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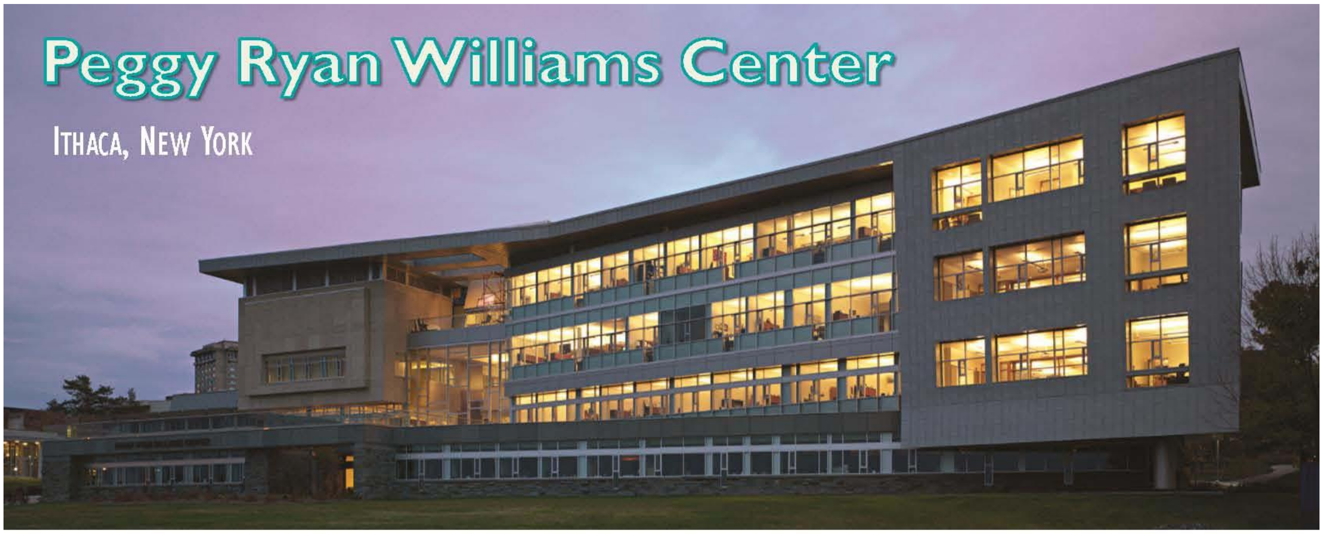
Structural Option
Advisor: Dr. Boothby
Peggy Ryan Williams Center
Ithaca, New York
9 April 2014

Peggy Ryan Williams Center

Final Report



Primary Project Team:

Owner | Ithaca College
Architect | Holt Architects
Structural Engineer | Ryan-Biggs Associates
Mechanical & Electrical Engineer | Delta Engineers
General Contractor | Christa Construction

General Building Data:

Building Occupant | Ithaca College
Occupancy | Office Use
Size | 58,200 gross square feet
Stories | 4 stories above grade
Substantial Completion | March 2010
Cost of Construction | approx. $19.3 million
Project Delivery Method | Design-Bid-Build

Architecture:

- Various aspects were driven by desire to be eco-friendly
- Large areas of glass provide views of Cayuga Lake
- Façade consists of zinc panels, blue stone veneer, composite aluminum panels, and limestone panels
- Pedestrian bridge connects PRWC to adjacent building

Structure:

- *Foundation*
  - Slab-on-grade, foundation walls, footings, various grade beams, piers and drilled piers
- *Framing System*
  - All floors are composed of composite steel decking
  - Steel framing consists of wide flange beams, girders, and columns
- *Lateral System*
  - Concentrically braced structural steel frames in both the North-South and East-West directions

Sustainability:

- Awarded LEED Platinum
- "V" shaped roof aids in rain water collection
- Day lighting made possible by large areas of glass
- Intensive Green Roof
- Atrium promotes natural ventilation

MEP:

- *Mechanical*
  - Main heating and cooling source is geothermal via a closed loop system adjacent to the building
  - Two dedicated outdoor air units (DOA) will utilize water to water heat pumps
- *Electrical*
  - Primary Service: 12.5 KV primary fused switches, 500 KVA transformer, 480/277 Volt Distribution Switchboard
  - Secondary Distribution: 150 KVA, 480V to 120/208 Volt transformer and (1) 120/208 Volt Main power panel
- *Plumbing*
  - Collect and store rainwater for gray water use
  - (3) rainwater collections tanks

Angela Mincemoyer | Structural | http://www.engr.psu.edu/ae/thesis/portfolios/2014/ahm5066/index.html

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Executive Summary

The Peggy Ryan Williams Center houses Ithaca College's admissions staff as well as numerous administrative offices at its location in Ithaca, New York. The building is an important feature of the college because it was intended to show its occupants and visitors that Ithaca College was moving forward and working to be more sustainable with their designs. Many of the architectural features of the building were influenced by the desire to be more "green" and to allow its occupants to view the nature around them. The existing building is a composite steel design with concentrically braced structural steel frames.

The following report consists of two main parts, the existing system and the redesigned system of the PRWC. The first section of the report explains some of the architectural and structural aspects of the building. The second portion of the report contains the details of the redesign of the existing steel building into a reinforced concrete building. In addition to presenting the existing and redesigned building, the pedestrian bridge, which is attached to the building, is also explained in detail. The bridge connects the PRWC to the adjacent Dillingham Center.

The first part of the redesign consisted of redesigning the gravity system of the building. One of the reasons that steel was originally chosen for the building material was due to a need to expedite the project schedule. However, a scenario was created in which the schedule was no longer critical. Therefore, the PRWC was redesigned using reinforced concrete. It was determined to complete the redesign using a one way concrete slab system with pan joists, girders, and columns. By using joists, the slab would only be required to span the small distance between the joists, thus allowing for a smaller slab depth. Therefore, it was hoped to decrease the original floor system depth. By orienting the joists along the existing steel beam span and then placing the concrete girders where the existing steel girders are located, the column locations would not need to be changed, thus the impact on the architecture would be low. By redesigning the building using concrete, the steel braced frames were no longer the best option. Since the building is only four stories, there was potential that the building's gravity system would double as its lateral system.

In addition to the redesign of the main building, a portion of the pedestrian bridge was also redesigned. Two inspirational concepts were considered for the redesign, a reflection on the building's original name, "The Gateway Building," and a reflection of New York's historical covered bridges. Upon choosing an inspiration to use for the redesign, one of the side trusses of the bridge was redesigned. This structural redesign led to both an architectural breadth on the façade of the pedestrian bridge and a lighting breadth of the exterior of the bridge.

Through the redesign, the floor system depth was decreased by changing the building material to concrete. This helped to open up the interior spaces and allow for a larger floor to ceiling height. The number of columns and girders in the building was decreased, which allowed for a more open floor plan. Finally, it was determined that the gravity system of the building was adequate to act as both the gravity system and the lateral system of the building.

Acknowledgements

I would first like to thank Ithaca College for allowing me to study the Peggy Ryan Williams Center this year. I would also like to thank Ryan-Biggs Associates and Holt Architects for providing to me the necessary information to complete this project.

None of this work would have been possible without the entire PSU AE Faculty. Thank you all for passing some of your knowledge onto me. I would specifically like to thank Dr. Boothby for always helping me and guiding me in my college career and beyond.

Finally, tremendous thanks to all of my family and friends for supporting me and always being there for me these last 5 years. I could not have done all of this without you guys. Thank you!

Building Introduction

With the global push towards sustainability, the Ithaca College decided that it was important to show that their college was moving forward with the times, being eco-friendly, and wanting to incorporate their beautiful surroundings into the campus design. This led to a new era of architecture at Ithaca campus.

The Peggy Ryan Williams Center (PRWC) is a key aspect of fulfilling the new architectural objectives of the college because it is seen as a gateway. The occupants of this 58,200 square foot, 74 foot tall building include the college's admissions staff as well as numerous administrative offices. A typical floor plan may be viewed below in Figure 1. The building is also one of the first sights that visitors see upon arriving to the campus. Therefore, Ithaca College saw the building as a way to show perspective students, employees, and visitors that their college was moving forward to be more "green" and incorporate the surrounding nature.

