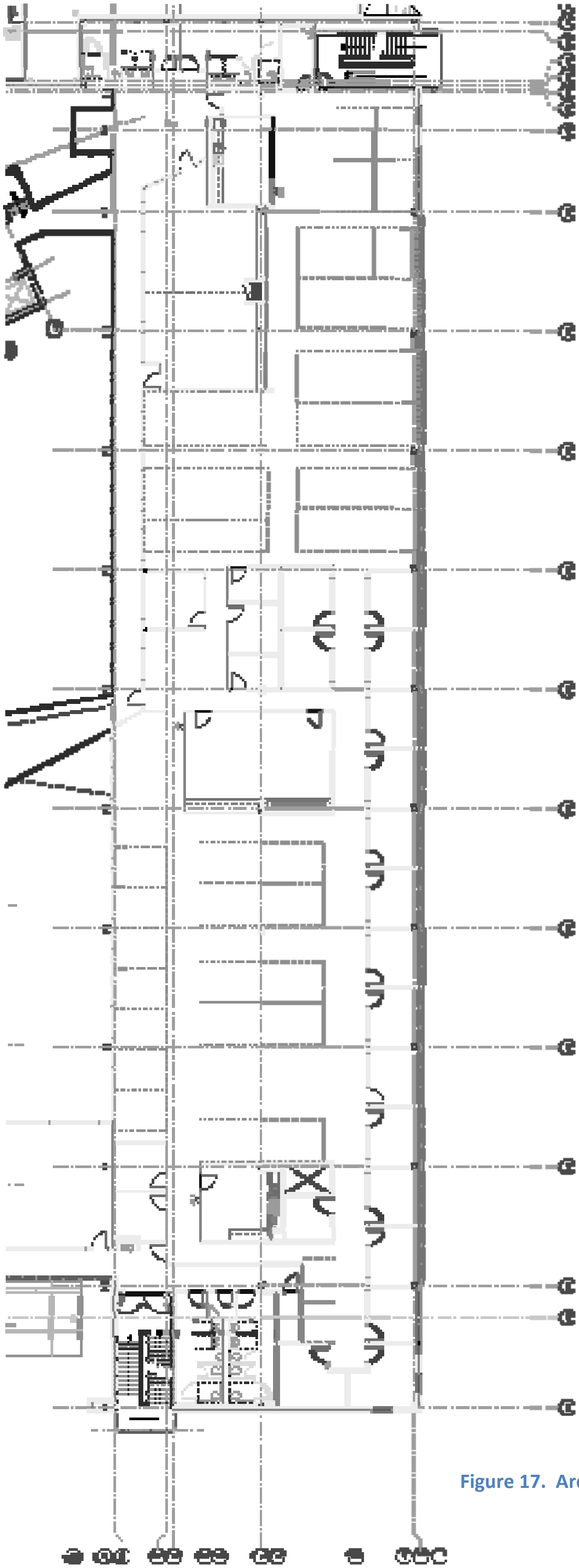


Figure 17. Architectural Floor Plan – Second Floor

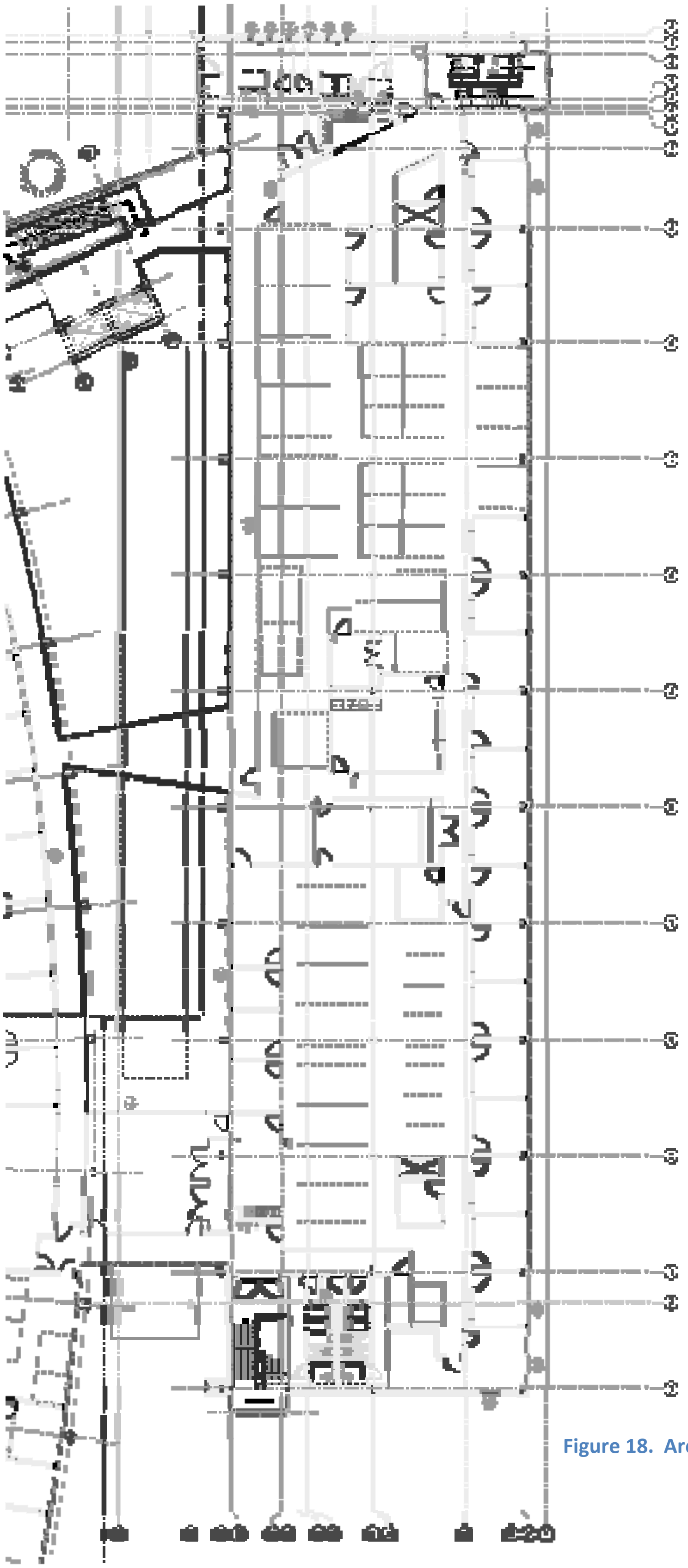


Figure 18. Architectural Floor Plan – Third Floor

Structural Depth

Existing System Summary

The existing gravity framing system consists of steel columns supporting steel framing with open web steel joists as filler beams in between bays. The steel beams and joists are non-composite with a 4 ½" non-composite concrete slab on metal decking. The bays span 31'-0" in the north-south direction and 24'-0" in the east-west direction. Joists are primarily K-series joists with 4'-0" spacing. Sizes are typically in the range of 28K9. Several KCS joists are specified in areas where special loading occurs, such as under heavy file storage rooms and to support mechanical roof top units. Columns along the northernmost exterior wall are HSS shapes while all other columns are W shapes. The HSS shapes are HSS12.75x0.375, interior W-shapes are W14x68 and the columns adjacent to the atrium are W14x82. The exterior foundations are 9'-0"x9'-0" with (10)#7 bars each way and interior foundations are 10'-6"x10'-6" with (10)#7 bars each way.

Framing Analysis

The new proposal for the gravity system involves several changes. Joists at 4'-0" on center (6 per bay) will be eliminated as filler between bays. They will be replaced with steel beams at 8'-0" on center (3 per bay). In addition, the slab will act compositely with the steel beams. The deck will be changed from the non-composite deck to a composite 1.5VL20 deck from Vulcraft with 4" concrete slab. See Appendix A for details of the deck. Gravity framing analysis was performed in RAM Structural System and supplemented with hand calculations. Figures 20 and 21 show the results of the new gravity system.

The following loading was input into the program for gravity load (Figure 19).

Gravity Loading			
	Type		Load (PSF)
Roof			
	Dead	Superimposed	20
	Live	Snow	25
Floor			
	Dead	Deck/Slab	45
		Superimposed	15
	Live	Corridor	80

Figure 19. Gravity Loading Table

As shown in Figure 20, the roof framing consists of W14x22 beams typical in the bays and girders. The exterior (northernmost) girders are sized at W16x31 where the floor slab is cantilevered 3'-10". Where heavier loads occur due to roof top mechanical units, W16x26 are common. Sample beam designs for each case are supplied in Appendix A with supplemented hand calculations.

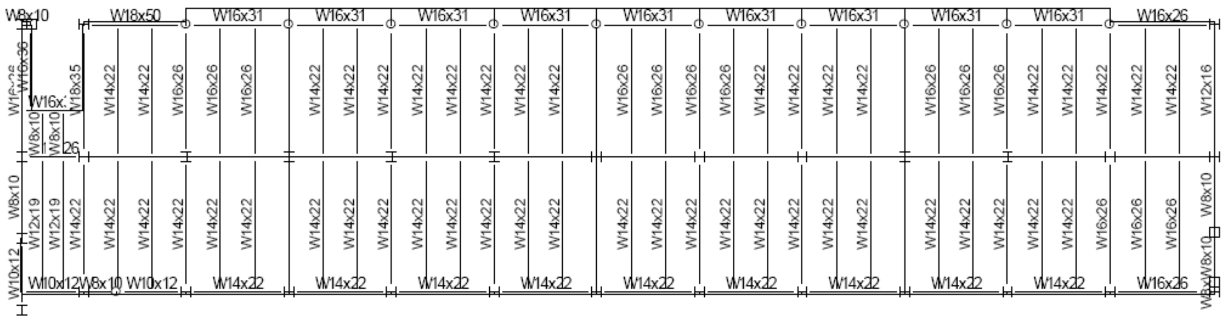


Figure 20. Results of Roof Framing

Floor beams are also typically W14x22 with exceptions in certain areas. The southern bays are typically W12x19. Beams that connect column to column in this area are also much larger due to the point loads applied where beams are framed into the beam rather than into the column.

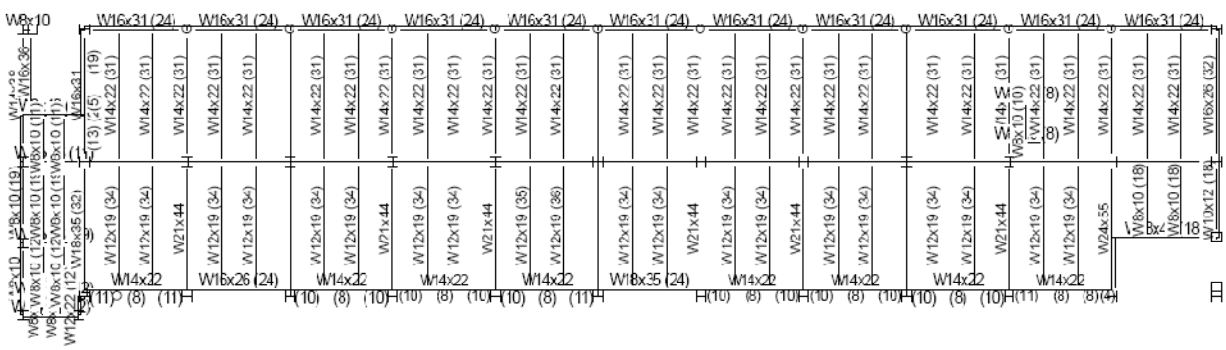


Figure 21. Results of Floor Framing

The number of studs is designated by the number in parentheses in the framing plan after the beam size.

Column Analysis

Column splicing occurs after the second floor at a height of 30'-8" so that the first two levels are the same column size and the top three stories have the same column size. Columns were given trial sizes of W10, W12, or W14, where possible, and HSS8x8, HSS10x10, or HSS12x12, and HSS8.00, HSS10.00, or HSS12.00 for square and round hollow structural shapes, where applicable. The column designs are provided in Figure 22 and 23. Typical sizes for W-shapes include W14x53 for the top floors and W14x120 for the bottom floors, as described above. Extraneous exterior shapes include W10x33 and W14x33. Round HSS shapes are commonly HSS10.00x0.25 for the upper floors and HSS10.00x0.625 for the lower floors. For more detailed information, please reference Figures 22 and 23. Columns changed in size from W14x82 in the original design to W14x53 in the redesign. The new columns represent an approximate 35%

reduction in weight. The reduction in weight is primarily due to the fact that these columns are now only taking gravity loading as opposed to being part of a moment frame. See the Construction Management section for further details on the impact of the new weights on the cost of the building. The new lateral columns will be discussed later in the Lateral Analysis section. Please reference Appendix A for further details on the gravity columns.

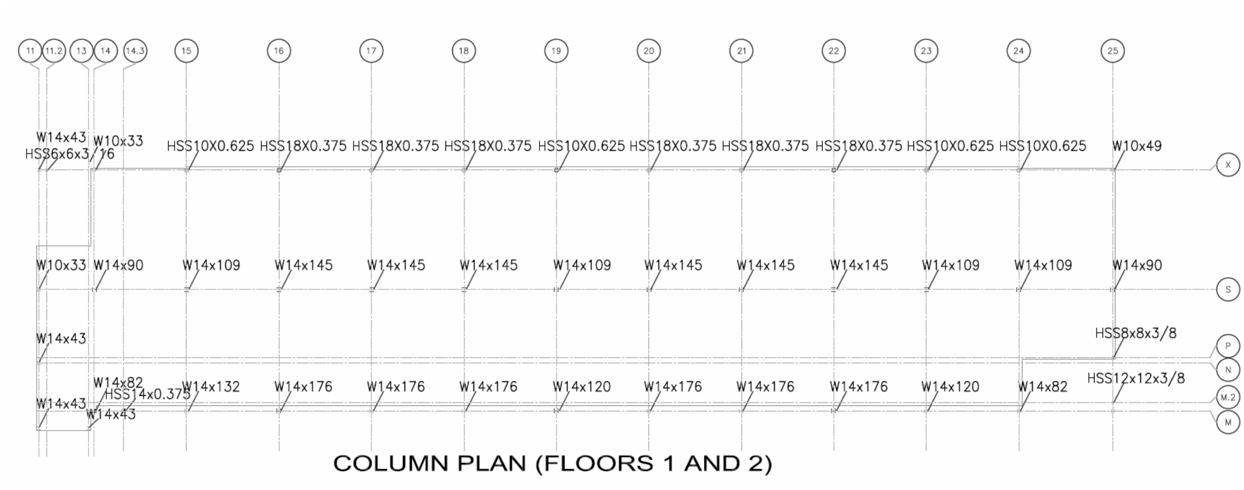


Figure 22. Column Plan for Floors 1 and 2

Figure 23. Column Plan for Floors 3, 4, and 5

Lateral Analysis

Seismic Changes

The seismic location of the building remains the same, so much of the initial calculations remain intact during the redesign. There are, however several changes that must be made. First, the height of the building has increased from 46’-0” to 76’-8” to the top of the fifth floor. The weight of the building has also changed. Each of the floors up to the roof weigh 1283 kips and the roof structure weighs 683 kips. The period thusly changes due to the increase in height from

0.5989 in the existing building to 0.904 in the redesign. The addition of weight causes a redistribution of the seismic forces at each story. The following table (Figure 24) shows the distribution of these forces. The table in Figure 25 shows the seismic calculation for each story. Figure 26 shows the east-west seismic forces. This is the direction for which design is controlled by the seismic forces.

TOTAL BUILDING WEIGHT		
FLOOR 1	1282.6	kips
FLOOR 2	1282.6	kips
FLOOR 3	1282.6	kips
FLOOR 4	1282.6	kips
ROOF	682.8	kips
	5813.2	kips

Figure 24. Total Building Weight. See Appendix A for Further Details

Seismic Calculations							
x	Wx	hx	k	Wxhx^k	Cvx	V	Vx
Story	Weight	Height					
1	1282.6	15.33	1.14	28814.381	0.067132	177.16	11.89302
2	1282.6	30.66	1.14	63501.427	0.147945	177.16	26.20997
3	1282.6	46	1.14	100840.54	0.234938	177.16	41.62154
4	1282.6	61.33	1.14	139971.11	0.326104	177.16	57.77253
5	682.8	76.66	1.14	96095.259	0.223882	177.16	39.66294
			Total	429222.72			

Figure 25. Seismic Story Force Calculations

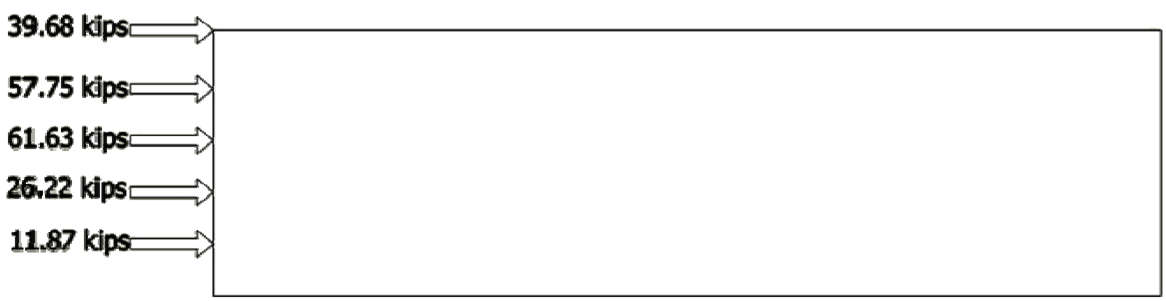


Figure 26. East-West Seismic Story Loading

Wind Changes

For the same reasons that seismic forces changed, the lateral forces from wind will also change. See Appendix A for calculations of wind forces. Figure 27 below displays the wind forces on the north-south face of the building. Figure 28 shows the wind force on the east-west face of the building.