The architecture of the building was also driven by a desire to allow its occupants to not only view the nature around them; but, also, to feel as if they are a part of it. These sensations were achieved by providing large areas of glass and designing a floor plan at angles other than 90 degrees. The irregular angles help to direct the occupants' eyes to the most appealing surroundings, such as the breath-taking view of the nearby Cayuga Lake. The resultant irregular floor plan may be seen on Figure 1 and Figure 2 below.

Another important feature of the PRWC is the pedestrian bridge, which may be viewed in Figure 3 below. The bridge allows its users to go between the PRWC and the adjacent Dillingham Center without going outdoors. A glass façade allows large amounts of light penetration while tying this façade feature to the main building.

LEED Platinum is the prestigious title that the Peggy Ryan Williams Center was awarded by USGBC. However, this achievement required years of planning and sustainability considerations. Most of the architectural appearance of the building was governed by sustainability. Some examples of sustainability include the main roof taking on a slight "V" shape as to help collect rain water, the atrium being designed to assist with natural ventilation, green roofs, geothermal heat wells, solar shading, and many large areas of glass to allow for day lighting.

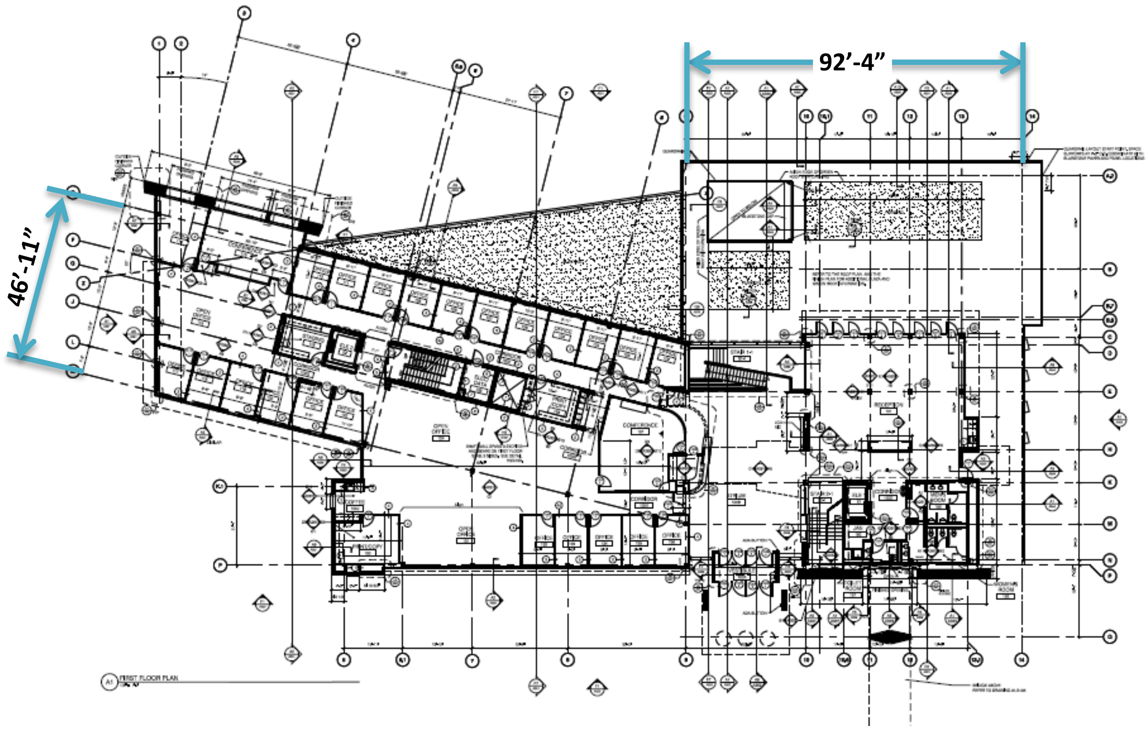


Figure 1: Typical Floor Plan (Level 1)
Drawing A101

Figure 2: View from the North Showing Irregular Facade of the PRWC
Photo provided courtesy of Holt Architects

Figure 3: View from the Southeast Showing the Pedestrian Bridge
Photo provided courtesy of Holt Architects

Structural Overview of the Building

The structural gravity system of the Peggy Ryan Williams Center consists of composite decking supported by wide flange beams, girders, and columns. The foundation consists of reinforced concrete grade beams and piers. The lateral system is comprised of concentrically braced structural steel frames. The following sections will discuss these components in detail, as well as material strengths.

Materials

The structural materials used throughout the PRWC are various strengths of steel and concrete. These material strengths may be viewed below in Table 1 and Table 2.

Steel Shape	Steel Grade
Rolled Steel W Shapes	ASTM A992 Grade 50
Rolled Steel C and MC Shapes	ASTM A36
Rolled Steel Plates, Bars, & Angles	ASTM A36
Hollow Structural Sections (HSS)	ASTM A500, Grade B or C
Pipe	ASTM A53, type E or S, Grade B
*For connections, provide higher grade as required for capacity.	

Table 1: Structural and Miscellaneous Steel Strengths (*Drawing S001*)

Concrete Component	Concrete Strength
Footings, Foundation Walls, Piers, Miscellaneous	f'c = 4,000 psi
Interior Slabs on Grade or Slabs on Deck	f'c = 3,500 psi
Retaining Walls, Basement Walls, Exterior Slabs, and Grade Beams	f'c = 4,000 psi
*Reinforcing Steel for Concrete → ASTM A615, Grade 60	

Table 2: Concrete Material Strengths (Drawing S001)

Geotechnical Report and Recommendations

Through their studies, the Geotechnical Engineer (CME Associates, Inc.) made numerous recommendations for the foundation of the Peggy Ryan Williams Center. On the north side, shale bedrock was found 15 feet below grade with unprepared fill on top. The bedrock stratum is underlain by silt. The 2002 Building Code of New York State (BCNYS) does not allow a foundation to bear on unprepared fill. Therefore, all foundations were required to bear on competent shale bedrock. The competent bedrock was presumed to have a soil bearing pressure of 20,000 psf. There is no need to drill into the exposed bedrock on the south side. In order to have competent bearing, CME Associates, Inc. recommends using drilled piers. This conclusion was drawn due to the variable depth to a competent bearing surface and the risks associated with large excavations close to groundwater. CME also recommended that all drilled piers should have a planned bottom elevation not less than 2'-6" below the top of the shale bedrock and a diameter not less than 2'-0". In regards to the drilled piers, the design and construction should follow ACI 336.3R.

Foundation System

The PRWC foundation includes a wide variety of structural components ranging from grade beams to drilled piers. The foundation walls themselves range from 1'-0" thick with 3'-0" wide footings to 1'-8.5" thick with 6'-0" wide footings. In areas where the footings cannot reach down to competent bedrock, drilled piers are used in combination with piers to reach bedrock. Most areas of the building on the Garden Level are provided with a 5" concrete slab-on-grade. This slab is depressed in areas where special flooring is used. In various portions of the building, grade beams are utilized to transfer the loads of bearing walls from above (stairwell and elevator shaft), braced frames, and to help tie back the column supporting the overhang in the north corner of the building. The grade beam sizes range from 12" wide and 36" deep to 51" wide and 48" deep.

Loads from the grade beams are then transferred to piers and in turn to the drilled piers in order to finally reach competent bedrock. The piers range in size and shape depending on the location. The loads from these piers are then transferred to the drilled piers. All of the drilled piers are 3'-0" in diameter. Pier depths range from simply resting on top of the bedrock to being drilled 4'-0" below the surface of the bedrock.

Gravity System

Floor System

Each level of the PRWC has a 6" concrete slab on a 3"x20 gauge galvanized composite metal deck. However, a few areas have some deviation from this typical floor system. One example of this deviation occurs on the plaza deck and green roof areas. These areas have reinforcement in the deck system to lessen the effects of shrinkage and thermal contraction/expansion. Due to this slab being exposed to the weather, it is prone to the above thermal effects. The corrugations of each of the various types of decking run perpendicular to the wide-flange beams.

(Note: The Garden Level floor system (slab-on-grade) was discussed above in the foundation system section.)

Framing System

The structural framing system of the PRWC is very irregular due to changes in geometry, cantilevers, and locations of increased loads (such as adjacent to elevator shafts and stairwells). Levels 1 through 3 include numerous beam and girder sizes and spans. On those levels there are three different regions which utilize consistent beam shapes and sizes up through the levels. These regions may be viewed in Figure 4 below.

The vast majority of the columns from the foundation (Garden Level) continue up through the building. The columns range from W8x28 to W10x60, while some HSS5x5x5/16 are also present. Column type 2 (W10x49) is the most commonly used size throughout the superstructure of the building. On Level 1, various W10x39 columns were added along the southern perimeter of the building. These columns bear on the load bearing foundation. A few columns are also added to the cantilevered regions in upper levels of the building. These columns are typically W8x48 or W8x31. The column schedule may be viewed in Figure 5 below. These columns have a pinned connection at their base which allows no moment transfer to the pier below.

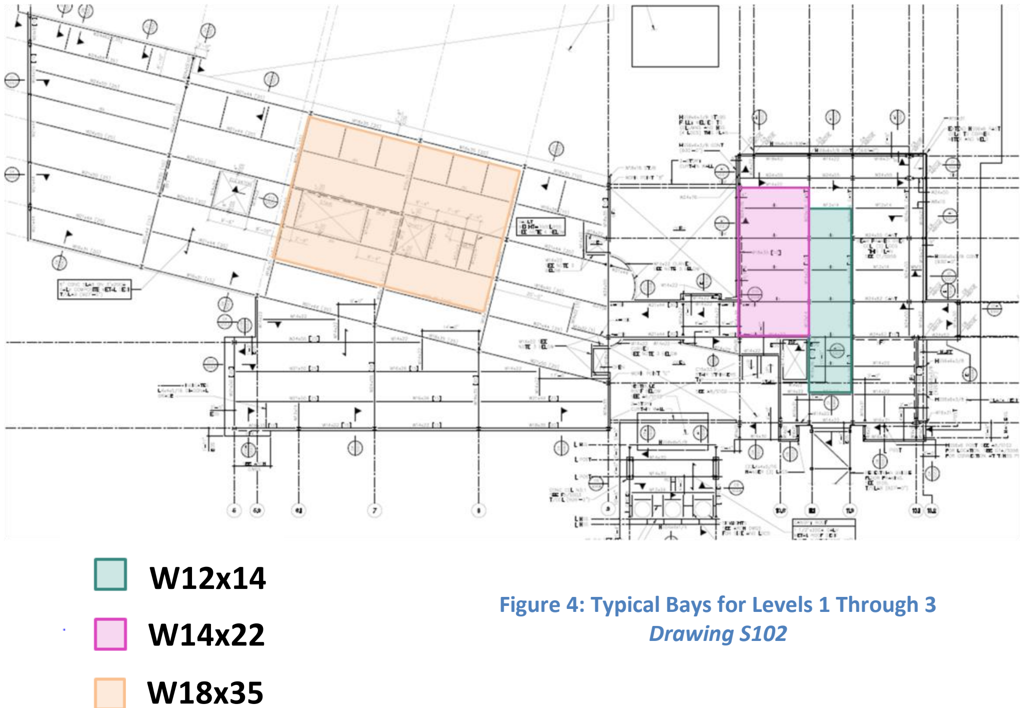


Figure 4: Typical Bays for Levels 1 Through 3
Drawing S102

COLUMN SCHEDULE		
MARK	SIZE	BASE PLATE TYPE
C1	W10x60	BP1
C2	W10x49	BP1
C3	W10x39	BP2
C4	W8x48	C1/S559
C5	W8x31	C1/S559
C6	HSS5x5x5/16	BP6
C7	W8x28	BP3

NOTE:
BASE PLATE TYPES ARE TYPICAL FOR COLUMNS INDICATED IN SCHEDULE UNLESS NOTED OTHERWISE ON PLAN. SEE F1/S555 FOR BASE PLATE DETAILS.

Figure 5: Column Schedule
Drawing S555

Roof Gravity System

The roof system of the PRWC follows the same basic structural system of the floors below; decking, wide-flange beams, girders, and columns. However, the roof is not supported by a composite deck. Instead, since the roof does not support as large of a load, a much lighter 1.5”x20 gauge galvanized metal roof deck is used. The deck is then supported by wide flange steel beams and girders. A tapered HSS8x6x3/8 sits on top of the wide-flange girders along the perimeter of the building. The HSS is tapered to match the slope of the roof deck which it supports. A roof cantilever (5’-10”) is formed from wide-flange beams spaced at 5’-3”.

Typical Gravity Loads

The following loads, as seen in Table 3 and Table 4, were based on ASCE7-98 and industry standards.

Location	Typical Dead Load (psf)	Typical Live Load (psf)
Floor	87.5	80
Green Roof	171	100
Roof	43.2	35 (snow)

Table 3: Typical Dead and Live Loads

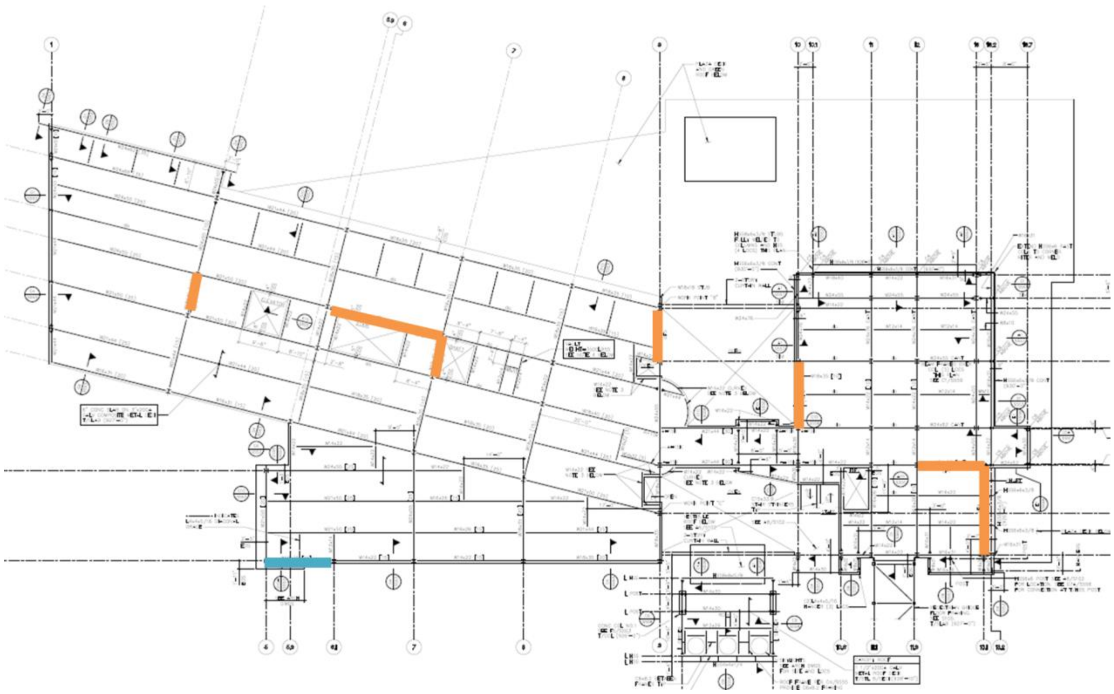
Exterior Wall Type	Typical Dead Load (psf)
Zinc Panel	13.9
Aluminum Storefront	12.0
Composit Aluminum Panel	12.9
Limestone Panel	28.0
Blue Stone Veneer	176.5

Table 4: Exterior Wall Loads

Lateral System

In both the North-South and the East-West directions, concentrically braced structural steel frames resist the lateral load. The braced frames are located throughout the building and may be seen on the plan below (Figure 6). Braced frame columns are typically W10s, while HSS6x6x3/8 are commonly used for the diagonal braces. A typical braced frame may be viewed below in Figure 7.

Various braced frames are provided in the north-south direction to resist the lateral loads. However, in the east-west direction, there is a lack of effective braced frames. In order to resist unbalanced loads there should be at least two (staggered) frames in each direction.