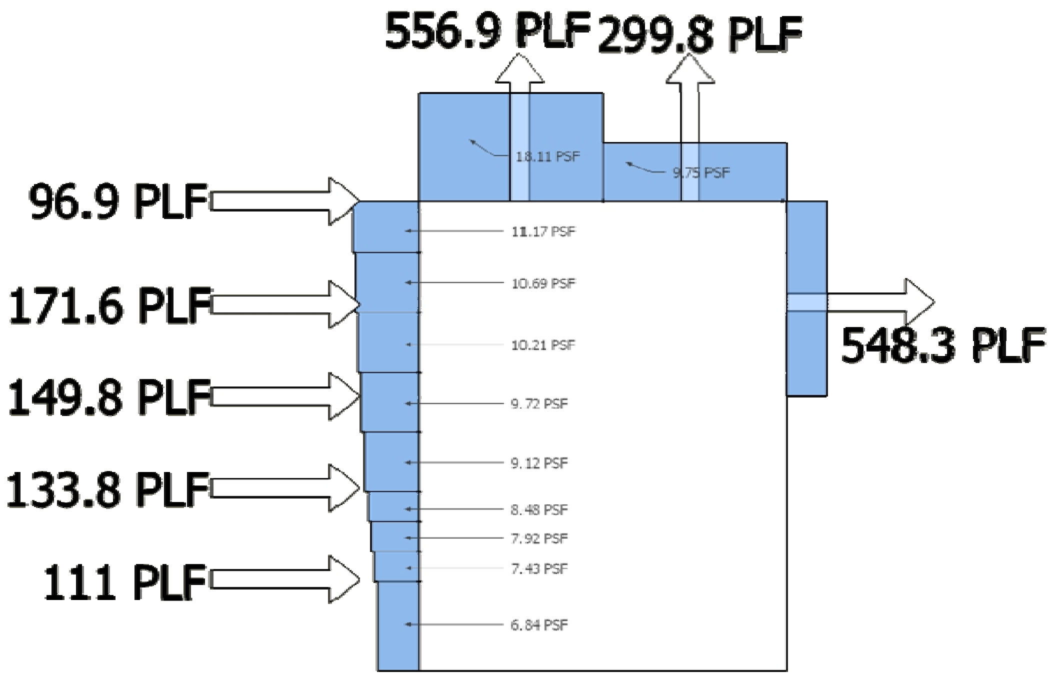


Figure 27. Wind Forces in the North-South Direction

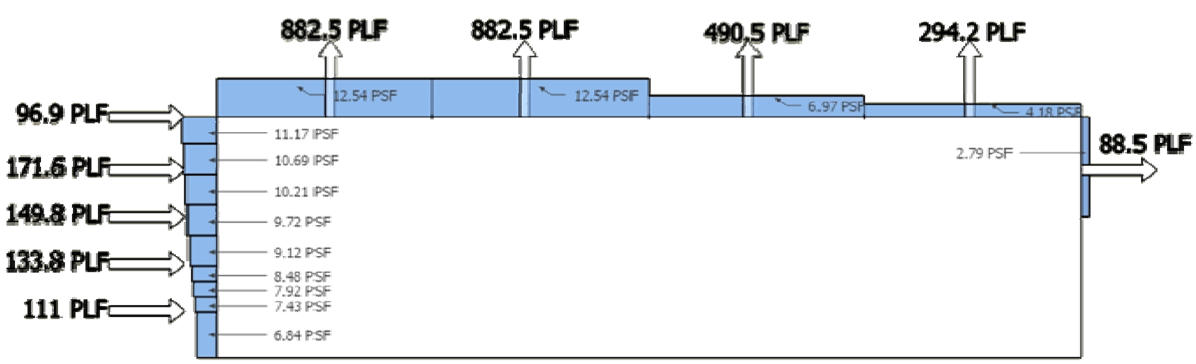


Figure 28. Wind Forces in the East-West Direction.

Existing Lateral System

The existing lateral system consists of entirely moment frames. In the east-west direction, there are 11 moment frames that are two bays deep (61'-6"). In the north-south direction, there are 3 moment frames that are 10 bays deep (240'-0").

When analyzing the system as a whole, it seemed at first glance rather excessive. On each floor there are 31 beams connected via a total of 96 moment connections to columns. At first, a new lateral system consisting of braced framing was proposed as a solution. After investigation into the feasibility of adding braced frames, it became cumbersome and ultimately determined as a less desirable alternative. Firstly, the exterior walls consist of ample amounts of openings and would, therefore, be inconvenient and inefficient to locate braced framing. As for the interior, as shown in Figure 16-18, the architectural plans prohibit any convenient locations for braced frames. The architectural layout on each floor is very different. Please note Figures 16, 17, and 18 which represent the plans for each of the original three floors. A convenient place to locate a braced frame on any one floor is not convenient or practical on any of the other two floors. It would require considerable architectural realignment to provide a convenient means to add braced framing. The main reason for the problem occurs due to the fact that the architectural program calls for a very open floor plan. Each floor has a minimum of 60 open-office workstations. Therefore, braced frames would often interfere with the architectural goals of the building. Because of these problems, it became clear after initial investigations why the design team chose moment frames as the lateral system. Masonry or concrete shear walls were also considered for the lateral system. For the same reasons from above, it was determined that this system would not be appropriate without a complete and major overhaul of the architectural flow of the building.

Therefore, the lateral system in the redesign remains as moment frames. However, the number of frames will be significantly reduced. During the first iteration of redesign, three moment frames were chosen. This resulted in almost twice as much drift as desired. One frame in the east to west direction was adequate for the lateral forces. Therefore, there will be 6 N-S frames in the east-west direction (2 bays, 61'-6") and 1 E-W frame (11 bays, 264'-0") in the new system. The load calculations for the new system can be found in Appendix A. The applied load diagrams can be found previously in Figures 26-28. The design of the north-south frames are shown in Figure 30 and the east-west frame is shown in Figure 31. The envelope of load combinations was based off of ASCE7-05. The following Table 29 displays the envelope of load combinations.

Load Combinations										
	Factor	Load	Factor	Load	Factor	Load	Factor	Load	Factor	Load
ASCE 1	1.4	DL								
ASCE 2 (a)	1.2	DL	1.6	LL	1.6	LLS	0.5	RLL		
ASCE 2 (b)	1.2	DL	1.6	LL	1.6	LLS	0.5	SL		
ASCE 2 (c)	1.2	DL	1.6	LL	1.6	LLS	0.5	RL		
ASCE 3 (a)	1.2	DL	1.6	RLL	1	LL	1	LLS		
ASCE 3 (b)	1.2	DL	1.6	RLL	0.8	WL				
ASCE 3 (c)	1.2	DL	1.6	SL	1	LL	1	LLS		
ASCE 3 (d)	1.2	DL	1.6	SL	0.8	WL				
ASCE 3 (e)	1.2	DL	1.6	RL	1	LL	1	LLS		
ASCE 3 (f)	1.2	DL	1.6	RL	0.8	WL				
ASCE 4 (a)	1.2	DL	1.6	WL	1	LL	1	LLS	0.5	RLL
ASCE 4 (b)	1.2	DL	1.6	WL	1	LL	1	LLS	0.5	SL
ASCE 4 (c)	1.2	DL	1.6	WL	1	LL	1	LLS	0.5	RL
ASCE 5	1.2	DL	1	EL	1	LL	1	LLS	0.2	SL
ASCE 6	0.9	DL	1.6	WL						
ASCE 7	0.9	DL	1	EL						

Figure 29. Table of Load Combinations

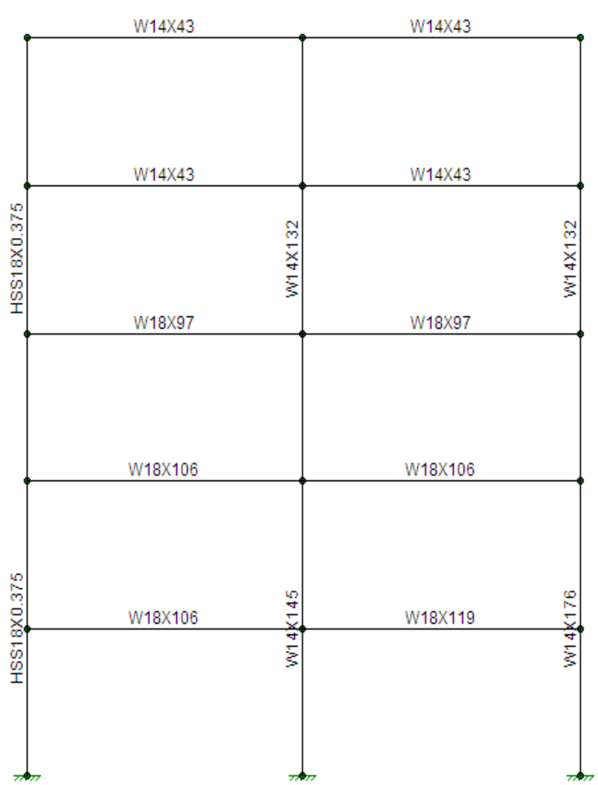


Figure 30. North-South Moment Frame Design

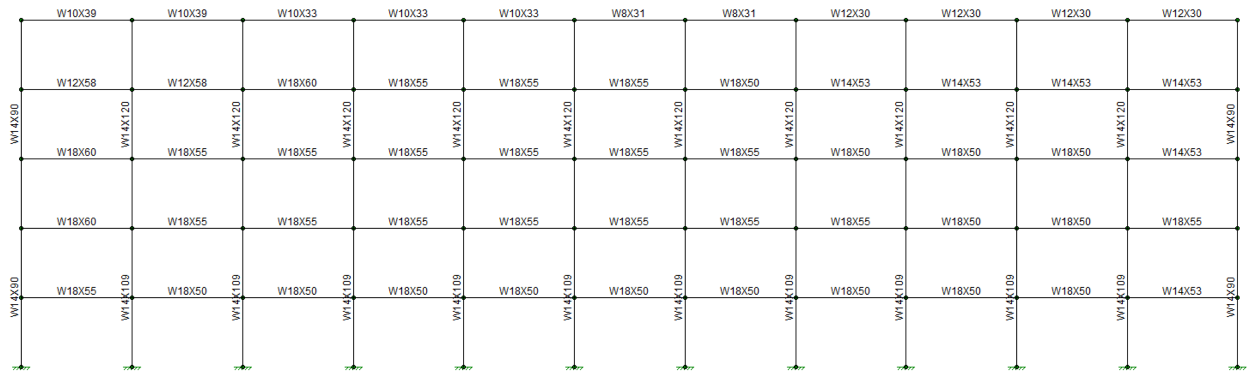


Figure 31. East-West Moment Frame Design

Torsion

The building geometry is roughly rectangular and therefore the center of mass is approximately in the center of the floor plan on each story. The center of mass was found to be located at x=138'-8" and y=36'-7" from the left lower-most column as the reference (See Figure 22). The center of rigidity was found using the following equations.

$$\frac{\sum\left(R_{x,i}\cdot y_i\right)}{\sum R_{x,i}}=\frac{\begin{bmatrix}\left(1\cdot 62'\text{-}4\tfrac{1}{2}"\right)+\left(1\cdot 86'\text{-}4\tfrac{1}{2}"\right)+\left(1\cdot 110'\text{-}4\tfrac{1}{2}"\right)\\+\left(1\cdot 158'\text{-}4\tfrac{1}{2}"\right)+\left(1\cdot 182'\text{-}4\tfrac{1}{2}"\right)+\left(1\cdot 206'\text{-}4\tfrac{1}{2}"\right)\end{bmatrix}}{6}$$

$$=134'\text{-}4\tfrac{1}{2}"$$

$$\frac{\sum\left(R_{y,i}\cdot x_i\right)}{\sum R_{y,i}}=\frac{\left(1\cdot 35'\text{-}11"\right)+\left(1\cdot 35'\text{-}11"\right)+\left(1\cdot 35'\text{-}11"\right)+\left(1\cdot 35'\text{-}11"\right)+\left(1\cdot 35'\text{-}11"\right)+\left(1\cdot 35'\text{-}11"\right)}{6}$$

$$=35'\text{-}11"$$

The center of rigidity as shown above is located at x=134'-4½", y=35'-11". The difference in location of the center of rigidity from the center of mass is therefore x=4.29' and y=0.667'. Because the locations are very close to one another, torsion is not of considerable issue. However, it still must be addressed.

For the east-west direction, the center of rigidity is located at the center point of the frame because there is only a single frame. The location is therefore x=146'-4½", y=35'-11". The difference in location of the center of rigidity from the center of mass is therefore x=8', y=0.667'. Figure 32 shows a visual representation of the story shear as well as its eccentricity from the center of rigidity.

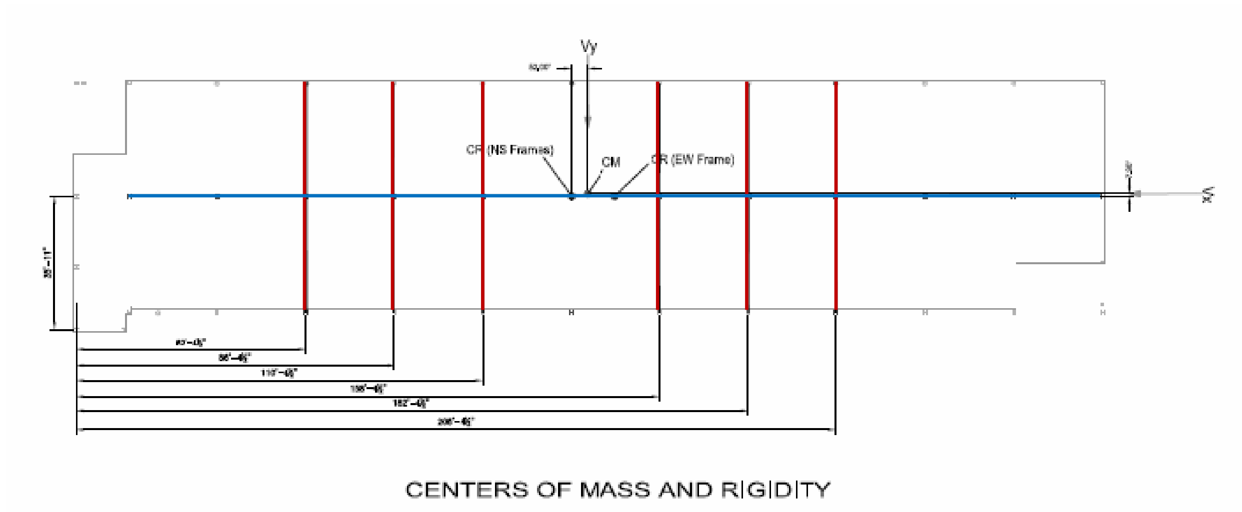


Figure 32. Diagram of the Center of Mass and Center of Rigidity

Story Drift

In the east-west direction, seismic is the controlling load combination. Table 33 illustrates the story drifts in the east to west direction. In the north-south direction, where the face of the building is much wider, wind controls the drift of the building and the design of the members. Table 34 shows the story drift of the north-south frames. The drift of each of the north-south frames was designed with a maximum story drift of

$$\Delta_{story,wind} = \frac{H}{400} = \frac{15'\text{-}4" \cdot 12 \, {}^{in}\!/_{ft'}}{400} = \frac{184"}{400} = 0.46"$$

The drift of the east-west frames was designed with a maximum drift of

$$\Delta_{story,seismic} = 0.015H = 0.015 \cdot 15' - 4" = 0.015 \cdot 184" = 2.76"$$

Story Drift of East West Frame					
Story	Drift (in.)	Load Combination	Cd/I	δCd/I	0.015H
1	0.54	Seismic	3	1.617	2.76"
2	0.75	Seismic	3	2.241	2.76"
3	0.63	Seismic	3	1.899	2.76"
4	0.47	Seismic	3	1.413	2.76"
5	0.30	Seismic	3	0.909	2.76"

Figure 33. Story Drifts for Seismic Loading in the East-West Direction

Story Drift of North South Frame				
Story	Drift (in.)	Load Combination	% of Height	H/400
1	0.33	Wind	0.18	0.46"
2	0.46	Wind	0.25	0.46"
3	0.45	Wind	0.25	0.46"
4	0.42	Wind	0.23	0.46"
5	0.33	Wind	0.18	0.46"

Figure 34. Story Drifts for Wind Loading in the North-South Direction

Figures 35 and 36 show the deflected shape of the frame under the respective controlling load combination.