Braced Frame Extends from Garden Level to Roof

Braced Frame Extends from Level 1 to Level 3

Figure 6: Level 2 Braced Frame Layout
Drawing S102

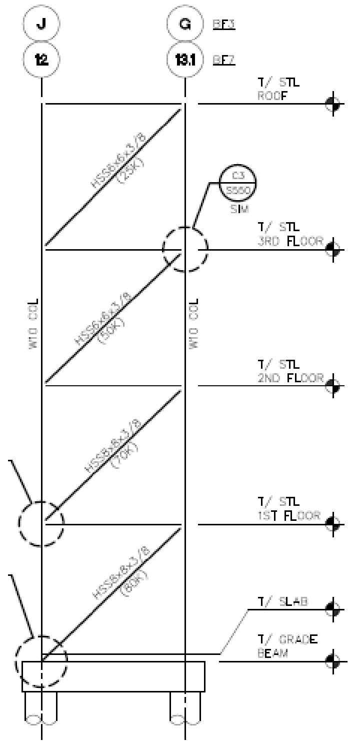


Figure 7: Typical Braced Frame
Drawing S550

Lateral Loads Summary

Note: The following lateral loads were calculated based on a simplified version of the PRWC, which only included the east end of the building.

Wind Loads

Wind loading was determined in accordance with ASCE7-98. Table 5 and Table 6 below show a summary of the wind forces in the North-South and the East-West directions. For the analysis, all four wind cases illustrated in ASCE7-98 Figure 6-9 were considered.

Diaphragm	Windward Pressure (psf)	Leeward Pressure (psf)
Level 1	7.73	-7.26
Level 2	9.18	-7.26
Level 3	10.17	-7.26
Roof	14.86	-10.51

Table 5: North-South Direction Wind Loads

Diaphragm	Windward Pressure (psf)	Leeward Pressure (psf)
Level 1	7.72	-4.29
Level 2	9.31	-4.29
Level 3	10.39	-4.29
Roof	15.05	-7.50

Table 6: East-West Direction Wind Loads

Seismic Loads

Four seismic load cases were used to calculate the applied seismic forces. Two of these load cases were in the North-South direction, accounting for positive and negative accidental torsion, and two were in the East-West direction, accounting for accidental torsion in that direction. The seismic loads may be seen below in Table 7 and Table 8.

Diaphragm	Story Force (kips)	Adjustment	Adj Story Force (kips)	Story Shear (V_i) (kips)	Bx (ft)	5% Bx (ft)	Ax	Mz (ft-kip)
Level 1	25.77	0.53	13.62	13.62	98.00	4.9	1.0	66.8
Level 2	15.42	0.35	5.45	19.07	88.00	4.4	1.0	24.0
Level 3	18.49	0.41	7.50	26.57	79.50	3.975	1.0	29.9
Roof	8.79	0.40	3.49	30.07	83.50	4.175	1.0	14.6

Table 7: North-South Direction Seismic Loads

Diaphragm	Story Force (kips)	Adjustment	Adj Story Force (kips)	Story Shear (V_i) (kips)	By (ft)	5% By (ft)	Ax	Mz (ft-kip)
Level 1	25.77	0.53	13.62	13.62	113.00	5.65	1.0	77.0
Level 2	15.42	0.35	5.45	19.07	74.50	3.725	1.0	20.4
Level 3	18.49	0.41	7.50	26.57	75.50	3.775	1.0	28.4
Roof	8.79	0.40	3.49	30.07	80.00	4	1.0	14.0

Table 8: East-West Direction Seismic Loads

Structural Overview of the Pedestrian Bridge

A 100-foot long box truss pedestrian bridge connects the Peggy Ryan Williams Center to the adjacent Dillingham Center.

Foundation and Columns

The pedestrian bridge has a separate foundation system from that of the PRWC, in which its columns rest on a 5'-0"x13'-0"x1'-6" footing.

The columns take on a hexagonal shape, roughly 11'-0"x3'-6". They are constructed of concrete with #8 vertical reinforcement and various #4 rebar ties. Figure 8 below shows the bridge column detail.

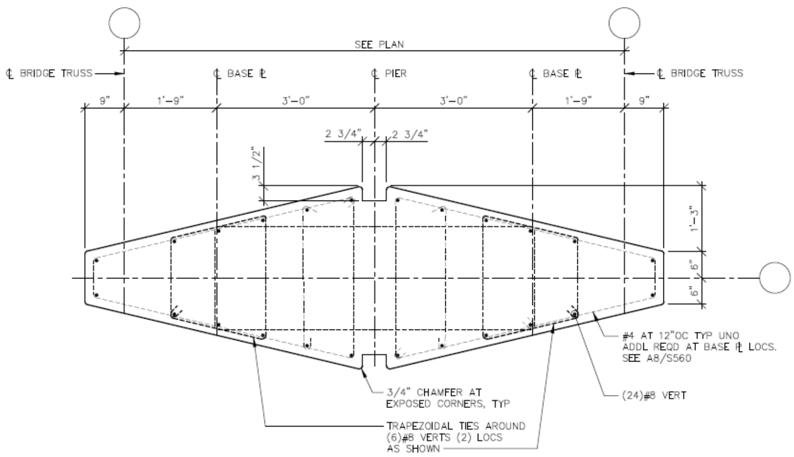


Figure 8: Bridge Column Detail
Drawing S560

Structural Framing

The pedestrian bridge is a box truss which is constructed using various hollow structural steel shapes and pipes. The top and bottom chords are both framed with HSS12x6x3/8 and the horizontal and diagonal braces are typically HSS4x4x1/4. The two side Pratt trusses have HSS5x5x5/16 vertical members and 3.5" pipe diagonal braces. There is a 2" expansion joint on either end of the bridge. This allows for expansion and contraction of the bridge due to variations in temperature. Figure 9 below shows the bridge truss schematic.

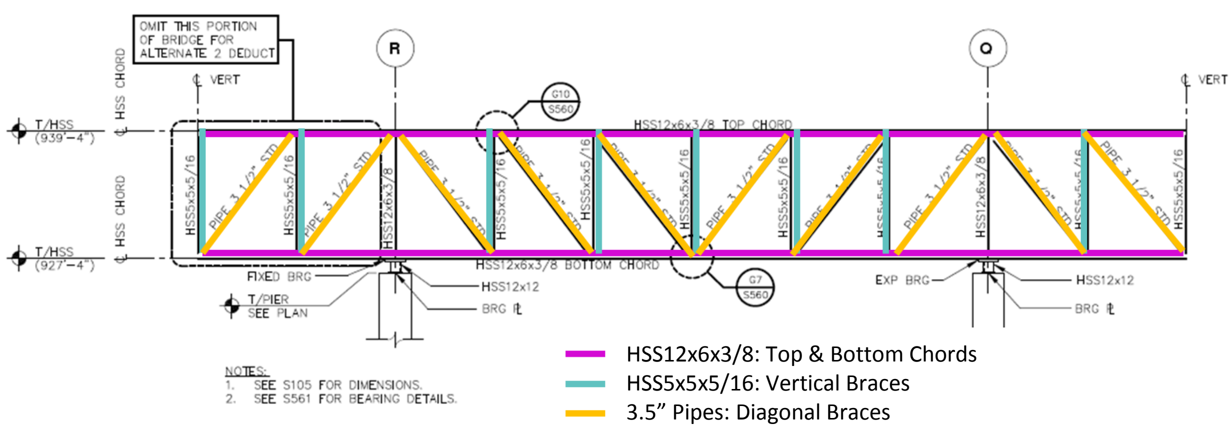


Figure 9: Bridge Truss Schematic
Drawing S560

Problem Statement

As previously stated, the steel structure of the Peggy Ryan Williams Center meets strength and serviceability requirements. The steel system was a good solution for dealing with the irregular geometry of the building and its floor openings. However, since a scenario has been created in which the schedule for the project is no longer critical, a reinforced concrete system may also prove to be a good design for the building. The concrete system would prove to be beneficial when it comes to the cantilevers since steel moment connections add significant cost to a project. A post-tensioned concrete slab design was explored in Technical Report 3. The system was found to be beneficial in terms of the floor system depth. However, the region of the building designed using that system had the building's longest spans. Therefore, when it is taken into account that the east end of the building contains much smaller spans, a post-tension slab is not the best solution. Instead, a one way concrete slab system with pan joists and girders will be designed. The pan joist system will better accommodate the varying spans. By changing the structural system to reinforced concrete, the lateral system will also need to be redesigned. Because the building is only four stories, the concrete gravity system may act as the lateral system.

The existing structure of the pedestrian bridge is a box truss comprised on Pratt trusses on either side. In order to create a learning opportunity, the bridge structure will be redesigned. As previously discussed, there are two options which will be considered for the bridge redesign. The first option is a reflection of New York's historical covered bridges, in particular that of the Newfield Bridge. The second option for the bridge redesign reflects on the original name of the building, "The Gateway Building" by mimicking the aesthetics of the Golden Gate Bridge.

Proposed Solution

Because the schedule is no longer critical, the structure of the building will be redesigned using reinforced concrete. For reasons previously stated, a one way concrete slab system with pan joists, girders, and columns will be designed. This system appears to be a good choice for the irregular geometry of the building because it lends itself to the varying bay sizes and the cantilevers. The various floor openings would also not cause problems with this system. A thinner slab can be used because it only needs to span the short distance between the pan joists. The pan joists will run in the direction of the existing beams of the structure. In turn, the girders will be located where the existing girders are located. This will minimize the architectural effects within the building due to columns' locations not changing. The floor system will first be designed through the use of computer programs such as spSlab and spBeam. Time permitting, the design will then be checked by hand. Because the building is only four stories, this gravity system may also work as the lateral system for the building. All structural framing members will be designed using ACI318-11.

In order to provide a learning opportunity, two different redesigns of the pedestrian bridge structure will be considered. Early on in the spring semester, sketches will be done to determine which redesign will best fit the existing site and its adjacent buildings. The first option is a reflection of New York's historical covered bridges, in particular that of the Newfield Bridge. For this redesign, the bridge supports will be moved closer to either building creating a longer span to give the illusion of the bridge only being supported by either building. A steel Warren truss will then be designed. The façade of the bridge will lend itself to an architectural breadth in which the façade will reflect on the covered bridge concept while incorporating some of the materials of the façade of the Peggy Ryan Williams Center. The second option for the bridge redesign reflects on the original name of the building, "The Gateway Building." This redesign will reflect upon the Golden Gate Bridge. Two towers (similar to those of the

Golden Gate Bridge) will be designed near the location of the existing two supports. A box truss will then be designed to be suspended from the towers. This option also lends itself to an architectural breadth. Both of these options allow for the consideration of a study of the exterior lighting systems.

Breadth Topics

Architectural Breadth – Bridge Façade Redesign

By changing the structure of the pedestrian bridge, an architectural breadth will need to be performed. If the covered bridge option is chosen, the roof of the bridge will mirror that of a traditional covered bridge. However, various façade materials will be considered which incorporate the materials of the nearby buildings, especially those of the Peggy Ryan Williams Center. The Warren truss will lend itself to large diamond shaped windows on the façade of the bridge. These windows will not only mirror the lattice truss of the Newfield Bridge; but, also, play off of the angles of the roof of the Peggy Ryan Williams Center. If the Golden Gate Bridge option is chosen, the façade of the bridge will most likely remain entirely glass. The appearance, placement, and materials of the towers will need to be taken into consideration. In order to explore these options, hand sketches will be completed. A Revit model of the chosen design will then be created and rendered.

Lighting Breadth – Exterior Lighting of the Bridge

In order to complement the structural redesign of the bridge and the architectural breadth, an exterior lighting breadth will be performed. Use of such techniques as wall washers will be investigated in order to create a modern façade that will complement its surroundings. Luminaires will then be selected. Revit and lighting software will be used to perform a rendering of the new lighting design.

Structural Depth

Gravity System of the Building

Through the use of concrete, numerous columns were able to be removed from the original design. The number of girders was also greatly reduced. The same depth was used throughout (24-½”) the floor system for a more economical and constructible design. The original system depth was 30-⅛”; therefore, the floor system depth was decreased by 6-⅝” throughout the building. This allowed for a slightly larger floor-to-ceiling height, thus opening up the interior spaces of the Peggy Ryan Williams Center. Due to time constraints only the level one framing was designed; however, the framing layout was drafted for level 2, level 3, and the roof level. These layouts may be viewed in Appendix A.2: Framing Layouts.

Pan Joist System

Because the pan joist system determined the depth of the floor system used throughout the building, it was designed first. The design was completed though the use of the *Concrete Reinforcing Steel Institute Design Handbook 2008 (CRSI)*. In order to be economical, the same joist size and spacing was used throughout the entire floor system. That allowed for the reuse of formwork. As seen in Table 9 below, two locations were considered in determining the joist size and reinforcement. Those two locations were found to have the worst loading and span conditions. It should be noted that all of the joist load capacities provided in the *CRSI* table have previously been investigated for deflection. Also, due to the footnote at the bottom of the table, additional deflection calculations did not need to be completed. It was determined to use 30” forms with 6” ribs at 36” on center. The required rib depth was found to be 20” with a 4.5” top slab, thus producing a 24.5” total system depth. The 24.5” depth was then continued throughout the entire floor system of the building. Using concrete with a compressive strength equal to 4000 psi and steel with a yielding strength of 60000 psi, the required top reinforcement was #5 bars at 8” and the required bottom reinforcement was (1) #6 bar and (1) #7 bar per rib. Because the slab only spans the 30” between ribs, only minimum reinforcement was required. Therefore, in the direction perpendicular to the joist span, the slab utilizes #3 bars at 12”. The pan joist system design may be viewed in Figure 10 below.

Location	Span (ft)	Live Load (psf)	Dead Load (psf)	Factored Loading (psf)	Top Reinf	Bottom Reinf
D3-D4	35	100	21	185.2	#5 @ 9"	#6 & #7 per rib
D4-D5	31.5	100	99	278.8	#5 @ 8"	#6 & #6 per rib

Table 9: Locations Considered for Pan Joist Design

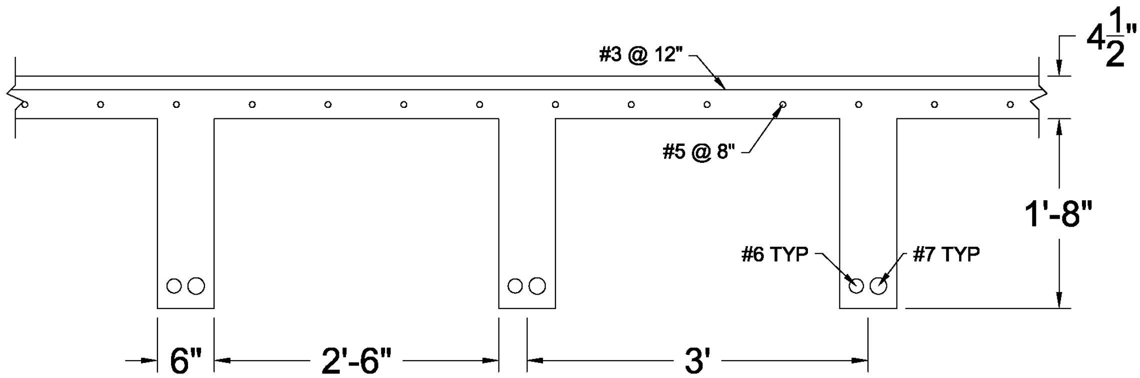


Figure 10: Pan Joist System Design

Girder Design

Three frames were chosen to be designed from the level one framing. Those frames were considered to be the most critical cases due to their long spans, large tributary widths, and high loadings. The girders were designed using spBeam. The predetermined dead and live loads were applied. An area load was also applied to account for the weight of the pan joists. Once the girders were designed for strength requirements, deflections were checked per ACI 318-11 Table 9.5b. The girder along column line 13 failed in deflection. In order to account for this, the width of the girder was increased slightly. The detailed designs for the girders along column lines 2, 8, and 13 may be viewed in Appendix A.3: Girder Designs. All of the stirrups are composed of two legs of their respective sizes, unless otherwise noted. The spBeam output is available in Appendix A.4: spBeam Output for Girders.