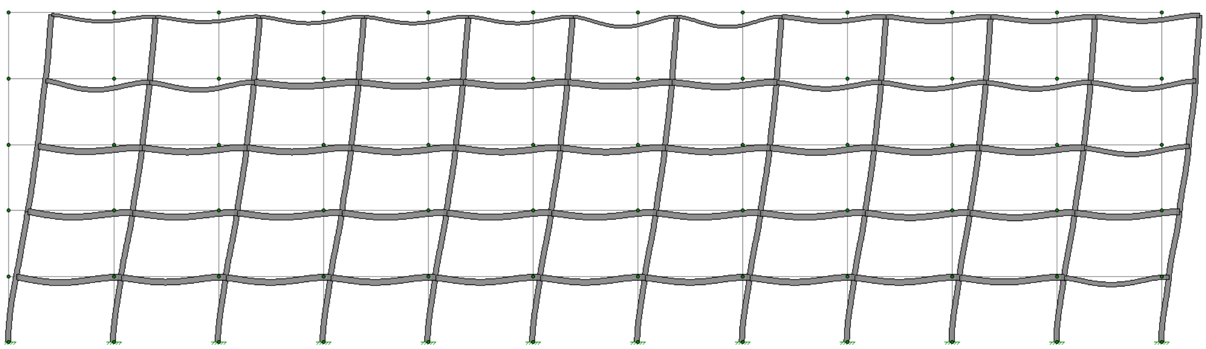


Figure 35. Frame Deflected Shape for the East-West Frame

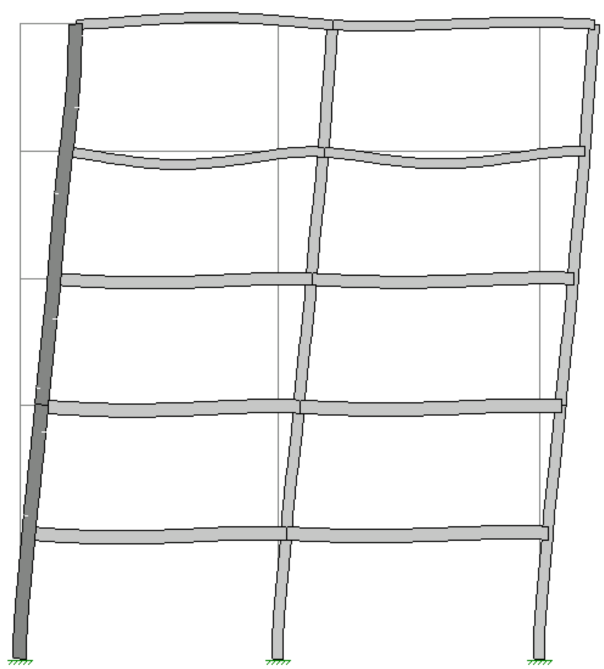


Figure 36. Frame Deflected Shape for the North-South Frame

Foundation Analysis

The existing foundation system consists of strip wall footings and spread column footings. The columns range in size from 9'-0"x9'-0" up to 10'-6"x10'-6" in this wing of the building. The new building will contribute a significant load addition to the foundation. The existing footings need to be checked for their adequacy and adjusted as necessary. The following table contains an excerpt of the complete foundation loading.

Description	Column Label	Column Coordinates	Axial Load (kips)
Southern Line	M-14	14.37,4.42	191.2
	M-15	38.38,4.42	235.4
	M-17	86.38, 4.42	228.3
	M-19	134.38, 4.42	332.8
	M-20	158.38, 4.42	252
Middle Line	S-14	14.37,35.92	394.8
	S-15	38.38,35.92	481.7
	S-17	86.38, 35.92	475.6
	S-19	134.38, 35.92	491.3
	S-20	158.38, 35.92	485.1
Northern Line	X-14	14.37,66.83	231.7
	X-15	38.38, 66.83	311
	X-17	86.38, 66.83	313.5
	X-19	134.38, 66.83	313.4
	X-20	158.38, 66.83	314.6

Figure 37. Sample Column Loading

The redesign of the foundation is presented in Figure 38 with accompanying calculations in Appendix A. The footing that was chosen was at grid line M-17 (86.38,4.42). The Axial Load at this column was 228.3 kips. The dead load was 106.5 kips and the live load was 121.8 kips. The applied moment, from wind analysis, is 291 ft-kips. The moment diagrams are presented in Appendix A. The footing results can also be found in Appendix A. The moment (caused by wind loading) created a controlling equivalent eccentricity of 48.9". The results for reinforcement were found to be an 11'-0"x11'-0" square spread footing that is 2'-1" thick with (11)#7 bars each way. This can be compared to the original footing which was 9'-0"x9'-0" with (10)#7 bars each way. The resulting increase in size and reinforcement was found to be relatively close to the size of the original footing and was logical due to the increase in axial load and moment.

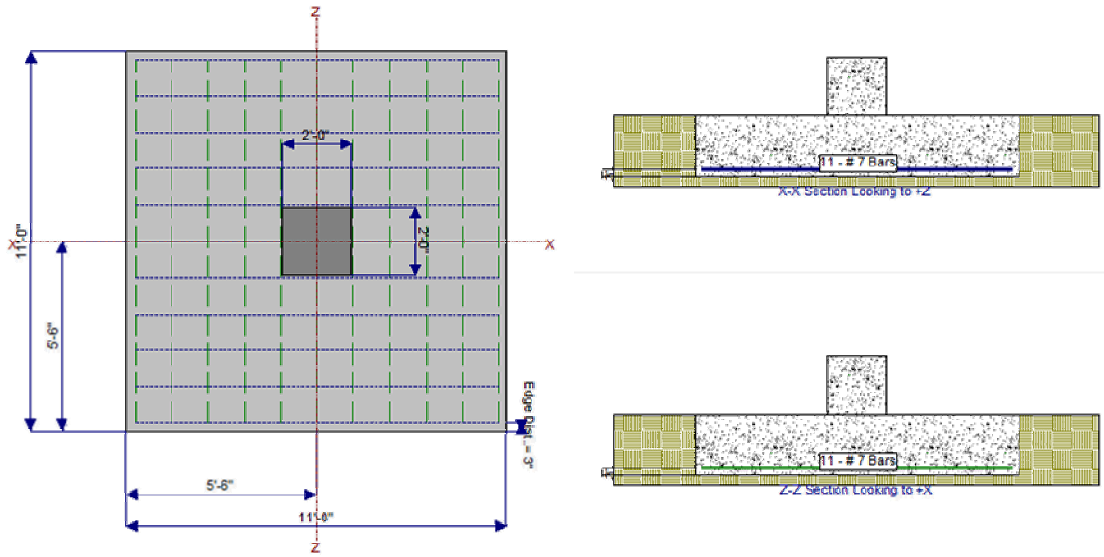


Figure 38. Redesign of Footing M-17

An interior footing was also tested for adequacy. The footing under consideration is S-17 (86.38,35.92). The axial load on the footing is 475.6 kips which includes 212.4 kips from dead load and 258.2 kips from live load. There is also a moment caused by wind loading of 255.2 foot-kips (see Appendix A). The existing spread footing is a 10'-6"x10'-6" footing with (10)#7 bars each way. The new footing is a 13'-6"x13'-6" square spread footing with (13)#8 bars each way. Appendix A contains further results from the design of this footing. The redesign of this footing is shown in Figure 39.

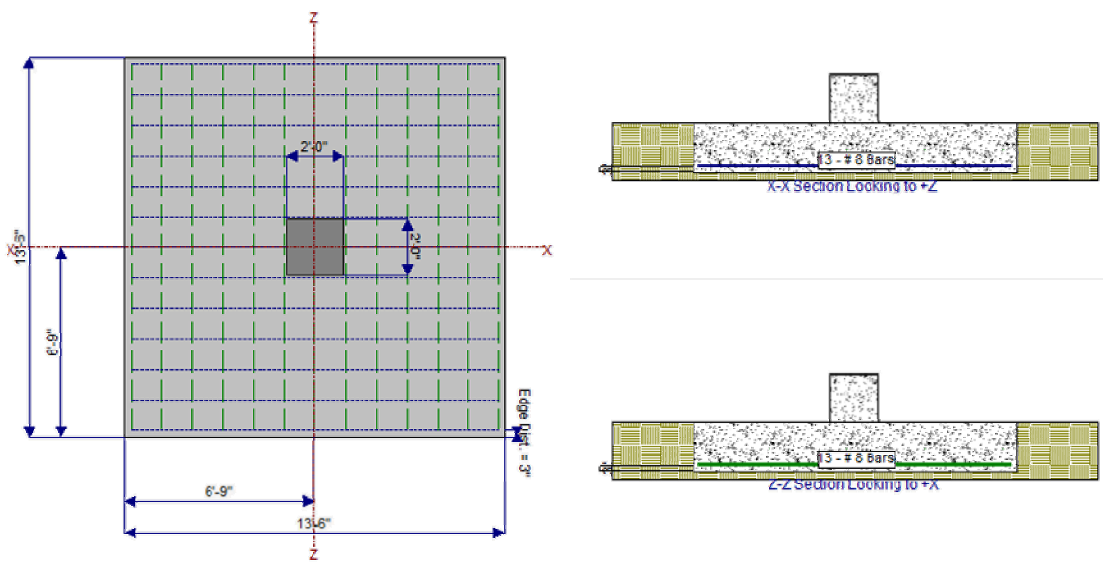


Figure 39. Redesign of Footing S-17

Green Roof

The existing roof of the atrium consists of steel beam framing while the radial building consists of open web steel joists much like the rectangular wing. As presented earlier in the architectural study, a green roof will be installed on these two portions of the buildings. This will contribute to a significant amount of load onto the structure. A breakdown of the loading that the green roof will apply to the structure is represented in Figure 40. The roof framing will be altered as it was in the rectangular wing to replace the OWSJ with steel beam sections but, of course, remaining non-composite.

Material	Height/ Thickness (inches)	Weight (psf)	
		dry	wet
Hydroflex RB	3/8	2.5	2.5
Dow STYROFOAM (R=5/inch)	3	0.5	0.5
Moisture Mat SSM45	3/16	0.2	1
FD60 w/ mineral soil	2 1/4	5	7.4
Pea Gravel	6	54	60
Intensive Soil	6	30	45
Intensive Plants (<5'-0")			4
Conrete Pavers			18-30

Figure 40. Green Roof Materials and Respective Weights

For a description of the materials and their uses as well as section drawings relating the materials, please refer to the Architectural section of this paper. In addition, sample product data sheets are available in Appendix A.

The section of the roof under consideration (pavers, gravel, or landscape) will determine the structural load and will change based on the materials in that location. Refer to the Architectural section for more details. The dead load will be conservatively estimated to be 100 psf, where the landscape materials will be conservatively taken to be 80 psf with a superimposed load of 20 psf that was standard in the design of this building.

Snow Drift

Snow drift will be of considerable issue at this part of the building. The 3D model displaying where the drifting will occur is shown in Figure 42. Leeward drift was considered from the 5-story wing onto the atrium. This drift was found to be 50.66 psf at the location noted in Figure 42. Windward drift was also considered over the radial wing and atrium at the junction of the 5-story wing. This load was found to be 51 psf and therefore controls. For simplicity, the atrium was modeled such that the highest windward snow drift loading exists over the entire area of the atrium. It should be noted that the actual snow drift load exists over a length of 4*hd = 4*2.85 = 11.2 feet from the edge point of the atrium (where the five story rectangular wing begins to rise above this plane). Also, the actual drift loading exists as a trapezoidal load with the flat roof snow load of 21 PSF (modeled as 25 PSF) as a rectangular distributed load while the snow drift load is a triangular distributed load which starts as a maximum of 51 PSF (modeled as 50 PSF such that the total snow load is 75 PSF) at the junction of the rectangular wing and the atrium plane and diminishes to zero at a point 11.2 feet from this junction. Snow drifts 1.6 ft at the mechanical units which equates to 28.6 PSF which extends for 6 feet around the mechanical units. Snow drifts 2.3 feet around the skylight in the atrium which equates to 41 PSF which extends 9.2 feet from the skylight. In addition, IBC 2006 dictates a required live load of 20 psf on landscaped roofs. Therefore, the total live load on the atrium will be 95 psf and for the radial wing will be 45 psf. The following Figure 43 represents the roof structural framing for the green roof. The following table summarizes this loading scenario.

Green Roof Loading			
Description	Type	Location	Load
Green Roof Weight	Dead	Full	80 PSF
Superimposed Dead Load	Dead	Full	20 PSF
Flat Roof Snow Load	Snow	Full	21 PSF
Drift Snow Load	Snow	Atrium	51 PSF
Live Load	Live	Full	20 PSF

Figure 41. Green Roof Loading Scenarios

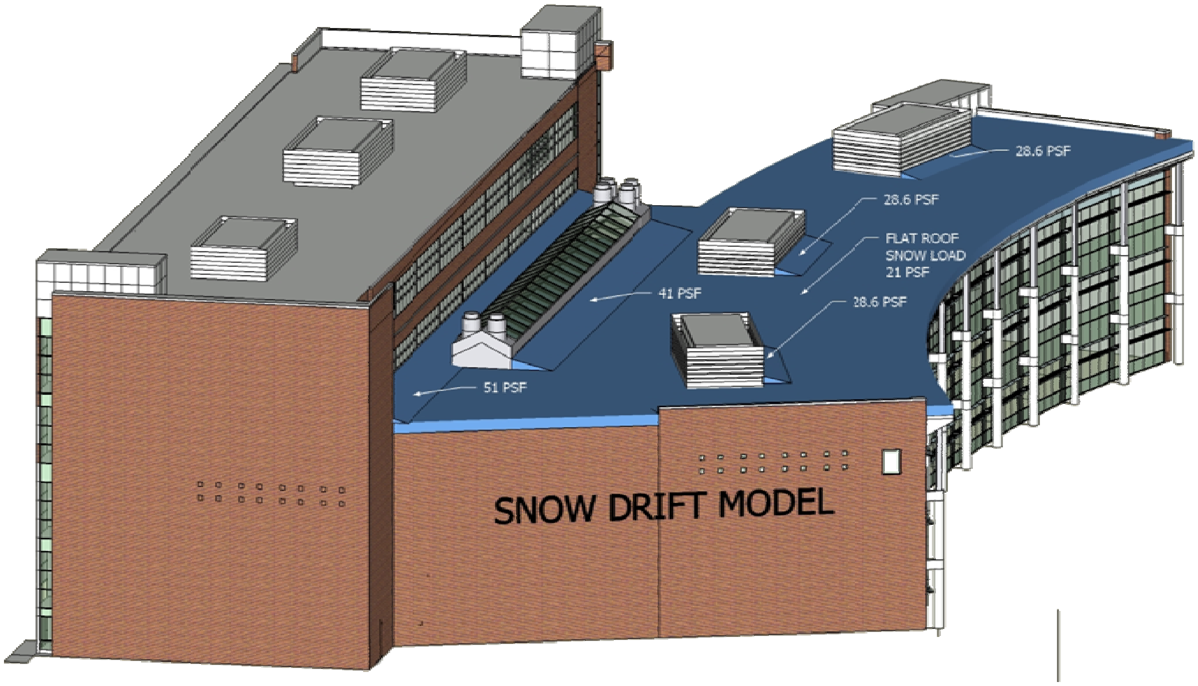


Figure 42. Snow Drift Loading on 3D Model

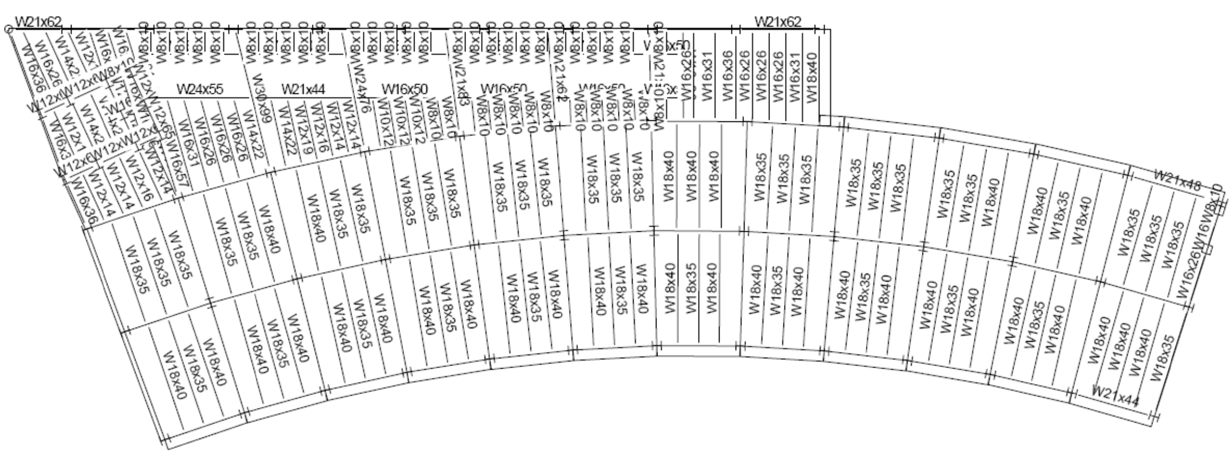


Figure 43. Green Roof Framing

While the original roof framing in the radial wing consisted of open web steel joists of size 20KCS2, the new design shows steel beams in the size range of W18x35 – W18x40. The atrium framing remains as steel beams just as it was in the original design. Because of the short spans in some cases, beams range in size from W8x10 to W16x31. It should be noted that in the original design, the beam size was governed by the size specification of the architect. The architect originally specified W16x36 beams in this area. This beam size is much more adequate than the design requires. The reason for this size of beam is because these beams are fully exposed structural steel. Therefore, the beams also serve as an architectural component. In addition, the architect wanted to visually expose beams at 4' on center. Therefore, the beams in the redesign were kept at 4' on center as well. It is shown through the analysis that these beams are also adequate with the additional snow and green roof dead and live load. See Appendix A for a sample calculation.