Beam Design

Beams were laid out along column lines D, F, and L in order to transfer the column loads from above back away from the cantilever and into the main structure of the building. The beam along column line D was designed due its large span. To be conservative in the design and not count on the dead load of the floor system to resist the uplift from the cantilever, the beam was designed with only considering the point load on the end of the cantilever and the member's own self weight. The assumed column loads from above first had to be carried down through the building and to the first level. This calculation was done using an excel sheet and may be viewed in Appendix A.5: Assumed Column Loads From Above. Once the beam was designed for strength requirements using spBeam, deflections were checked per ACI318-11 Table 9.5b. The cantilever failed in deflection. To provide better serviceability, compression reinforcement was added. Without the compression reinforcement, the beam would have needed to be over seven feet wide. The beam design may be viewed in Figure 11 below. Top reinforcement and stirrups were used in order to support the loaded cantilever. All of the stirrups are composed of two legs of their respective sizes. The spBeam output is available in Appendix A.6: spBeam Output.

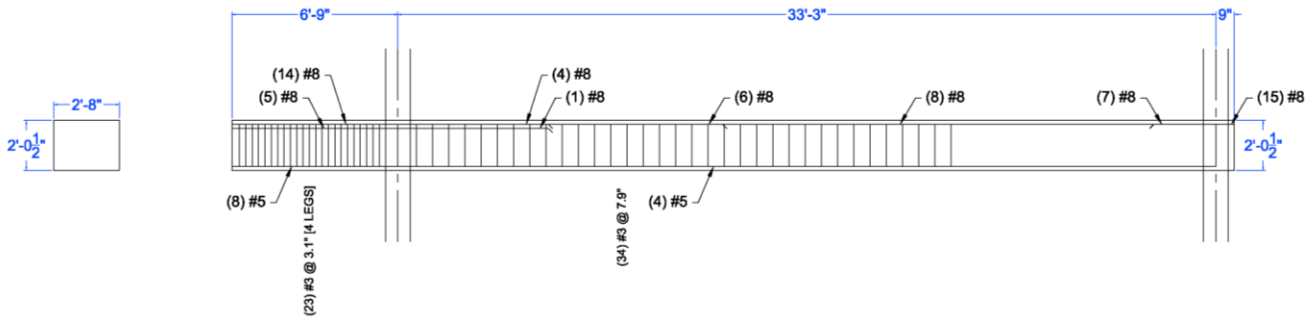


Figure 11: Beam Design for Column Line D

Column Design

The columns on column lines 2, 8, and 13 were designed using spColumn. The assumed loadings from above, as well as the loads from the floor system of level one, were applied to the level one columns. A Microsoft Excel sheet was utilized to determine the loads applied to each column, including both axial loads and moments. The Excel sheet may be viewed in Appendix A.7: Load to Apply to Level 1 Columns. A square section was chosen for the columns for ease of construction and to aid with the future lateral system design. Constructability was considered when designing the columns. The reinforcement was kept as #6, #8, and #10 bars. Equal spacing was also kept between bars. The column designs for frame 2 may be viewed below in Figure 12, while the remaining designed columns may be seen in Appendix A.8: Column Designs. The spColumn output is available in Appendix A.9: spColumn Output. For constructability, all column sizes were increased to be an 18" x 18" section. This increase in size is shown in the lateral system of the building.

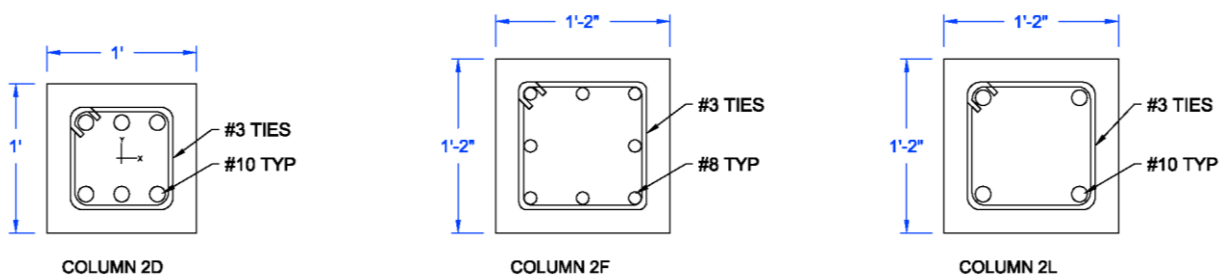


Figure 12: Column Designs for Column Line 2

Final Gravity Framing

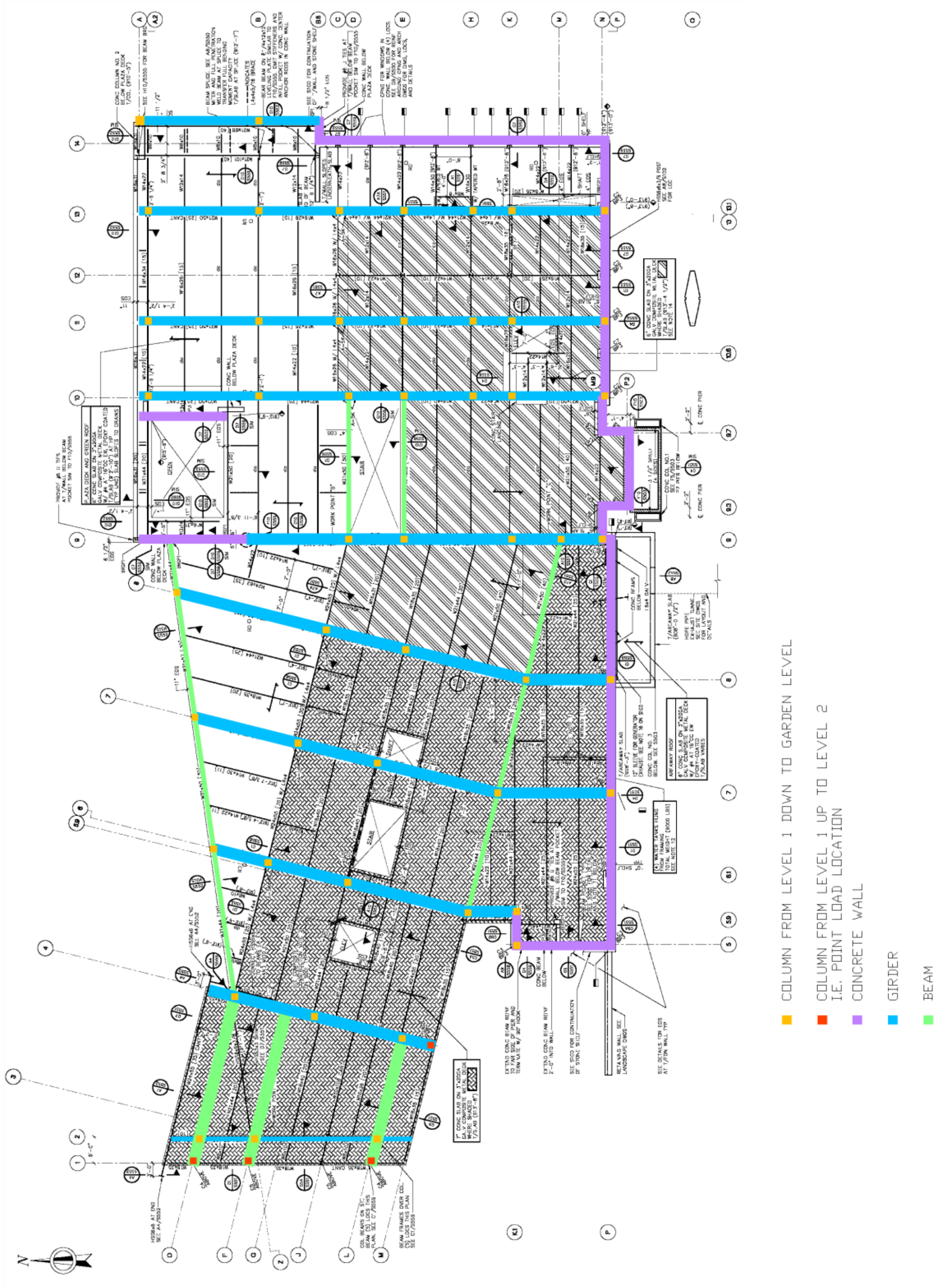


Figure 13: Gravity Framing for Level 1

Lateral System of the Building

The existing lateral system of the building consists of concentrically braced structural steel frames in both the North-South and East-West directions. By changing the building material to concrete, the braced steel frames were no longer the best option for the lateral system. As previously mentioned, since the building is only four stories, the gravity system had the potential to also perform as the lateral system for the building. The columns and girders would act as frames in the North-South direction and the columns and joists would act in the East-West direction. Therefore, every column line would essentially act as a lateral resisting frame. Once the gravity system for level one was designed, the system was then checked at level one for adequacy in resisting the lateral forces on the building. To allow for ease of analysis, only four concrete moment frames were considered in each the North-South and the East-West direction. It was determined that if these frames were found to be adequate to resist the lateral loads, then by allowing all of the frames of the building to help resist the load, the system would surely be adequate. The concrete moment frames considered are shown in red in Figure 14.

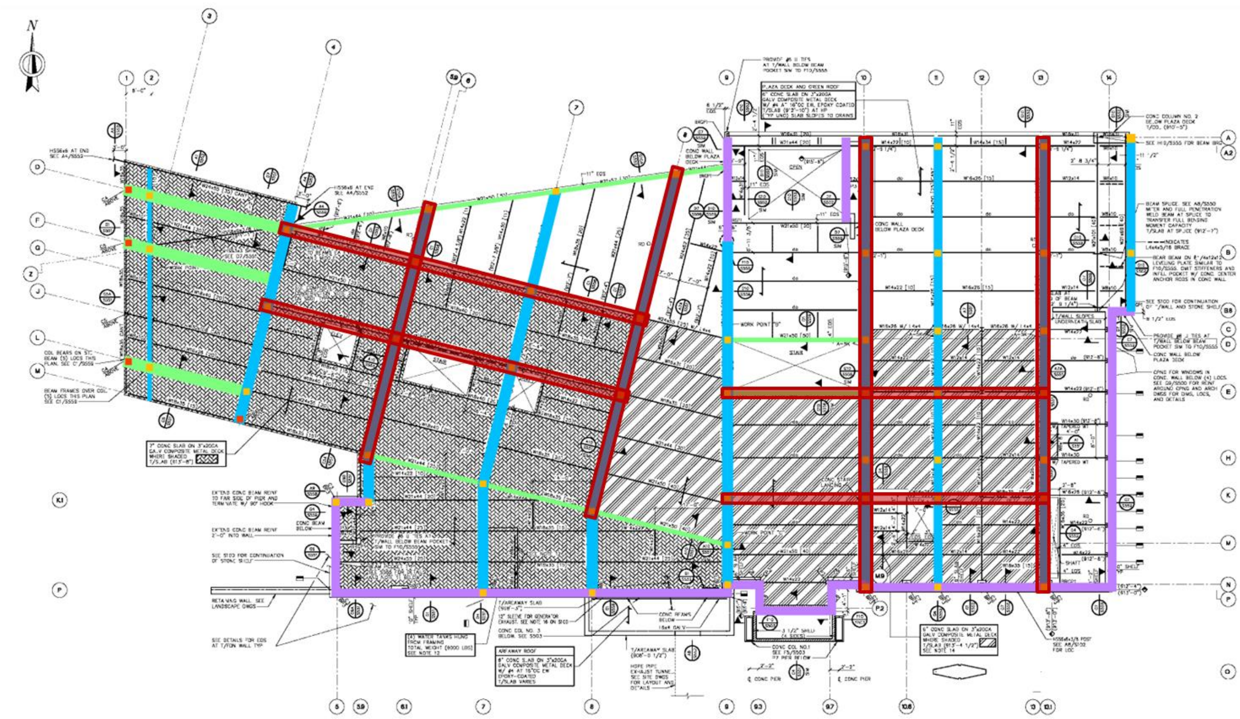


Figure 14: Framing for Level 1 Indicating the Frames Considered in Lateral Analysis

Lateral Load Determination

The existing lateral system of the building was governed by wind. However, since the building material was changed to concrete, both seismic and wind calculations needed to be completed to ensure that wind loads still controlled the design. ASCE7-10 was used to calculate the lateral loads. Because the building is Seismic Design Category A, the simplified procedure was able to be used, in which Fx = 0.01Wx. The wind loads were calculated using the Main Wind Force Resisting System (MWFRS) procedure. Through various hand calculations, which may be seen in Appendix B.1: Lateral Load Calculations, it was determined that wind loads still controlled the building's lateral system design. A summary of the seismic and wind loads may be seen below in Table 10, Table 11, and Table 12.

Seismic Load Base Shear			
	Force (k)	Story Shear (k)	Overturning Moment (ft-k)
Level 1	45.0	132.0	599.9
Level 2	29.0	87.0	773.5
Level 3	31.0	58.0	1240
Roof	27.0	27.0	1869.8
Total	132.0		4483.2

Table 10: Summary of Seismic Loads

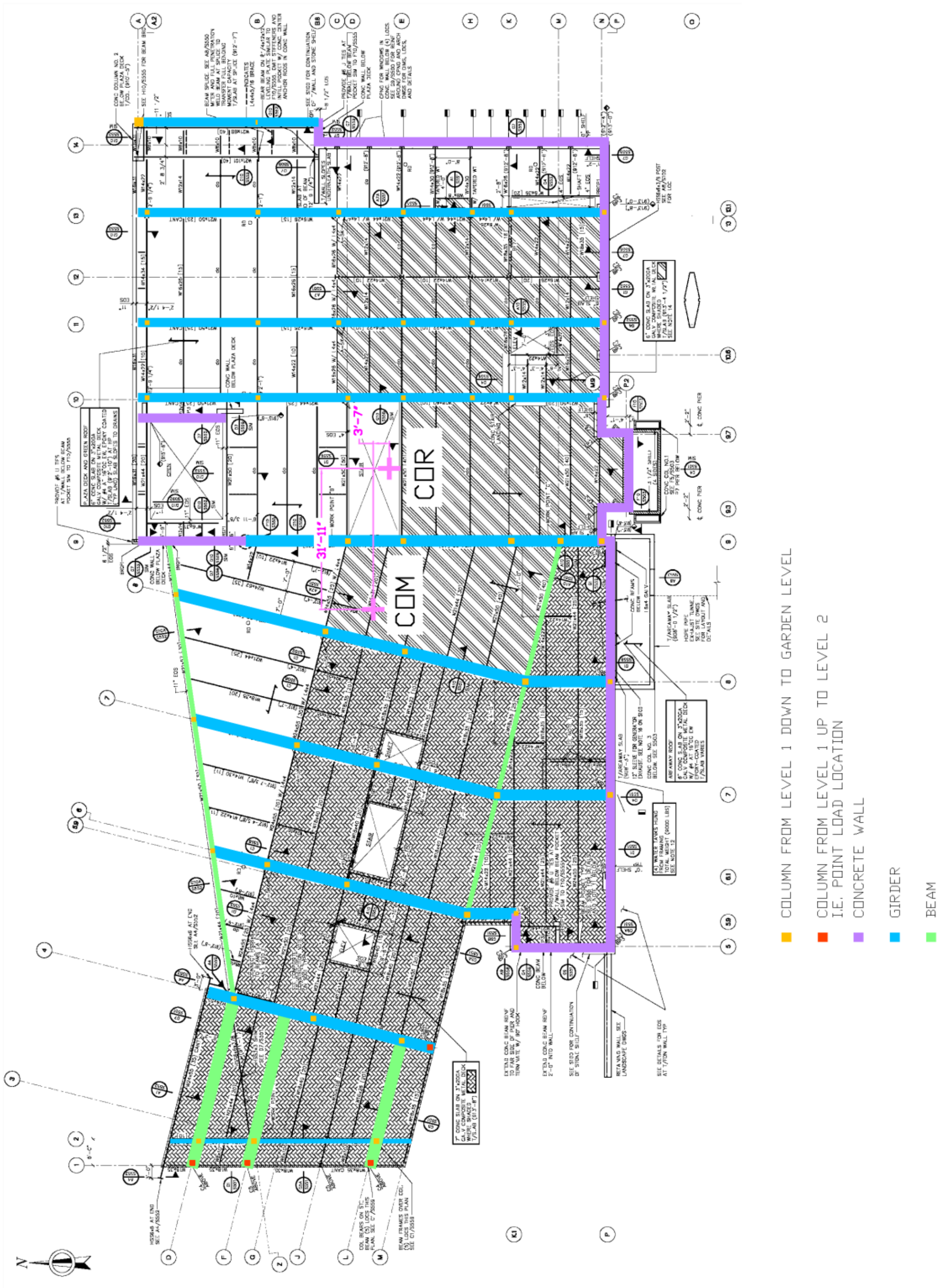
Wind Load Base Shear & Overturning Moment			
	Force (k)	Story Shear (k)	Overturning Moment (ft-k)
Garden Level	36.6	463.7	0.0
Level 1	73.2	427.0	976.4
Level 2	80.2	353.8	2139.8
Level 3	136.8	273.6	5471.3
Roof	136.8	136.8	9472.8
Total	463.7		18060.3