Summary and Conclusions

The redesign of the structure in the rectangular wing of the building was a success. The system depth was decreased from about 28" to typically 14"-16" for the floor and roof beams to 21" the girders in those areas. Columns were reduced in size by approximately 35%. Lateral forces logically increased because of the additional two stories. These additional forces added extra strain to the lateral resisting members. The reduction in the number of frames also concentrated greater forces in each frame. Thusly, the frames were drastically increased in size due to the lateral drift experienced on the north-south frames. Braced frames would have been a good solution provided that the architect was willing to work with the architectural floor plan to create more uniformity between the floor plans. Since this is not the case and the architect required a very open plan, moment frames are still a good solution. However, based on the results, the number of original moment frames in the north-south direction (originally 11 frames) now seems appropriate. Contrarily, the frames in the east-west direction, from analysis, suggests that the number of frames should be reduced from the original number of three frames and the one frame in the redesign is adequate to resist the forces without dramatically increasing the frame member sizes too drastically. The foundation increases from the original size by about 22-28% and was logical due to the increased moment caused and the greater axial load from the additional two floors. The green roof structure would not need to be increased dramatically since the architect previously specified certain sections that he wished to see in the atrium. For the most part, the already existing structure can handle the additional load from the green roof.

Construction Management

Cost and Schedule Analysis

Cost analysis of the building was done through a comparison of the redesign to the existing design of the building. Both designs consist of steel construction. The difference occurs in the type of steel roof and floor framing. The existing floor structure is comprised of non-composite steel joists with non-composite steel deck and floor slab with steel beam lateral moment frames. The redesign consists of composite steel beam (W-shapes) framing with composite steel deck and floor slab. The slab thickness is identical to the existing structure.

Therefore, cost analysis will be done with a comparison of the cost of the old structure to the new structure. RSMeans 2008 will be referenced for cost data. Since the building is currently under construction, a time factor will not be necessary.

The takeoff of the open web steel joists, steel beams, and columns is shown in Appendix A. According to RSMeans, K-Series open web steel joists cost $1500 per ton of steel. The construction requires a crew of "E-7." This crew can output 12 tons of steel joists per day and it takes one laborer 6.67 hours to construct one ton of joists. Based upon the data in RSMeans, it was found that one pound of steel material costs $1.21. This cost was used to find the total material cost for the steel W-shapes. The labor and equipment costs were found in RSMeans for each shape that is specified.

The tables below (Table xx-Table xx) show the breakdown of the cost and scheduling information. The units are given in total units, units per square foot, and units per floor. This allows a more relative comparison of the two designs considering that the re-design is larger than the existing design. As the table shows (Figure 44), the re-design costs approximately $1,200 more per floors. The total construction process will take approximately 4 less days to complete.

American Hydrotech was used as a primary resource for all of the garden roof specifications. According to the sales department, the cost of the entire green roof will cost $22-25 per square foot. This includes 6 inches of soil which is specified in the design. The total area of the proposed green roof is 30,000 square feet. Therefore, the total cost of this part of the project is $660,000 to $750,000.

Construction Management Summary					
	Total	Building SF	Per SF	Floors	Per Floor
Existing Cost	$ 779,349.10	56655	$ 13.76	3	$ 259,783.03
Re-Design Cost	$ 1,302,722.00	94425	$ 13.80	5	$ 260,544.40
Scheduled Days for Existing Design	24.27 days			3	8.09 days
Scheduled Days for Re-design	20.17 days			5	4.034 days

Figure 44. Cost Comparison of Existing and Re-design

Overall Conclusions

The overall proposal proved to be a success. The architectural features of the building provide the additional space requirements that the county may need for the future. The addition of the two stories in the one phase will save time and money should the county need the space. From research of the growing school system and the initial indication from the architect and owner that this was a possibility, it seems logical to go ahead with the proposal. The green roof provides a great aesthetic appeal with potential energy savings. The structure only increased slightly in certain areas (moment frames and foundations) due to the higher load from the additional stories and the additional lateral forces. Since the cost of the structure increases by a very little amount per floor ($0.04 per square foot), no definitive conclusion can be made based on cost alone. However, scheduling seems to go along faster and, therefore, the new system was a good alternative.

Re-Design Cost Analysis

Member Class	Member Size	No. Items	Total Length	Total Weight	Quanitity of Units	Cost Per Pound	Material (RSMEANS)	Labor (Per Unit)	Equip (Per Unit)	Total Labor Cost	Total Equip Cost	Per LF Total (Per LF)	Material Cost	Total Cost
Column	HSS10.000X0.250	1	76.7	1011	1	$ 1.21		$ 49.00	$ 32.50	$ 3,758.30	$ 2,492.75	$ 81.50	$ 1,223.31	$ 7,474.36
Column	HSS10.000X0.625	7	76.7	4479	7	$ 1.21		$ 49.00	$ 32.50	$ 3,758.30	$ 2,492.75	$ 81.50	$ 5,419.59	$ 11,670.64
Column	HSS12X12X3/8	2	61.3	3339	2	$ 1.21		$ 49.00	$ 32.50	$ 3,003.70	$ 1,992.25	$ 81.50	$ 4,040.19	$ 9,036.14
Column	HSS12X12X3/8	1	15.3	835	1	$ 1.21		$ 49.00	$ 32.50	$ 749.70	$ 497.25	$ 81.50	$ 1,010.35	$ 2,257.30
Column	HSS14.000X0.375	2	76.7	3913	2	$ 1.21		$ 52.00	$ 35.00	$ 3,988.40	$ 2,684.50	$ 87.00	$ 4,734.73	$ 11,407.63
NS Moment	HSS18X0.375	2	76.7	5.1	2	$ 1.21		$ 52.00	$ 35.00	$ 3,988.40	$ 2,684.50	$ 87.00	$ 37.03	$ 6,709.93
Column	HSS6X6X3/16	2	76.7	1038	2	$ 1.21		$ 43.50	$ 29.00	$ 3,336.45	$ 2,224.30	$ 72.50	$ 1,255.98	$ 6,816.73
Column	HSS8X8X1/4	1	30.7	741	1	$ 1.21		$ 47.00	$ 31.50	$ 1,442.90	$ 967.05	$ 78.50	$ 896.61	$ 3,306.56
Column	HSS8X8X3/8	1	46	1628	1	$ 1.21		$ 47.00	$ 31.50	$ 2,162.00	$ 1,449.00	$ 78.50	$ 1,969.88	$ 5,580.88
Beam	W10X12	7	118.2	1418.2	118.2	$ 1.21	$ 14.50	$ 3.91	$ 2.61	$ 462.08	$ 308.45	$ 21.02	$ 1,715.97	$ 2,486.51
Column	W10X33	4	138	4554	138	$ 1.21	$ 40.00	$ 4.26	$ 2.85	$ 587.88	$ 393.30	$ 47.11	$ 5,510.34	$ 6,491.52
EW Moment	W10X33	3	73.3	2418.9	73.3	$ 1.21	$ 40.00	$ 4.26	$ 2.85	$ 312.26	$ 208.91	$ 47.11	$ 2,926.87	$ 3,448.03
EW Moment	W10X39	2	48.8	1903.2	48.8	$ 1.21	$ 40.00	$ 4.26	$ 2.85	$ 207.89	$ 139.08	$ 47.11	$ 2,302.87	$ 2,649.84
Column	W10X49	2	92	4508	92	$ 1.21	$ 59.50	$ 4.26	$ 2.85	$ 391.92	$ 262.20	$ 66.61	$ 5,454.68	$ 6,108.80
Beam	W12X16	1	30.92	494.72	30.92	$ 1.21	$ 16.95	$ 2.66	$ 1.78	$ 82.25	$ 55.04	$ 21.39	$ 598.61	$ 735.90
Beam	W12X19	82	2436	46275	2436	$ 1.21	$ 16.95	$ 2.66	$ 1.78	$ 6,478.43	$ 4,335.19	$ 21.39	$ 55,992.15	$ 66,805.77
Beam	W12X22	8	69.21	1522.6	69.21	$ 1.21	$ 26.50	$ 2.66	$ 1.78	$ 184.10	$ 123.19	$ 30.94	$ 1,842.37	$ 2,149.66
EW Moment	W12X30	4	97.7	2931	97.7	$ 1.21	$ 31.50	$ 2.66	$ 1.78	$ 259.88	$ 173.91	$ 35.94	$ 3,546.51	$ 3,980.30
EW Moment	W12X58	2	48.8	2830.4	48.8	$ 1.21	$ 60.50	$ 3.13	$ 2.09	$ 152.74	$ 101.99	$ 65.72	$ 3,424.78	$ 3,679.52
EW Moment	W14X109	10	306.7	33430	306.7	$ 1.21	$ 145.00	$ 3.26	$ 2.18	$ 999.84	$ 668.61	$ 150.44	$ 40,450.66	$ 42,119.11
Column	W14X120	4	184	22080	184	$ 1.21	$ 145.00	$ 3.26	$ 2.18	$ 599.84	$ 401.12	$ 150.44	$ 26,716.80	$ 27,717.76
EW Moment	W14X120	10	460	55200	460	$ 1.21	$ 145.00	$ 3.26	$ 2.18	$ 1,499.60	$ 1,002.80	$ 150.44	$ 66,792.00	$ 69,294.40
Column	W14X132	2	92	12144	92	$ 1.21	$ 145.00	$ 3.26	$ 2.18	$ 299.92	$ 200.56	$ 150.44	$ 14,694.24	$ 15,194.72
NS Moment	W14X132	2	92	12144	92	$ 1.21	$ 145.00	$ 3.26	$ 2.18	$ 299.92	$ 200.56	$ 150.44	$ 88,165.44	$ 88,665.92
NS Moment	W14X145	1	30.7	4451.5	30.7	$ 1.21	$ 145.00	$ 3.26	$ 2.18	$ 100.08	$ 66.93	$ 150.44	$ 32,317.89	$ 32,484.90
NS Moment	W14X176	1	30.7	5403.2	30.7	$ 1.21	$ 145.00	$ 3.26	$ 2.18	$ 100.08	$ 66.93	$ 150.44	$ 39,227.23	$ 39,394.24
Beam	W14X22	203	6008	132182	6008	$ 1.21	$ 31.50	$ 2.37	$ 1.58	$ 14,239.55	$ 9,493.04	$ 35.45	$ 159,939.62	$ 183,672.20
Beam	W14X38	4	123.7	4699.5	123.7	$ 1.21	$ 41.00	$ 2.89	$ 1.93	$ 357.41	$ 238.68	$ 45.82	$ 5,686.35	$ 6,282.44
Column	W14X43	8	291.3	12526	291.3	$ 1.21	$ 52.00	$ 2.89	$ 1.93	$ 841.86	$ 562.21	$ 56.82	$ 15,156.34	$ 16,560.41
NS Moment	W14X43	4	125	5375	125	$ 1.21	$ 52.00	$ 2.89	$ 1.93	$ 361.25	$ 241.25	$ 56.82	$ 39,022.50	$ 39,625.00
Column	W14X53	6	184	9752	184	$ 1.21	$ 64.00	$ 2.93	$ 1.96	$ 539.12	$ 360.64	$ 68.89	$ 11,799.92	$ 12,699.68
EW Moment	W14X53	6	146.5	7764.5	146.5	$ 1.21	$ 64.00	$ 2.93	$ 1.96	$ 429.25	$ 287.14	$ 68.89	$ 9,395.05	$ 10,111.43
Column	W14X61	1	30.7	1872.7	30.7	$ 1.21	$ 64.00	$ 2.93	$ 1.96	$ 89.95	$ 60.17	$ 68.89	$ 2,265.97	$ 2,416.09
Column	W14X82	3	122.7	10061	122.7	$ 1.21	$ 89.50	$ 3.08	$ 2.06	$ 377.92	$ 252.76	$ 94.64	$ 12,174.29	$ 12,804.97
Column	W14X90	2	76.7	6903	76.7	$ 1.21	$ 145.00	$ 3.26	$ 2.18	$ 250.04	$ 167.21	$ 150.44	$ 8,352.63	$ 8,769.88
EW Moment	W14X90	4	153.3	13797	153.3	$ 1.21	$ 145.00	$ 3.26	$ 2.18	$ 499.76	$ 334.19	$ 150.44	$ 16,694.37	$ 17,528.32
Beam	W16X26	37	890	23139	890	$ 1.21	$ 31.50	$ 2.34	$ 1.57	$ 2,082.51	$ 1,397.24	$ 35.41	$ 27,998.14	$ 31,477.89
Beam	W16X31	57	1397	43317	1397	$ 1.21	$ 37.50	$ 2.60	$ 1.74	$ 3,633.06	$ 2,431.35	$ 41.84	$ 52,413.85	$ 58,478.26
Beam	W16X36	6	114.6	4127	114.6	$ 1.21	$ 37.50	$ 2.60	$ 1.74	$ 298.06	$ 199.47	$ 41.84	$ 4,993.72	$ 5,491.26
NS Moment	W18X106	3	93.5	9911	93.5	$ 1.21	$ 128.00	$ 3.77	$ 1.87	$ 352.50	$ 174.85	$ 133.64	$ 71,953.86	$ 72,481.20
NS Moment	W18X119	1	31.5	3748.5	31.5	$ 1.21	$ 128.00	$ 3.77	$ 1.89	$ 118.76	$ 59.54	$ 133.66	$ 27,214.11	$ 27,392.40
Beam	W18X35	9	252.9	8852.2	252.9	$ 1.21	$ 42.50	$ 3.53	$ 1.77	$ 892.81	$ 447.67	$ 47.80	$ 10,711.16	$ 12,051.64
Beam	W18X40	4	97.67	3906.8	97.67	$ 1.21	$ 48.50	$ 3.53	$ 1.77	$ 344.78	$ 172.88	$ 53.80	$ 4,727.23	$ 5,244.88
Beam	W18X50	1	24	1200	24	$ 1.21	$ 60.50	$ 3.72	$ 1.86	$ 89.28	$ 44.64	$ 66.08	$ 1,452.00	$ 1,585.92
EW Moment	W18X50	15	366.3	18315	366.3	$ 1.21	$ 66.50	$ 3.72	$ 1.86	$ 1,362.64	$ 681.32	$ 72.08	$ 22,161.15	$ 24,205.10
EW Moment	W18X55	18	439.5	24173	439.5	$ 1.21	$ 78.50	$ 3.77	$ 1.89	$ 1,656.92	$ 830.66	$ 84.16	$ 29,248.73	$ 31,736.30
EW Moment	W18X60	3	73.3	4398	73.3	$ 1.21	$ 78.50	$ 3.77	$ 1.89	$ 276.34	$ 138.54	$ 84.16	$ 5,321.58	$ 5,736.46
NS Moment	W18X97	2	62.5	6062.5	62.5	$ 1.21	$ 104.00	$ 3.77	$ 1.89	$ 235.63	$ 118.13	$ 109.66	$ 44,013.75	$ 44,367.50
Beam	W21X44	24	756	33264	756	$ 1.21	$ 53.00	$ 3.19	$ 1.60	$ 2,411.64	$ 1,209.60	$ 57.79	$ 40,249.44	$ 43,870.68
Beam	W24X55	4	126	6930	126	$ 1.21	$ 66.50	$ 3.06	$ 1.53	$ 385.56	$ 192.78	$ 71.09	$ 8,385.30	$ 8,963.64
Beam	W8X10	76	908.8	9088.2	908.8	$ 1.21	$ 24.00	$ 3.91	$ 2.61	$ 3,553.49	$ 2,372.02	$ 30.52	$ 10,996.72	$ 16,922.23
EW Moment	W8X31	2	48.8	1512.8	48.8	$ 1.21	$ 37.50	$ 4.23	$ 2.85	$ 206.23	$ 139.08	$ 44.58	$ 1,830.49	$ 2,175.80
													$ 1,056,425.34	$ 1,180,318.61
Cost Equivalent of 10116 shear studs at 10# of steel weight per stud														
Shear Stud		10116				$ 12.10								$ 122,403.60
													TOTAL	$ 1,302,722.21

Figure 45. Re-Design Cost Analysis

Re-Design Scheduling Analysis												
Member Class	Member Size	No. Items	Total Length	Total Weight	Crew	Output	Labor Hours	Unit	Quanitity of Units	No. of Groups	Labor Hours	Daily Productivity
Column	HSS10.000X0.250	1	76.7	1011	E-2	48	1.167	Each	1	1	1.167	0.021
Column	HSS10.000X0.625	7	76.7	4479	E-2	48	1.167	Each	7	1	8.169	0.146
Column	HSS12X12X3/8	2	61.3	3339	E-2	48	1.167	Each	2	1	2.334	0.042
Column	HSS12X12X3/8	1	15.3	835	E-2	48	1.167	Each	1	1	1.167	0.021
Column	HSS14.000X0.375	2	76.7	3913	E-2	45	1.244	Each	2	1	2.488	0.044
NS Moment	HSS18X0.375	2	76.7	5.1	E-2	45	1.244	Each	2	6	2.488	0.044
Column	HSS6X6X3/16	2	76.7	1038	E-2	54	1.037	Each	2	1	2.074	0.037
Column	HSS8X8X1/4	1	30.7	741	E-2	50	1.12	Each	1	1	1.120	0.020
Column	HSS8X8X3/8	1	46	1628	E-2	50	1.12	Each	1	1	1.120	0.020
Beam	W10X12	7	118.2	1418.2	E-2	660	0.093	LF	118.2	1	10.991	0.179
Column	W10X33	4	138	4554	E-2	550	0.102	LF	138	1	14.076	0.251
EW Moment	W10X33	3	73.3	2418.9	E-2	550	0.102	LF	73.3	1	7.477	0.133
EW Moment	W10X39	2	48.8	1903.2	E-2	550	0.102	LF	48.8	1	4.978	0.089
Column	W10X49	2	92	4508	E-2	550	0.102	LF	92	1	9.384	0.167
Beam	W12X16	1	30.92	494.72	E-2	880	0.064	LF	30.92	1	1.979	0.035
Beam	W12X19	82	2436	46275	E-2	880	0.064	LF	2436	1	155.872	2.768
Beam	W12X22	8	69.21	1522.6	E-2	880	0.064	LF	69.21	1	4.429	0.079
EW Moment	W12X30	4	97.7	2931	E-2	880	0.064	LF	97.7	1	6.253	0.111
EW Moment	W12X58	2	48.8	2830.4	E-2	750	0.075	LF	48.8	1	3.660	0.065
EW Moment	W14X109	10	306.7	33430	E-2	720	0.078	LF	306.7	1	23.923	0.426
Column	W14X120	4	184	22080	E-2	720	0.078	LF	184	1	14.352	0.256
EW Moment	W14X120	10	460	55200	E-2	720	0.078	LF	460	1	35.880	0.639
Column	W14X132	2	92	12144	E-2	720	0.078	LF	92	1	7.176	0.128
NS Moment	W14X132	2	92	12144	E-2	720	0.078	LF	92	6	7.176	0.128
NS Moment	W14X145	1	30.7	4451.5	E-2	720	0.078	LF	30.7	6	2.395	0.043
NS Moment	W14X176	1	30.7	5403.2	E-2	720	0.078	LF	30.7	6	2.395	0.043
Beam	W14X22	203	6008	132182	E-2	990	0.057	LF	6008	1	342.470	6.069

Figure 46. Scheduling Analysis of the Re-design

Re-Design Scheduling Analysis												
Member Class	Member Size	No. Items	Total Length	Total Weight	Crew	Output	Labor Hours	Unit	Quanitity of Units	No. of Groups	Labor Hours	Daily Productivity
Beam	W14X38	4	123.7	4699.5	E-2	810	0.069	LF	123.7	1	8.533	0.153
Column	W14X43	8	291.3	12526	E-2	810	0.069	LF	291.3	1	20.100	0.360
NS Moment	W14X43	4	125	5375	E-2	810	0.069	LF	125	6	8.625	0.154
Column	W14X53	6	184	9752	E-2	800	0.07	LF	184	1	12.880	0.230
EW Moment	W14X53	6	146.5	7764.5	E-2	800	0.07	LF	146.5	1	10.255	0.183
Column	W14X61	1	30.7	1872.7	E-2	800	0.07	LF	30.7	1	2.149	0.038
Column	W14X82	3	122.7	10061	E-2	760	0.074	LF	122.7	1	9.080	0.161
Column	W14X90	2	76.7	6903	E-2	720	0.078	LF	76.7	1	5.983	0.107
EW Moment	W14X90	4	153.3	13797	E-2	720	0.078	LF	153.3	1	11.957	0.213
Beam	W16X26	37	890	23139	E-2	1000	0.056	LF	890	1	49.838	0.890
Beam	W16X31	57	1397	43317	E-2	900	0.062	LF	1397	1	86.634	1.553
Beam	W16X36	6	114.6	4127	E-2	900	0.062	LF	114.6	1	7.108	0.127
NS Moment	W18X106	3	93.5	9911	E-2	900	0.089	LF	93.5	6	8.322	0.104
NS Moment	W18X119	1	31.5	3748.5	E-2	900	0.089	LF	31.5	6	2.804	0.035
Beam	W18X35	9	252.9	8852.2	E-5	960	0.083	LF	252.9	1	20.992	0.263
Beam	W18X40	4	97.67	3906.8	E-5	960	0.083	LF	97.67	1	8.107	0.102
Beam	W18X50	1	24	1200	E-5	912	0.088	LF	24	1	2.112	0.026
EW Moment	W18X50	15	366.3	18315	E-5	912	0.088	LF	366.3	1	32.234	0.402
EW Moment	W18X55	18	439.5	24173	E-5	900	0.089	LF	439.5	1	39.116	0.488
EW Moment	W18X60	3	73.3	4398	E-5	900	0.089	LF	73.3	1	6.524	0.081
NS Moment	W18X97	2	62.5	6062.5	E-5	900	0.089	LF	62.5	6	5.563	0.069
Beam	W21X44	24	756	33264	E-5	1064	0.075	LF	756	1	56.700	0.711
Beam	W24X55	4	126	6930	E-5	1110	0.072	LF	126	1	9.072	0.114
Beam	W8X10	76	908.8	9088.2	E-2	600	0.093	LF	908.8	1	84.520	1.515
EW Moment	W8X31	2	48.8	1512.8	E-2	550	0.102	LF	48.8	1	4.978	0.089
											1191.175	
											148.897	
											18.612	20.170

Figure 47. Scheduling Analysis of the Re-design

Existing Design Cost Analysis														
										Total	Total	Per LF		
Member Class	Member Size	No. Items	Total Length	Total Weight	Quanitity of Units	Cost Per Pound	Material (RSMEANS)	Labor (Per Unit)	Equip (Per Unit)	Total Labor Cost	Total Equip Cost	Total (Per LF)	Material Cost	Total Cost
Joist	10K1	8	61.13	306	0.153		$ 1,500.00	$ 283.00	$ 153.00	$ 43.30	$ 23.41	$ 1,936.00	$ 229.50	$ 296.21
Joist	10K1	4	17.67	88	0.044		$ 1,500.00	$ 283.00	$ 153.00	$ 12.45	$ 6.73	$ 1,936.00	$ 66.00	$ 85.18
Joist	10K1	4	17.67	88	0.044		$ 1,500.00	$ 283.00	$ 153.00	$ 12.45	$ 6.73	$ 1,936.00	$ 66.00	$ 85.18
Joist	10K1	16	96.46	482	0.241		$ 1,500.00	$ 283.00	$ 153.00	$ 68.20	$ 36.87	$ 1,936.00	$ 361.50	$ 466.58
Joist	12K1	7	80.8	404	0.202		$ 1,500.00	$ 283.00	$ 153.00	$ 57.17	$ 30.91	$ 1,936.00	$ 303.00	$ 391.07
Joist	12K1	7	80.8	404	0.202		$ 1,500.00	$ 283.00	$ 153.00	$ 57.17	$ 30.91	$ 1,936.00	$ 303.00	$ 391.07
Joist	12K1	14	161.6	808	0.404		$ 1,500.00	$ 283.00	$ 153.00	$ 114.33	$ 61.81	$ 1,936.00	$ 606.00	$ 782.14
Joist	12K1SP	1	12.45	62	0.031		$ 1,500.00	$ 283.00	$ 153.00	$ 8.77	$ 4.74	$ 1,936.00	$ 46.50	$ 60.02
Joist	12K1SP	1	12.45	62	0.031		$ 1,500.00	$ 283.00	$ 153.00	$ 8.77	$ 4.74	$ 1,936.00	$ 46.50	$ 60.02
Joist	12K1SP	2	24.89	124	0.062		$ 1,500.00	$ 283.00	$ 153.00	$ 17.55	$ 9.49	$ 1,936.00	$ 93.00	$ 120.03
Joist	16K4	10	182.09	1275	0.638		$ 1,500.00	$ 283.00	$ 153.00	$ 180.41	$ 97.54	$ 1,936.00	$ 956.25	$ 1,234.20
Joist	16K4	10	182.09	1275	0.638		$ 1,500.00	$ 283.00	$ 153.00	$ 180.41	$ 97.54	$ 1,936.00	$ 956.25	$ 1,234.20
Joist	16K4	20	364.18	2549	1.275		$ 1,500.00	$ 283.00	$ 153.00	$ 360.68	$ 195.00	$ 1,936.00	$ 1,911.75	$ 2,467.43
Joist	26K9	1	30.92	377	0.189		$ 1,500.00	$ 283.00	$ 153.00	$ 53.35	$ 28.84	$ 1,936.00	$ 282.75	$ 364.94
Joist	26K9	1	30.92	377	0.189		$ 1,500.00	$ 283.00	$ 153.00	$ 53.35	$ 28.84	$ 1,936.00	$ 282.75	$ 364.94
Joist	28K8	49	1453.2	18455	9.228		$ 1,500.00	$ 283.00	$ 153.00	$ 2,611.38	$ 1,411.81	$ 1,936.00	$ 13,841.25	$ 17,864.44
Joist	28K8	49	1453.2	18455	9.228		$ 1,500.00	$ 283.00	$ 153.00	$ 2,611.38	$ 1,411.81	$ 1,936.00	$ 13,841.25	$ 17,864.44
Joist	28K8	98	2906.3	36910	18.455		$ 1,500.00	$ 283.00	$ 153.00	$ 5,222.77	$ 2,823.62	$ 1,936.00	$ 27,682.50	$ 35,728.88
Joist	28K9	51	1576.8	20498	10.249		$ 1,500.00	$ 283.00	$ 153.00	$ 2,900.47	$ 1,568.10	$ 1,936.00	$ 15,373.50	$ 19,842.06
Joist	28K9	51	1576.8	20498	10.249		$ 1,500.00	$ 283.00	$ 153.00	$ 2,900.47	$ 1,568.10	$ 1,936.00	$ 15,373.50	$ 19,842.06
Joist	28K9	102	3153.5	40996	20.498		$ 1,500.00	$ 283.00	$ 153.00	$ 5,800.93	$ 3,136.19	$ 1,936.00	$ 30,747.00	$ 39,684.13
Joist	28K9	119	3714.7	48290.7	24.145		$ 1,500.00	$ 283.00	$ 153.00	$ 6,833.14	$ 3,694.24	$ 1,936.00	$ 36,218.03	$ 46,745.41
Joist	28K9	8	244.81	3182.53	1.591		$ 1,500.00	$ 283.00	$ 153.00	$ 450.33	$ 243.46	$ 1,936.00	$ 2,386.90	$ 3,080.69
Joist	28K9	10	305.39	3970.07	1.985		$ 1,500.00	$ 283.00	$ 153.00	$ 561.76	$ 303.71	$ 1,936.00	$ 2,977.55	$ 3,843.03
Joist	28K9	137	4265	55445	27.723		$ 1,500.00	$ 283.00	$ 153.00	$ 7,845.47	$ 4,241.54	$ 1,936.00	$ 41,583.75	$ 53,670.76
Column	HSS12.75x0.375	10	46	21160	10.000	$ 1.21	$ 1,200.00	$ 49.00	$ 32.50	$ 490.00	$ 325.00	$ 1,281.50	$ 25,603.60	$ 26,418.60
Column	HSS6x6x0.188	1	46	623	46.000	$ 1.21	$ 297.00	$ 43.50	$ 29.00	$ 2,001.00	$ 1,334.00	$ 369.50	$ 753.83	$ 4,088.83
Column	HSS6x6x1/4	1	46	820	46.000	$ 1.21	$ 297.00	$ 43.50	$ 29.00	$ 2,001.00	$ 1,334.00	$ 369.50	$ 992.20	$ 4,327.20
Column	HSS8.625x0.188	1	46	723	46.000	$ 1.21	$ 645.00	$ 47.00	$ 31.50	$ 2,162.00	$ 1,449.00	$ 723.50	$ 874.83	$ 4,485.83
Column	HSS8x8x1/4	1	46	1111	46.000	$ 1.21	$ 645.00	$ 47.00	$ 31.50	$ 2,162.00	$ 1,449.00	$ 723.50	$ 1,344.31	$ 4,955.31
Beam	W10x33	1	46	1518	46.000	$ 1.21	$ 40.00	$ 4.26	$ 2.85	$ 195.96	$ 131.10	$ 47.11	$ 1,836.78	$ 2,163.84
Beam	W12X19	82	2435.5	46274.5	2435.500	$ 1.21	$ 16.95	$ 2.66	$ 1.78	$ 6,478.43	$ 4,335.19	$ 21.39	$ 55,992.15	$ 66,805.77
Beam	W12X19	82	2435.5	46274.5	2435.500	$ 1.21	$ 16.95	$ 2.66	$ 1.78	$ 6,478.43	$ 4,335.19	$ 21.39	$ 55,992.15	$ 66,805.77
Beam	W12X22	8	69.21	1522.62	69.210	$ 1.21	$ 26.50	$ 2.66	$ 1.78	$ 184.10	$ 123.19	$ 30.94	$ 1,842.37	$ 2,149.66
Beam	W12X22	8	69.21	1522.62	69.210	$ 1.21	$ 26.50	$ 2.66	$ 1.78	$ 184.10	$ 123.19	$ 30.94	$ 1,842.37	$ 2,149.66
Beam	W12X22	8	69.21	1522.62	69.210	$ 1.21	$ 26.50	$ 2.66	$ 1.78	$ 184.10	$ 123.19	$ 30.94	$ 1,842.37	$ 2,149.66
Column	W14x43	1	230	9890	230.000	$ 1.21	$ 41.00	$ 2.89	$ 1.93	$ 664.70	$ 443.90	$ 45.82	$ 11,966.90	$ 13,075.50
Column	W14x68	10	46	31280	46.000	$ 1.21	$ 89.50	$ 3.08	$ 2.06	$ 141.68	$ 94.76	$ 94.64	$ 37,848.80	$ 38,085.24
Column	W14x82	1	92	7544	92.000	$ 1.21	$ 109.00	$ 3.17	$ 2.12	$ 291.64	$ 195.04	$ 114.29	$ 9,128.24	$ 9,614.92
Beam	W14x82	10	46	37720	46.000	$ 1.21	$ 109.00	$ 3.17	$ 2.12	$ 145.82	$ 97.52	$ 114.29	$ 45,641.20	$ 45,884.54
Beam	W16X26	3	43.73	1143	43.730	$ 1.21	$ 31.50	$ 2.34	$ 1.57	$ 102.33	$ 68.66	$ 35.41	$ 1,383.03	$ 1,554.01
Beam	W16X26	3	43.13	1127	43.130	$ 1.21	$ 31.50	$ 2.34	$ 1.57	$ 100.92	$ 67.71	$ 35.41	$ 1,363.67	$ 1,532.31
Beam	W16X26	3	43.13	1127	43.130	$ 1.21	$ 31.50	$ 2.34	$ 1.57	$ 100.92	$ 67.71	$ 35.41	$ 1,363.67	$ 1,532.31
Beam	W16X26	9	129.98	3397	129.980	$ 1.21	$ 31.50	$ 2.34	$ 1.57	$ 304.15	$ 204.07	$ 35.41	$ 4,110.37	$ 4,618.59
Beam	W16X36	2	34.43	1242	34.430	$ 1.21	$ 37.50	$ 2.60	$ 1.74	$ 89.52	$ 59.91	$ 41.84	$ 1,502.82	$ 1,652.25
Beam	W16X36	5	83.23	3002	83.230	$ 1.21	$ 37.50	$ 2.60	$ 1.74	$ 216.40	$ 144.82	$ 41.84	$ 3,632.42	$ 3,993.64
Beam	W16X36	2	34.43	1242	34.430	$ 1.21	$ 37.50	$ 2.60	$ 1.74	$ 89.52	$ 59.91	$ 41.84	$ 1,502.82	$ 1,652.25

Figure 48. Cost Analysis of the Existing Design

Existing Design Cost Analysis														
										Total	Total	Per LF		
Member Class	Member Size	No. Items	Total Length	Total Weight	Quanitity of Units	Cost Per Pound	Material (RSMEANS)	Labor (Per Unit)	Equip (Per Unit)	Total Labor Cost	Total Equip Cost	Total (Per LF)	Material Cost	Total Cost
Beam	W16X36	9	152.08	5486	152.080	$ 1.21	$ 37.50	$ 2.60	$ 1.74	$ 395.41	$ 264.62	$ 41.84	$ 6,638.06	$ 7,298.09
Beam	W18X35	6	160.17	5614	160.170	$ 1.21	$ 42.50	$ 3.53	$ 1.77	$ 565.40	$ 283.50	$ 47.80	$ 6,792.94	$ 7,641.84
Beam	W18X35	6	160.17	5614	160.170	$ 1.21	$ 42.50	$ 3.53	$ 1.77	$ 565.40	$ 283.50	$ 47.80	$ 6,792.94	$ 7,641.84
Beam	W18x35	20	31	21700	31.000	$ 1.21	$ 42.50	$ 3.53	$ 1.77	$ 109.43	$ 54.87	$ 47.80	$ 26,257.00	$ 26,421.30
Beam	W18X40	1	24.42	980	24.420	$ 1.21	$ 48.50	$ 3.53	$ 1.77	$ 86.20	$ 43.22	$ 53.80	$ 1,185.80	$ 1,315.23
Beam	W18X40	1	24.42	980	24.420	$ 1.21	$ 48.50	$ 3.53	$ 1.77	$ 86.20	$ 43.22	$ 53.80	$ 1,185.80	$ 1,315.23
Beam	W18X40	2	48.83	1961	48.830	$ 1.21	$ 48.50	$ 3.53	$ 1.77	$ 172.37	$ 86.43	$ 53.80	$ 2,372.81	$ 2,631.61
Beam	W18X46	1	24.42	1122	24.420	$ 1.21	$ 55.50	$ 3.53	$ 1.77	$ 86.20	$ 43.22	$ 60.80	$ 1,357.62	$ 1,487.05
Beam	W18X46	1	30.92	1420	30.920	$ 1.21	$ 55.50	$ 3.53	$ 1.77	$ 109.15	$ 54.73	$ 60.80	$ 1,718.20	$ 1,882.08
Beam	W18X46	2	55.33	2542	55.330	$ 1.21	$ 55.50	$ 3.53	$ 1.77	$ 195.31	$ 97.93	$ 60.80	$ 3,075.82	$ 3,369.07
Beam	W18x50	10	24	12000	24.000	$ 1.21	$ 60.50	$ 3.72	$ 1.86	$ 89.28	$ 44.64	$ 66.08	$ 14,520.00	$ 14,653.92
Beam	W21x44	11	24	11616	24.000	$ 1.21	$ 53.00	$ 3.19	$ 1.60	$ 76.56	$ 38.40	$ 57.79	$ 14,055.36	$ 14,170.32
Beam	W21X48	2	48.56	2330	48.560	$ 1.21	$ 53.00	$ 3.19	$ 1.60	$ 154.91	$ 77.70	$ 57.79	$ 2,819.30	$ 3,051.90
Beam	W21X48	1	17.65	847	17.650	$ 1.21	$ 53.00	$ 3.19	$ 1.60	$ 56.30	$ 28.24	$ 57.79	$ 1,024.87	$ 1,109.41
Beam	W21X48	3	66.21	3177	66.210	$ 1.21	$ 53.00	$ 3.19	$ 1.60	$ 211.21	$ 105.94	$ 57.79	$ 3,844.17	$ 4,161.32
Beam	W21x55	20	24	26400	24.000	$ 1.21	$ 60.50	$ 3.19	$ 1.60	$ 76.56	$ 38.40	$ 65.29	$ 31,944.00	$ 32,058.96
Beam	W21x57	20	31	35340	31.000	$ 1.21	$ 75.00	$ 3.27	$ 1.64	$ 101.37	$ 50.84	$ 79.91	$ 42,761.40	$ 42,913.61
Beam	W21X62	4	62.38	3884	62.380	$ 1.21	$ 75.00	$ 3.27	$ 1.64	$ 203.98	$ 102.30	$ 79.91	$ 4,699.64	$ 5,005.93
Beam	W21X62	4	62.38	3884	62.380	$ 1.21	$ 75.00	$ 3.27	$ 1.64	$ 203.98	$ 102.30	$ 79.91	$ 4,699.64	$ 5,005.93
Beam	W21x62	10	24	14880	24.000	$ 1.21	$ 75.00	$ 3.06	$ 1.53	$ 73.44	$ 36.72	$ 79.59	$ 18,004.80	$ 18,114.96
Beam	W21X68	4	117.33	7985	117.330	$ 1.21	$ 82.50	$ 3.27	$ 1.64	$ 383.67	$ 192.42	$ 87.41	$ 9,661.85	$ 10,237.94
Beam	W21X68	4	117.33	7985	117.330	$ 1.21	$ 82.50	$ 3.27	$ 1.64	$ 383.67	$ 192.42	$ 87.41	$ 9,661.85	$ 10,237.94
Beam	W21X68	8	234.67	15970	234.670	$ 1.21	$ 82.50	$ 3.27	$ 1.64	$ 767.37	$ 384.86	$ 87.41	$ 19,323.70	$ 20,475.93
Beam	W21x68	18	24	29376	24.000	$ 1.21	$ 82.50	$ 3.27	$ 1.64	$ 78.48	$ 39.36	$ 87.41	$ 35,544.96	$ 35,662.80
Beam	W21x68	20	31	42160	31.000	$ 1.21	$ 82.50	$ 3.27	$ 1.64	$ 101.37	$ 50.84	$ 87.41	$ 51,013.60	$ 51,165.81
Beam	W24x55	20	24	26400	24.000	$ 1.21	$ 66.50	$ 3.06	$ 1.53	$ 73.44	$ 36.72	$ 71.09	$ 31,944.00	$ 32,054.16
Beam	W24X62	2	48.42	3015	48.420	$ 1.21	$ 75.00	$ 3.06	$ 1.53	$ 148.17	$ 74.08	$ 79.59	$ 3,648.15	$ 3,870.40
Beam	W24X62	2	48.42	3015	48.420	$ 1.21	$ 75.00	$ 3.06	$ 1.53	$ 148.17	$ 74.08	$ 79.59	$ 3,648.15	$ 3,870.40
Beam	W24X62	4	96.83	6030	96.830	$ 1.21	$ 75.00	$ 3.06	$ 1.53	$ 296.30	$ 148.15	$ 79.59	$ 7,296.30	$ 7,740.75
Beam	W24X68	1	31.5	2154	31.500	$ 1.21	$ 82.50	$ 3.06	$ 1.53	$ 96.39	$ 48.20	$ 87.09	$ 2,606.34	$ 2,750.93
Beam	W24X68	1	31.5	2154	31.500	$ 1.21	$ 82.50	$ 3.06	$ 1.53	$ 96.39	$ 48.20	$ 87.09	$ 2,606.34	$ 2,750.93
Beam	W24X76	1	31.5	2401	31.500	$ 1.21	$ 92.00	$ 3.06	$ 1.53	$ 96.39	$ 48.20	$ 96.59	$ 2,905.21	$ 3,049.80
Beam	W24X76	1	31.5	2401	31.500	$ 1.21	$ 92.00	$ 3.06	$ 1.53	$ 96.39	$ 48.20	$ 96.59	$ 2,905.21	$ 3,049.80
Beam	W8X10	3	8.63	87	8.630	$ 1.21	$ 12.10	$ 3.91	$ 2.61	$ 33.74	$ 22.52	$ 18.62	$ 105.27	$ 161.54
Beam	W8X10	9	36.84	371	36.840	$ 1.21	$ 12.10	$ 3.91	$ 2.61	$ 144.04	$ 96.15	$ 18.62	$ 448.91	$ 689.11
Beam	W8X10	7	28.84	290	28.840	$ 1.21	$ 12.10	$ 3.91	$ 2.61	$ 112.76	$ 75.27	$ 18.62	$ 350.90	$ 538.94
Beam	W8X10	19	74.3	748	74.300	$ 1.21	$ 12.10	$ 3.91	$ 2.61	$ 290.51	$ 193.92	$ 18.62	$ 905.08	$ 1,389.52
Beam	W8X15	1	12.89	195	12.890	$ 1.21	$ 18.15	$ 3.91	$ 2.61	$ 50.40	$ 33.64	$ 24.67	$ 235.95	$ 319.99
Beam	W8X15		12.89	195	12.890	$ 1.21	$ 18.15	$ 3.91	$ 2.61	$ 50.40	$ 33.64	$ 24.67	$ 235.95	$ 319.99
													$ 673,798.14	$ 779,349.10

Figure 49. Cost Analysis of the Existing Building

Existing Design Scheduling Analysis													
Member Class	Member Size	PLF	No. Items	Total Length	Total Weight	Crew	Output	Labor Hours	Unit	Quanitity of Units	Labor Hours	Daily Productivity
Joist	10K1		8	61.13	306	E-7	12	6.667	Ton	0.153	1.020	0.013
Joist	10K1		4	17.67	88	E-7	12	6.667	Ton	0.044	0.293	0.004
Joist	10K1		4	17.67	88	E-7	12	6.667	Ton	0.044	0.293	0.004
Joist	10K1		16	96.46	482	E-7	12	6.667	Ton	0.241	1.607	0.020
Joist	12K1		7	80.8	404	E-7	12	6.667	Ton	0.202	1.347	0.017
Joist	12K1		7	80.8	404	E-7	12	6.667	Ton	0.202	1.347	0.017
Joist	12K1		14	161.6	808	E-7	12	6.667	Ton	0.404	2.693	0.034
Joist	12K1SP		1	12.45	62	E-7	12	6.667	Ton	0.031	0.207	0.003
Joist	12K1SP		1	12.45	62	E-7	12	6.667	Ton	0.031	0.207	0.003
Joist	12K1SP		2	24.89	124	E-7	12	6.667	Ton	0.062	0.413	0.005
Joist	16K4		10	182.09	1275	E-7	12	6.667	Ton	0.638	4.250	0.053
Joist	16K4		10	182.09	1275	E-7	12	6.667	Ton	0.638	4.250	0.053
Joist	16K4		20	364.18	2549	E-7	12	6.667	Ton	1.275	8.497	0.106
Joist	26K9		1	30.92	377	E-7	12	6.667	Ton	0.189	1.257	0.016
Joist	26K9		1	30.92	377	E-7	12	6.667	Ton	0.189	1.257	0.016
Joist	28K8		49	1453.2	18455	E-7	12	6.667	Ton	9.228	61.520	0.769
Joist	28K8		49	1453.2	18455	E-7	12	6.667	Ton	9.228	61.520	0.769
Joist	28K8		98	2906.3	36910	E-7	12	6.667	Ton	18.455	123.039	1.538
Joist	28K9		51	1576.8	20498	E-7	12	6.667	Ton	10.249	68.330	0.854
Joist	28K9		51	1576.8	20498	E-7	12	6.667	Ton	10.249	68.330	0.854
Joist	28K9		102	3153.5	40996	E-7	12	6.667	Ton	20.498	136.660	1.708
Joist	28K9		119	3714.7	48290.7	E-7	12	6.667	Ton	24.145	160.977	2.012
Joist	28K9		8	244.81	3182.53	E-7	12	6.667	Ton	1.591	10.609	0.133
Joist	28K9		10	305.39	3970.07	E-7	12	6.667	Ton	1.985	13.234	0.165
Joist	28K9		137	4265	55445	E-7	12	6.667	Ton	27.723	184.826	2.310
Column	HSS12.75x0.375		10	46	21160	E-2	48	1.167	Each	10.000	11.670	0.208
Column	HSS6x6x0.188		1	46	623	E-2	54	1.037	Each	46.000	47.702	0.852
Column	HSS6x6x1/4		1	46	820	E-2	54	1.037	Each	46.000	47.702	0.852
Column	HSS8.625x0.188		1	46	723	E-2	50	1.12	Each	46.000	51.520	0.920
Column	HSS8x8x1/4		1	46	1111	E-2	50	1.12	Each	46.000	51.520	0.920
Beam	W10x33		1	46	1518	E-2	550	0.102	Each	46.000	4.692	0.084
Beam	W12X19	19	82	2435.5	46274.5	E-2	880	0.064	LF	2435.500	155.872	2.768
Beam	W12X19	19	82	2435.5	46274.5	E-2	880	0.064	LF	2435.500	155.872	2.768
Beam	W12X22	22	8	69.21	1522.62	E-2	880	0.064	LF	69.210	4.429	0.079
Beam	W12X22	22	8	69.21	1522.62	E-2	880	0.064	LF	69.210	4.429	0.079
Beam	W12X22	22	8	69.21	1522.62	E-2	880	0.064	LF	69.210	4.429	0.079
Column	W14x43	43	1	230	9890	E-2	810	0.069	LF	230.000	15.870	0.284
Column	W14x68	68	10	46	31280	E-2	760	0.074	LF	46.000	3.404	0.061
Column	W14x82	82	1	92	7544	E-2	740	0.076	LF	92.000	6.992	0.124
Beam	W14x82	82	10	46	37720	E-2	740	0.076	LF	46.000	3.496	0.062
Beam	W16X26	26	3	43.73	1143	E-2	1000	0.056	LF	43.730	2.449	0.044

Figure 50. Scheduling Analysis of Existing Design

Existing Design Scheduling Analysis

Member Class	Member Size	PLF	No. Items	Total Length	Total Weight	Crew	Output	Labor Hours	Unit	Quanitity of Units	Labor Hours	Daily Productivity
Beam	W16X26	26	3	43.13	1127	E-2	1000	0.056	LF	43.130	2.415	0.043
Beam	W16X26	26	3	43.13	1127	E-2	1000	0.056	LF	43.130	2.415	0.043
Beam	W16X26	26	9	129.98	3397	E-2	1000	0.056	LF	129.980	7.279	0.130
Beam	W16X36	36	2	34.43	1242	E-2	900	0.062	LF	34.430	2.135	0.038
Beam	W16X36	36	5	83.23	3002	E-2	900	0.062	LF	83.230	5.160	0.092
Beam	W16X36	36	2	34.43	1242	E-2	900	0.062	LF	34.430	2.135	0.038
Beam	W16X36	36	9	152.08	5486	E-2	900	0.062	LF	152.080	9.429	0.169
Beam	W18X35	35	6	160.17	5614	E-5	960	0.083	LF	160.170	13.294	0.167
Beam	W18X35	35	6	160.17	5614	E-5	960	0.083	LF	160.170	13.294	0.167
Beam	W18x35	35	20	31	21700	E-5	960	0.083	LF	31.000	2.573	0.032
Beam	W18X40	40	1	24.42	980	E-5	960	0.083	LF	24.420	2.027	0.025
Beam	W18X40	40	1	24.42	980	E-5	960	0.083	LF	24.420	2.027	0.025
Beam	W18X40	40	2	48.83	1961	E-5	960	0.083	LF	48.830	4.053	0.051
Beam	W18X46	46	1	24.42	1122	E-5	960	0.083	LF	24.420	2.027	0.025
Beam	W18X46	46	1	30.92	1420	E-5	960	0.083	LF	30.920	2.566	0.032
Beam	W18X46	46	2	55.33	2542	E-5	960	0.083	LF	55.330	4.592	0.058
Beam	W18x50	50	10	24	12000	E-5	912	0.088	LF	24.000	2.112	0.026
Beam	W21x44	44	11	24	11616	E-5	1064	0.075	LF	24.000	1.800	0.023
Beam	W21X48	48	2	48.56	2330	E-5	1064	0.075	LF	48.560	3.642	0.046
Beam	W21X48	48	1	17.65	847	E-5	1064	0.075	LF	17.650	1.324	0.017
Beam	W21X48	48	3	66.21	3177	E-5	1064	0.075	LF	66.210	4.966	0.062
Beam	W21x55	55	20	24	26400	E-5	1064	0.075	LF	24.000	1.800	0.023
Beam	W21x57	57	20	31	35340	E-5	1036	0.077	LF	31.000	2.387	0.030
Beam	W21X62	62	4	62.38	3884	E-5	1036	0.077	LF	62.380	4.803	0.060
Beam	W21X62	62	4	62.38	3884	E-5	1036	0.077	LF	62.380	4.803	0.060
Beam	W21x62	62	10	24	14880	E-5	1110	0.072	LF	24.000	1.728	0.022
Beam	W21X68	68	4	117.33	7985	E-5	1036	0.077	LF	117.330	9.034	0.113
Beam	W21X68	68	4	117.33	7985	E-5	1036	0.077	LF	117.330	9.034	0.113
Beam	W21X68	68	8	234.67	15970	E-5	1036	0.077	LF	234.670	18.070	0.227
Beam	W21x68	68	18	24	29376	E-5	1036	0.077	LF	24.000	1.848	0.023
Beam	W21x68	68	20	31	42160	E-5	1036	0.077	LF	31.000	2.387	0.030
Beam	W24x55	55	20	24	26400	E-5	1110	0.72	LF	24.000	17.280	0.022
Beam	W24X62	62	2	48.42	3015	E-5	1110	0.072	LF	48.420	3.486	0.044
Beam	W24X62	62	2	48.42	3015	E-5	1110	0.072	LF	48.420	3.486	0.044
Beam	W24X62	62	4	96.83	6030	E-5	1110	0.072	LF	96.830	6.972	0.087
Beam	W24X68	68	1	31.5	2154	E-5	1110	0.072	LF	31.500	2.268	0.028
Beam	W24X68	68	1	31.5	2154	E-5	1110	0.072	LF	31.500	2.268	0.028
Beam	W24X76	76	1	31.5	2401	E-5	1110	0.072	LF	31.500	2.268	0.028
Beam	W24X76	76	1	31.5	2401	E-5	1110	0.072	LF	31.500	2.268	0.028
Beam	W8X10	10	3	8.63	87	E-2	600	0.093	LF	8.630	0.803	0.014
Beam	W8X10	10	9	36.84	371	E-2	600	0.093	LF	36.840	3.426	0.061
Beam	W8X10	10	7	28.84	290	E-2	600	0.093	LF	28.840	2.682	0.048
Beam	W8X10	10	19	74.3	748	E-2	600	0.093	LF	74.300	6.910	0.124
Beam	W8X15	15	1	12.89	195	E-2	600	0.093	LF	12.890	1.199	0.021
Beam	W8X15	15		12.89	195	E-2	600	0.093	LF	12.890	1.199	0.021
											203.6744	
										Days	3.182412	2.611432

Figure 51. Scheduling Analysis of Existing Design

Appendix A

Table of Contents

List of Figures

I section

Size	#	Length (ft)	Weight (lbs)
W10X33	4	138.0	4559
W14X43	8	291.3	12490
W10X49	2	92.0	4508
W14X53	6	184.0	9766
HSS12X12X3/8	2	61.3	3339
W14X61	1	30.7	1868
W14X82	3	122.7	10017
W14X90	2	76.7	6913
W14X120	4	184.0	22100
W14X132	2	92.0	12145
	34		87704

eel Grade: Other

I section

Size	#	Length (ft)	Weight (lbs)
HSS6X6X3/16	2	76.7	1038
HSS8X8X1/4	1	30.7	741
HSS10.000X0.250	7	214.6	5222
HSS8X8X3/8	1	46.0	1628
HSS12X12X3/8	1	15.3	835
HSS10.000X0.625	7	322.0	18844
	19		28308

Round HS

Size	#	Length (ft)	Weight (lbs)
HSS14.000X0.375	2	76.7	3913
	2		3913

Figure 1 Gravity Column Takeoff

SPAN INFORMATION (ft): I-End (174.38,35.92) J-End (174.38,66.83)

Beam Size (Optimum) = W14X22 Fy = 50.0 ksi

Total Beam Length (ft) = 30.92

COMPOSITE PROPERTIES (Not Shored):

	Left	Right
Concrete thickness (in)	2.50	2.50
Unit weight concrete (pcf)	145.00	145.00
f'c (ksi)	3.00	3.00
Decking Orientation	perpendicular	perpendicular
Decking type	VULCRAFT 1.5VL	VULCRAFT 1.5VL

beff (in)	=	92.75	Y bar(in)	=	14.49
Mnf (kip-ft)	=	274.85	Mn (kip-ft)	=	253.65
C (kips)	=	236.70	PNA (in)	=	13.52
Ieff (in4)	=	616.55	Itr (in4)	=	687.90
Stud length (in)	=	3.50	Stud diam (in)	=	0.75

Stud Capacity (kips) Qn = 15.8

\# of studs: Full = 42 Partial = 31 Actual = 31

Number of Stud Rows = 1 Percent of Full Composite Action = 72.94

USER DEFINED FLANGE BRACE POINTS:

Dist (ft)	Top	Bottom
-1.#J	No	Yes
30.92	No	Yes

LINE LOADS (k/ft):

Load	Dist	DL	CDL	LL	Red%	Type	CLL
1	0.000	0.480	0.000	0.640	---	NonR	0.000
	30.916	0.480	0.000	0.640			0.000
2	0.000	0.022	0.022	0.000	---	NonR	0.000
	30.916	0.022	0.022	0.000			0.000

SHEAR (Ultimate): Max Vu (1.2DL+1.6LL) = 25.14 kips 0.90Vn = 85.08 kips

MOMENTS (Ultimate):

Span	Cond	LoadCombo	Mu kip-ft	@ ft	Lb ft	Cb	Phi	Phi*Mn kip-ft
Center	PreCmp+	1.4DL	3.7	15.5	0.0	1.00	0.90	124.50

DEFLECTIONS:

Initial load (in)	at	15.46 ft	=	-0.079	L/D =	4716
Live load (in)	at	15.46 ft	=	-0.736	L/D =	504
Post Comp load (in)	at	15.46 ft	=	-1.288	L/D =	288
Net Total load (in)	at	15.46 ft	=	-1.366	L/D =	272

Figure 2 Sample Composite Floor Beam Calculation

SPAN INFORMATION (ft): I-End (198.38,35.92) J-End (198.38,66.83)

Beam Size (Optimum) = W14X22 Fy = 50.0 ksi
Total Beam Length (ft) = 30.92
Mp (kip-ft) = 138.33

USER DEFINED FLANGE BRACE POINTS:

Dist (ft)	Top	Bottom
-1.#J	No	Yes

LINE LOADS (k/ft):

Load	Dist	DL	LL	Red%	Type
1	0.000	0.160	0.200	---	NonR
	30.916	0.160	0.200		
2	0.000	0.022	0.000	---	NonR
	30.916	0.022	0.000		

SHEAR (Ultimate): Max Vu (1.2DL+1.6LL) = 8.32 kips 0.90Vn = 85.08 kips

MOMENTS (Ultimate):

Span	Cond	LoadCombo	Mu kip-ft	@ ft	Lb ft	Cb	Phi	Phi*Mn kip-ft
Center	Max +	1.2DL+1.6LL	64.3	15.5	0.0	1.00	0.90	124.50
Controlling		1.2DL+1.6LL	64.3	15.5	0.0	1.00	0.90	124.50

REACTIONS (kips):

	Left	**Right**
DL reaction	2.81	2.81
Max +LL reaction	3.09	3.09
Max +total reaction (factored)	8.32	8.32

DEFLECTIONS:

Dead load (in)	at	15.46 ft =	-0.649	L/D =	572
Live load (in)	at	15.46 ft =	-0.712	L/D =	521
Net Total load (in)	at	15.46 ft =	-1.361	L/D =	273

Figure 3 Sample Non-Composite Roof Beam Calculation

Wind Calculations

Wind Direction														
N/S E/W	WINDWARD													
	HEIGHT	KZ	KZT	KD	V	I	QZ	GF	CP	qGCP	(+)GCPI	(-)GCPI	Qi(+)GCPI	Qi(-)GCPI
	0-15	0.57	1	0.85	90	1	10.05	0.85	0.8	6.83	0.18	-0.18	1.80838656	-1.8083866
	20	0.62	1	0.85	90	1	10.93	0.85	0.8	7.43	0.18	-0.18	1.96701696	-1.967017
	25	0.66	1	0.85	90	1	11.63	0.85	0.8	7.91	0.18	-0.18	2.09392128	-2.0939213
	30	0.7	1	0.85	90	1	12.34	0.85	0.8	8.39	0.18	-0.18	2.2208256	-2.2208256
	40	0.76	1	0.85	90	1	13.40	0.85	0.8	9.11	0.18	-0.18	2.41118208	-2.4111821
	50	0.81	1	0.85	90	1	14.28	0.85	0.8	9.71	0.18	-0.18	2.56981248	-2.5698125
	60	0.85	1	0.85	90	1	14.98	0.85	0.8	10.19	0.18	-0.18	2.6967168	-2.6967168
	70	0.89	1	0.85	90	1	15.69	0.85	0.8	10.67	0.18	-0.18	2.82362112	-2.8236211
	78	0.93	1	0.85	90	1	16.39	0.85	0.8	11.15	0.18	-0.18	2.95052544	-2.9505254
	80	0.93	1	0.85	90	1	16.39	0.85	0.8	11.15	0.18	-0.18	2.95052544	-2.9505254
Roof	WINDWARD													
N/S	Horiz Dist	KZ	KZT	KD	V	I	QZ	GF	CP	qGCP	(+)GCPI	(-)GCPI	Qi(+)GCPI	Qi(-)GCPI
h/L=80/61	0 to h/2	0.93	1	0.85	90	1	16.39	0.85	-1.3	-18.11				
>1	>h/2	0.93	1	0.85	90	1	16.39	0.85	-0.7	-9.75				
Roof	WINDWARD													
E/W	Horiz Dist	KZ	KZT	KD	V	I	QZ	GF	CP	qGCP	(+)GCPI	(-)GCPI	Qi(+)GCPI	Qi(-)GCPI
h/L=80/61	0 to h/2	0.93	1	0.85	90	1	16.39	0.85	-0.9	-12.54				
<0.5	h/2 to h	0.93	1	0.85	90	1	16.39	0.85	-0.9	-12.54				
	h to 2h	0.93	1	0.85	90	1	16.39	0.85	-0.5	-6.97				
	>2h	0.93	1	0.85	90	1	16.39	0.85	-0.3	-4.18				
N/S	LEEWARD													
	HEIGHT	KZ	KZT	KD	V	I	QZ	GF	CP	qGCP	(+)GCPI	(-)GCPI	Qi(+)GCPI	Qi(-)GCPI
	80	0.93	1	0.85	90	1	16.39	0.85	-0.5	-6.97	0.18	-0.18	2.95052544	-2.9505254
E/W	LEEWARD													
	HEIGHT	KZ	KZT	KD	V	I	QZ	GF	CP	qGCP	(+)GCPI	(-)GCPI	Qi(+)GCPI	Qi(-)GCPI
	80	0.93	1	0.85	90	1	16.39	0.85	-0.2	-2.79	0.18	-0.18	2.95052544	-2.9505254

Seismic Calculations

Floor Weight Calculations

TYPICAL FLOOR WEIGHT CALCULATION					
	SIZE	#	LENGTH (ft)	WEIGHT (lbs)	
	W8X10	16	192.33	1937	
FLOOR FRAMING	W12X19	20	593.12	11242	
	W12X22	2	17.3	382	
	W14X22	23	704.17	15551	
	W16X26	3	43.13	1127	
	W16X31	1	31.5	979	
	W16X36	2	34.43	1242	
	W18X40	1	24.42	980	
	W18X46	1	30.92	1420	
	W21X48	1	17.65	847	
	W21X55	11	264.42	14576	
	W24X55	12	295.92	16413	
	W21X57	10	309.17	17569	
	W24X62	1	24	1495	
	W21X68	19	537.33	36569	
				122329	lbs
DECK/SLAB	TYPE	PSF	SF		
	1.5VL20	39	16797	655083	lbs
	SI DL	20	16797	335940	lbs
	PSF	Length	Height		
WALL WT	15	736	15.33	169243.2	lbs
				1282595.2	lbs
			TOTAL WT	1282.5952	kips

Roof Weight Calculation

ROOF WEIGHT CALCULATION					
	SIZE	#	LENGTH (ft)	WEIGHT (lbs)	
	W8X10	4	25.94	261	
FLOOR FRAMING	W12X19	2	63	1194	
	W14X22	39	1218	26898	
	W16X26	13	354.06	9253	
	W16X31	5	120	3728	
	W18X35	22	653.17	22893	
	W16X36	2	34.43	1242	
	W21X44	8	192.42	8512	
	W18X46	1	24.42	1122	
	W18X50	8	192.42	9625	
	W21X62	12	254.38	15840	
				100568	
DECK/SLAB	TYPE	PSF	SF		
	NON-com	3	16797	50391	lbs
	SI DL	20	16797	335940	lbs
	PSF	Length	Height		
WALL WT	15	736	15.33	169243.2	lbs
MECH UNITS	RTU1			8048	
	RTU2			10588	
	RTU3			8036	
				682814.2	lbs
			TOTAL WT	682.8142	kips

Seismic Force Calculation

11.4.1

0.2 Second Spectral Response Acceleration [5% of Critical Damping]

$S_s = 0.162$ [Figure 21-1]

1.0 Second Spectral Response Acceleration [5% of Critical Damping]

$S_1 = 0.052$ [Figure 21-3]

11.4.2

Site Classification: D

Ss<0.25

S1<0.1

11.4.3

Site Coefficients and Adjusted Maximum Considered Earthquake Spectral Response Acceleration

$F_a = 1.6$ [Table 11.4-1]

$S_{MS} = F_a S_s = (1.6)(0.162) = 0.2592$ [Equation 11.4-1]

$F_v = 2.4$ [Table 11.4-2]

$S_{M1} = F_v S_1 = (2.4)(0.052) = 0.1248$ [Equation 11.4-2]

11.4.4

Design Spectral Acceleration

$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3}(0.2592) = 0.1728$ [Equation 11.4-3]

$S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3}(0.1428) = 0.0832$ [Equation 11.4-4]

12.8.2

Period Determination

12.8.2.1

Approximate Fundamental Period

$C_t = 0.028$ [Table 12.8-2]

$x = 0.8$

$h_n = 46$

$T_a = C_t h_n^x = (0.028)(77)^{0.8} = 0.904$

$C_u = 1.7$ [Table 12.8-1]

$T = C_u T_a = (1.7)(0.904) = 1.54$

11.4.5
Design Response Spectrum

$$T_0 = 0.2\frac{S_{D1}}{S_{DS}} = 0.2\frac{0.0832}{0.1728} = 0.0930$$

$$T_S = \frac{S_{D1}}{S_{DS}} = \frac{0.0832}{0.1728} = 0.4815$$

$T_L = 8$ [Figure 22-15]

3. For $T > T_S$ and $T \le T_L$

$$S_a = \frac{S_{D1}}{T} = \frac{0.0832}{1.018} = 0.0817$$

11.5.1
Occupancy Category: II [Table 1-1]
Importance Factor: $I = 1.0$ [Table 11.5-1]

11.6 Seismic Design Category
Seismic Design Category Based on 1-s Period Response Acceleration: B [Table 11.6-2]

$$0.067 \le S_{D1} < 0.133$$

12.8 Equivalent Lateral Force Procedure
12.8.1 Seismic Base Shear
$V = C_s W$ [Equation 12.8-2]

12.2 Structural System Selection
Response Modification Coefficient: $R = 3.5$
System Overstrength Factor: $\omega_0 = 3.0$
Deflection Amplification Factor: $C_d = 3.0$
Structural System Limitations and Building Height Limit: NL for SDC B, C, D, E, F
12.8.1.1 Calculation of Seismic Response Coefficient
For $T \le T_L$

$$C_s = \min\begin{cases} \dfrac{S_{DS}}{\left(\dfrac{R}{I}\right)} = \dfrac{0.1728}{\left(\dfrac{3.5}{1.0}\right)} = 0.0494 \\ \dfrac{S_{D1}}{T\left(\dfrac{R}{I}\right)} = \dfrac{0.0832}{0.904\left(\dfrac{3.5}{1.0}\right)} = 0.0263 \end{cases} \ge 0.01$$

12.8.3 Vertical Distribution of Seismic Forces

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$$ [Equation 12.8-12]

Base Shear

$$V = C_s W = (0.0263)(5813.2) = 177.16\,\text{kips}$$

Shear at Roof Level

$$C_{v1} = 0.224$$

$$C_{v1}V = (0.224)(177.16) = 39.68\,\text{ips}$$

Shear at Fifth Floor

$$C_{v1} = 0.326$$

$$C_{v1}V = (0.326)(177.16) = 57.75\,\text{kips}$$

Shear at Fourth Floor

$$C_{v1} = 0.235$$

$$C_{v1}V = (0.235)(177.16) = 41.63\,\text{kips}$$

Shear at Third Floor

$$C_{v1} = 0.235$$

$$C_{v1}V = (0.148)(177.16) = 26.22\,\text{kips}$$

Shear at Second Floor

$$C_{v1} = 0.067$$

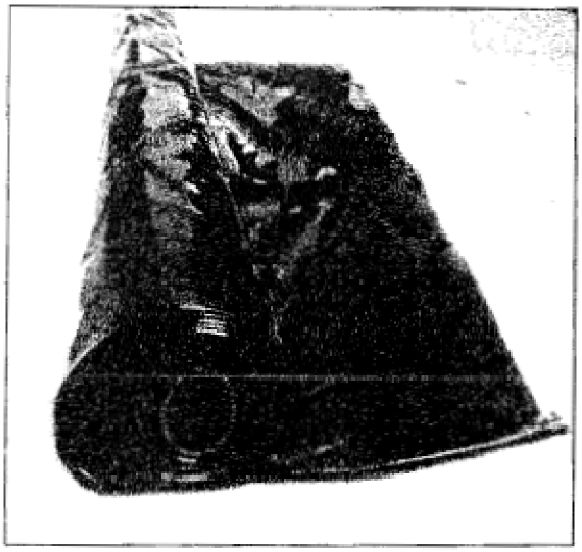
$$C_{v1}V = (0.067)(177.16) = 11.87\,\text{kips}$$

Green Roof Material Product Data Sheets

Root Stop WSF 40

HYDROTECH

ROOT STOP WSF 40 **PRODUCT DATA SHEET**



GENERAL DESCRIPTION

Root Stop WSF 40 is a specially formulated, electronically tested, root protection sheet made of polyethylene. WSF 40 has been subjected to testing in accordance with FLL guidelines.

BASIC USE

Root Stop WSF 40 is specifically designed to be loose-laid over the Hydroflex protection sheet. This product is used as a root barrier for the MM 6125-EV roofing membrane in extensive landscaping conditions as part of Hydrotech's Garden Roof™ system.

TECHNICAL DATA

COLOR:	Black
THICKNESS:	0.015 in. (15 mils)
SIZE:	26.4 ft. X 82.5 ft. (8m X 25m) - in rolls (2,152 sq.ft. total)
WEIGHT:	approx. 122 lb./roll (55 kg/roll)

INSTALLATION

- Root Stop WSF 40 is to be unrolled over a completed MM 6125-EV/Hydroflex 30 roofing membrane assembly and unfolded minimizing wrinkles.
- Adjacent sheets must be lapped a minimum of 5 feet to prevent lateral root growth down to the membrane level.
- Root Stop WSF 40 should be dressed up all roofing termination points and flashings to provide total root protection to all roofed/flashed surfaces that will be subsequently covered with soil.
- As Root Stop WSF 40 is installed loose-laid, temporary ballasting may be required prior to the installation of subsequent topping materials.

American Hydrotech, Inc.
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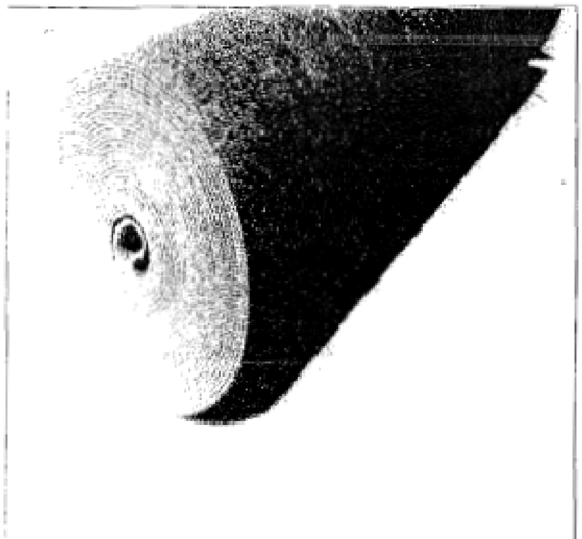
Moisture Retention Mat

HYDROTECH

MOISTURE RETENTION *PRODUCT DATA SHEET*



GENERAL DESCRIPTION

Moisture Retention Mat SSM 45 is composed of high-quality, recycled, non-rotting, poly-propylene fibers, stitched through a polyethylene sheet carrier.

BASIC USE

The SSM 45 Moisture Retention Mat is used in the Hydrotech Garden Roof™ system to retain moisture and nutrients for use by the vegetation layer.

TECHNICAL DATA

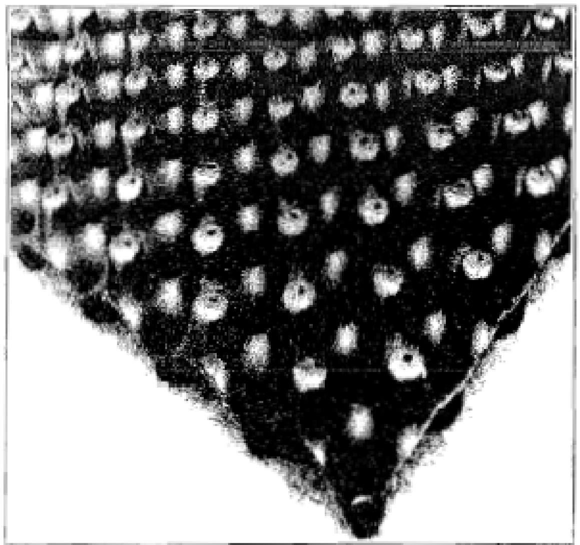
THICKNESS:	3/16 in. (≈200 mils)
SIZE:	6.6 ft. X 164 ft. (2m X 50m) - in rolls (1,076 sq.ft. total)
WEIGHT:	111 lb./roll (50 kg./roll) 0.2 lb./sq.ft. (1 kg./sq.m.) - dry; 1 lb./sq.ft. (5 kg./sq.m.) - wet
WATER RETENTION CAPACITY:	0.12 gal./sq.ft. (5 l./sq.m.)

INSTALLATION

- Moisture Retention Mat SSM 45 is to be rolled out over the root barrier or insulation/air layer.
- Adjacent sheets must be lapped a minimum of 4 inches.
- SSM 45 should be extended a minimum of 6 inches beyond the anticipated level of the soil. Excess can be trimmed down to the soil surface once the soil has been installed.
- As SSM 45 is installed loose-laid, temporary ballasting may be required prior to the installation of subsequent topping materials.

FLORADRAIN FD40

FLORADRAIN FD40 PRODUCT DATA SHEET



GENERAL DESCRIPTION

Floradrain FD40 is made of recycled polyethylene, molded into a three-dimensional panel. The unique design provides retention cups on the top side, drainage channels on top and bottom and holes in the tops of the "domes" for ventilation and evaporation.

BASIC USE

Floradrain FD40 is specifically designed to act as a drainage and water retention element in Hydrotech's Garden Roof™ system. It is typically utilized under "extensive" landscaping.

TECHNICAL DATA

PANEL DIMENSIONS:	3.3 ft. X 6.6 ft. (1 m X 2 m) – in panels (21.52 sq.ft. total)	
PANEL HEIGHT:	1 ½ in. (40 mm)	
WEIGHT:	0.4 lb./sq.ft. - dry; 1.2 lb./sq.ft. - wet	w/cups empty
	3.5 lb./sq.ft. - dry; 4.7 lb./sq.ft. - wet	w/cups filled
COMPRESSIVE STRENGTH: (ASTM D1621)	3,326 lb./sq.ft. w/cups empty; 7,330 lb./sq.ft. w/cups filled	
FLOW RATE: (ASTM D4716)	513 gal./min./ft^2	
WATER RETENTION:	≈0.1 gal./sq.ft. (4 l./sq.m.)	w/cups empty or filled
VOLUME TO FILL:	≈0.06 cu.ft. for every 1 sq.ft. in area (17 l./sq.m.)	

INSTALLATION

- FD40 is to be installed loose-laid over the Moisture Retention Mat SSM45 over the entire surface of the roof.
- FD40 is easily cut to fit around penetrations, perimeters, drains, etc. with a heavy-duty utility knife or small- toothed saw.
- Adjacent panels are typically butt together.
- As substrate soil loads are typically shallow, lightweight and made up primarily of mineral soil, the retention cups of FD40 are typically not filled with mineral soil.

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Lateral Calculations

Moment Diagrams

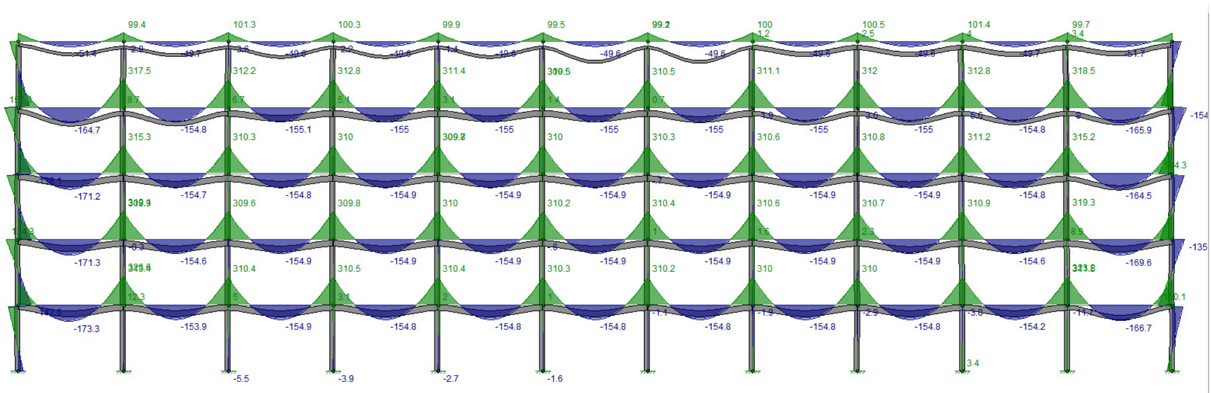


Figure 4 Moment Diagram with Deflected Shape for East-West Frame under Load Combination 2

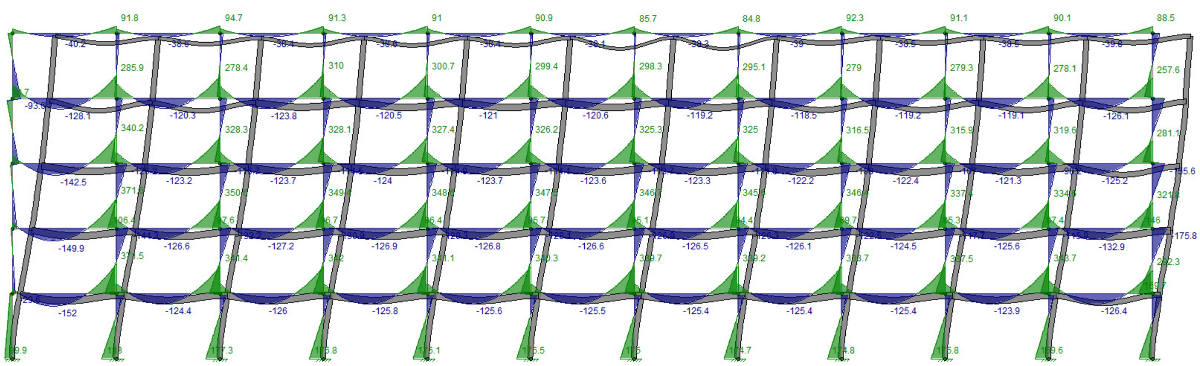


Figure 5 Moment Diagram with Deflected Shape for East-West Frame under Load Combination 14

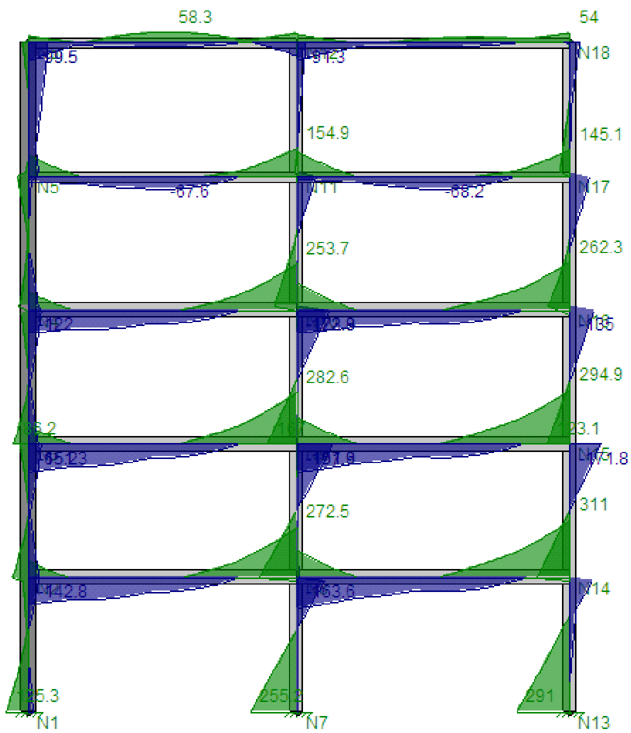


Figure 6 North-South Frame Moment Diagram

Foundation Calculations

M-17 Footing Results

Min. Ratio	Item	Applied	Capacity	Governing Load Combination
0.99797	Soil Bearing	2.9939 ksf	3.0 ksf	+0.60D+W+H
n/a	Overturning - X-X	0.0 k-ft	0.0 k-ft	No Overturning
1.7620	Overturning - Z-Z	291.0 k-ft	512.744 k-ft	+0.60D+W+H
n/a	Sliding - X-X	0.0 k	0.0 k	No Sliding
n/a	Sliding - Z-Z	0.0 k	0.0 k	No Sliding
n/a	Uplift	0.0 k	0.0 k	No Uplift
0.49553	Z Flexure (+X)	28.6475 k-ft	57.8118 k-ft	+1.20D+0.50Lr+1.60L+1.60H
0.49553	Z Flexure (-X)	28.6475 k-ft	57.8118 k-ft	+1.20D+0.50Lr+1.60L+1.60H
0.57482	X Flexure (+Z)	33.2311 k-ft	57.8118 k-ft	+1.20D+0.50Lr+0.50L+1.60W
0.49553	X Flexure (-Z)	28.6475 k-ft	57.8118 k-ft	+1.20D+0.50Lr+1.60L+1.60H
0.30390	1-way Shear (+X)	28.2969 psi	93.1128 psi	+1.20D+0.50Lr+1.60L+1.60H
0.30390	1-way Shear (-X)	28.2969 psi	93.1128 psi	+1.20D+0.50Lr+1.60L+1.60H
0.30390	1-way Shear (+Z)	28.2969 psi	93.1128 psi	+1.20D+0.50Lr+1.60L+1.60H
0.30390	1-way Shear (-Z)	28.2969 psi	93.1128 psi	+1.20D+0.50Lr+1.60L+1.60H
0.39533	2-way Punching	73.6210 psi	186.226 psi	+1.20D+0.50Lr+1.60L+1.60H

Figure 7 M-17 Foundation Redesign

Load Combination	Overturning	Resisting	Safety Ratio
+D	None	854.574 k-ft	Infinity
+D+L+H	None	1,524.47 k-ft	Infinity
+D+Lr+H	None	854.574 k-ft	Infinity
+D+0.750Lr+0.750L+H	None	1,357.0 k-ft	Infinity
+D+W+H	None	854.574 k-ft	Infinity
+D+0.750Lr+0.750L+0.75	None	1,357.0 k-ft	Infinity
+0.60D+W+H	None	512.744 k-ft	Infinity
Z-Z			
+D	None	854.574 k-ft	Infinity
+D+L+H	None	1,524.47 k-ft	Infinity
+D+Lr+H	None	854.574 k-ft	Infinity
+D+0.750Lr+0.750L+H	None	1,357.0 k-ft	Infinity
+D+W+H	291.0 k-ft	854.574 k-ft	2.9367
+D+0.750Lr+0.750L+0.75	218.250 k-ft	1,357.0 k-ft	6.2176
+0.60D+W+H	291.0 k-ft	512.744 k-ft	1.7620

Figure 8 M-17 Foundation Redesign

S-17 Footing Results

	Min. Ratio	Item	Applied	Capacity	Governing Load Combination
✔	0.93041	Soil Bearing	2.7912 ksf	3.0 ksf	+D+0.750Lr+0.750L+0.750W+H
✔	n/a	Overturning - X-X	0.0 k-ft	0.0 k-ft	No Overturning
✔	4.5415	Overturning - Z-Z	255.20 k-ft	1,158.99 k-ft	+0.60D+W+H
✔	n/a	Sliding - X-X	0.0 k	0.0 k	No Sliding
✔	n/a	Sliding - Z-Z	0.0 k	0.0 k	No Sliding
✔	n/a	Uplift	0.0 k	0.0 k	No Uplift
✔	0.86690	Z Flexure (+X)	63.0754 k-ft	72.7601 k-ft	+1.20D+0.50Lr+1.60L+1.60H
✔	0.86690	Z Flexure (-X)	63.0754 k-ft	72.7601 k-ft	+1.20D+0.50Lr+1.60L+1.60H
✔	0.86690	X Flexure (+Z)	63.0754 k-ft	72.7601 k-ft	+1.20D+0.50Lr+1.60L+1.60H
✔	0.86690	X Flexure (-Z)	63.0754 k-ft	72.7601 k-ft	+1.20D+0.50Lr+1.60L+1.60H
✔	0.61466	1-way Shear (+X)	57.2330 psi	93.1128 psi	+1.20D+0.50Lr+1.60L+1.60H
✔	0.61466	1-way Shear (-X)	57.2330 psi	93.1128 psi	+1.20D+0.50Lr+1.60L+1.60H
✔	0.61466	1-way Shear (+Z)	57.2330 psi	93.1128 psi	+1.20D+0.50Lr+1.60L+1.60H
✔	0.61466	1-way Shear (-Z)	57.2330 psi	93.1128 psi	+1.20D+0.50Lr+1.60L+1.60H
✔	0.85013	2-way Punching	158.316 psi	186.226 psi	+1.20D+0.50Lr+1.60L+1.60H

Figure 9 S-17 Foundation Redesign

M-17 Footing Load

Column Line 62.38ft - 35.92ft at Level Story 4

Dead	+	Live Loads			+	Min Total	Max Total
Load		Pos	% Red	Neg	% Red		
	Reducible	0.00	0.00	0.00	0.00		
	Storage	0.00	0.00	0.00	0.00		
	UnRed.	141.47	--	0.00	--		
Load		Pos	% Red	Neg	% Red		
	Roof	0.00	0.00	0.00	0.00		
117.94		141.47		0.00		117.94	259.41

S-17 Footing Load

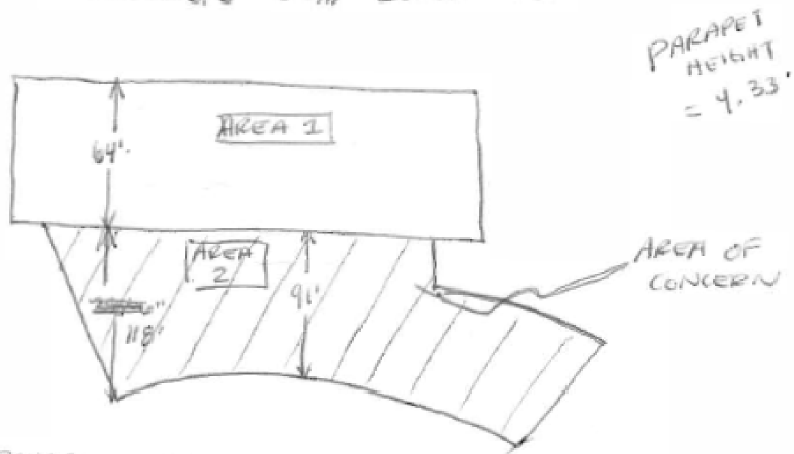
Column Line 86.38ft - 35.92ft

Level	Col#	Height	Dead	Self	+Live	-Live	MinTot	MaxTot
Roof	17	15.33	18.1	0.7	18.7	0.0	18.8	37.5
Story 5	18	15.33	66.7	1.4	78.6	0.0	68.1	146.7
Story 4	18	15.33	115.3	2.6	138.5	0.0	117.8	256.3
Story 3	18	15.33	163.9	3.8	198.3	0.0	167.6	366.0
Story 2	18	15.33	212.4	5.0	258.2	0.0	217.4	475.6

Snow Load Calculations

SNOW LOAD CALCULATION

SURCHARGE ON LOWER ROOF



SURCHARGE FROM

AREA 1 ONTO AREA 2 (LEEWARD)

GROUND SNOW LOAD = 30 PSF = P_g

FLAT ROOF SNOW LOAD = 0.7 (1.0)(1.0)(1.0) (30) = 21 PSF

$\gamma = 17.9$ LENGTH OF UPPER ROOF $\ell_u = 64'-0''$

SLOPE OF UPPER ROOF = ≈ 0 (FLAT

$h_b = \frac{21 \text{ PSF}}{17.9} = 1.17'$

$h_c = \approx 34'$ $\qquad \frac{h_c}{h_b} > 0.2$

$h_d = 0.43 \sqrt[3]{\ell_u} \sqrt[4]{P_g + 10} - 1.5$

$= 0.43 \sqrt[3]{64} \sqrt[4]{30 + 10} - 1.5 = 2.83'$ ← HEIGHT OF DRIFT ABOVE BALANCED LOAD

THE LEEWARD DRIFT, IN PSF, IS

$2.83' \times 17.9 \text{ PCF} = \boxed{50.66 \text{ PSF}}$

WINDWARD DRIFT CALCULATION
(AREA 2)

USE $\frac{3}{4}h_d$ for WINDWARD

LENGTH OF LOWER ROOF, l_u = 118'

p_f = 21

Slope ≈ 0

h_b = 1.17

$$h_d = 0.75\left[0.43\sqrt[3]{l_u}\,\sqrt[4]{p_g + 10} - 1.5\right] =$$

$$= 0.75\left[0.43\sqrt[3]{118}\,\sqrt[4]{40} - 1.5\right] = 2.85'$$

2.85 > 2.83, WINDWARD CONTROLS

$2.85' \times 17.9\ \text{PCF} = \boxed{51\ \text{PSF}}$

SURCHARGE LOADING