Table 11: Summary of Wind Loads in the North-South Direction

Wind Load Base Shear & Overturning Moment			
	Force (k)	Story Shear (k)	Overturning Moment (ft-k)
Garden Level	14.1	185.9	0.0
Level 1	28.1	171.9	374.5
Level 2	31.5	143.8	838.8
Level 3	54.4	112.3	2175.3
Roof	57.9	57.9	4011.7
Total	185.9		7400.3

Table 12: Summary of Wind Loads in the East-West Direction

Each of the frames being considered in the design were then modeled in RISA-2D. Through the use of member properties and by applying a 10 kip "dummy load" the stiffness of each frame was determined. Appendix B.2: Determination of Frame Stiffness's may be referenced for the stiffness calculations. Those stiffness's could then be used to calculate the center of rigidity of level one. Figure 15 below shows where the center of mass and center of rigidity are located on level 1. Detailed calculations of these centers may be seen in Appendix B.3: Center of Mass and Center of Rigidity.



Center of Rigidity		
Xr =	158.59	ft
Yr =	51.07	ft

Center of Mass	
X - Direction	126.7
Y - Direction	54.7

Figure 15: Center of Mass and Center of Rigidity of Level 1

Through the use of stiffnesses, the level one story shear was then distributed to each of the frames for all six wind load cases, including both positive and negative moments. Both direct shear and torsional shear were considered. Excel sheets containing these calculations are available in Appendix B.4: Wind Load Cases. Based on the total shear force, a worst load case was found for each frame. A summary of these results may be viewed in Table 13 below.

Determination of Worse Case on Each Frame

	Frame	WC 1: N-S	WC 1: E-W	WC 2: N-S + 0.15 By	WC 2: N-S - 0.15 By	WC 2: E-W + 0.15 Bx	WC 2: E-W - 0.15 Bx	WC 3: NS + EW	WC 4: (NS + 0.15By) + (EW + 0.15Bx)	WC 4: (NS + 0.15By) + (EW - 0.15Bx)	WC 4: (NS - 0.15By) + (EW + 0.15Bx)	WC 4: (NS - 0.15By) + (EW - 0.15Bx)	Worst Case Shear (kips)	Worst Case	Deflection Under Worst Case (in)
North-South	6	162.8	3.6	185.5	58.7	14.9	-9.5	124.8	150.4	132.1	55.3	36.9	185.5	WC 2: N-S + 0.15 By	0.58900
	8	112.7	1.4	109.6	59.4	5.9	-3.8	85.6	86.7	79.4	49.0	41.8	112.7	WC 1: N-S	0.37400
	10	125.1	-1.1	74.6	113.0	-4.5	2.9	93.0	52.6	58.2	81.5	87.0	125.1	WC 1: N-S	0.23900
	13	63.1	-3.9	-21.9	116.6	-16.3	10.4	44.4	-28.6	-8.6	75.3	95.3	116.6	WC 2: N-S - 0.15 By	0.22300
East-West	D	14.9	46.0	23.1	-0.9	36.8	32.2	45.6	45.0	41.5	27.0	23.5	46.0	WC 1: E-W	0.23900
	E	-2.0	48.7	-3.2	0.1	36.2	36.9	35.0	24.8	25.3	27.3	27.8	48.7	WC 1: E-W	0.21600
	G	4.1	45.5	6.4	-0.2	34.8	33.5	37.2	30.9	29.9	25.9	25.0	45.5	WC 1: E-W	0.23700
	K	-16.9	45.7	-26.3	1.0	31.6	36.9	21.6	4.0	7.9	24.5	28.4	45.7	WC 1: E-W	0.22800

Worst Deflection (North-South)	0.58900	inches
Worst Deflection (East-West)	0.23900	inches

Table 13: Determination of Worst Case Wind on Each Frame

Moment Frame Design

Using RISA, the worst case wind load for each frame was applied to their respective frames and their deflections were measured. The deflections may be seen in Table 13 above. Per ACI318-11, various columns were checked to see if they were sway or nonsway. In order to be conservative, when determining if the columns were sway, the worst case deflection was used for the story deflection. As seen in Table 14 below, all of the columns that were checked were found to be nonsway.

Sway vs. Nonsway							
Frame	Column	∑Pu (kip)	Δ (in)	V_{us} (kip)	L_c (in)	Q	Sway/ Nonsway
8	Z	168	0.589	463.7	160.0	0.0013	Nonsway
8	D	545	0.589	463.7	160.0	0.0043	Nonsway
8	G	580	0.589	463.7	160.0	0.0046	Nonsway
8	M	681	0.589	463.7	160.0	0.0054	Nonsway
8	P	247	0.589	463.7	160.0	0.0020	Nonsway
13	A.2	143	0.589	463.7	160.0	0.0011	Nonsway
13	B	250	0.589	463.7	160.0	0.0020	Nonsway
13	C	296	0.589	463.7	160.0	0.0023	Nonsway
13	E	256	0.589	463.7	160.0	0.0020	Nonsway
13	H	207	0.589	463.7	160.0	0.0016	Nonsway
13	K	290	0.589	463.7	160.0	0.0023	Nonsway
13	N	226	0.589	463.7	160.0	0.0018	Nonsway
D	8	545	0.239	185.9	160.0	0.0044	Nonsway
E	13	256	0.239	185.9	160.0	0.0021	Nonsway
G	8	580	0.239	185.9	160.0	0.0047	Nonsway
K	13	290	0.239	185.9	160.0	0.0023	Nonsway

NOTE: The deflections given above are based on the worst case deflection due to wind in the direction in which the frame acts.

Equation Used: $$Q = \frac{\sum P_u * \Delta}{V_{us} * L_c} \leq 0.05 \rightarrow \text{Nonsway}$$

Table 14: Determination of Sway vs. Nonsway

The moments and axial forces on the columns, as a result of the worst case wind load being applied to their respective frames, were then taken into spColumn to complete the column designs. These reactions may be seen in Appendix B.5: Column Lateral Loadings to be Used in spColumn Analysis. Because RISA and spColumn do not use the same sign convention, the member forces provided by RISA were changed to match the sign convention used by spColumn. Because some of the axial forces in the columns were tension forces, which would counteract the gravity compressive forces, those tension forces were excluded from the spColumn analysis.

As previously mentioned, the gravity column sizes were all increased to 18" x 18" for constructability. This new size was used in the lateral system analysis. Because the columns were nonsway, slenderness did not need to be considered. The concrete was also cracked for the analysis. Each of the designed columns were checked for biaxial bending. This was particularly important since several of the designed columns participated in both a North-South frame and an East-West frame. However, the two reactions were not added. Since only one wind load case would occur at a time, the load cases were investigated separately. The final column designs may be seen in Figure 16 below. spColumn output is available in Appendix B.6: spColumn Output for Final Column Designs.

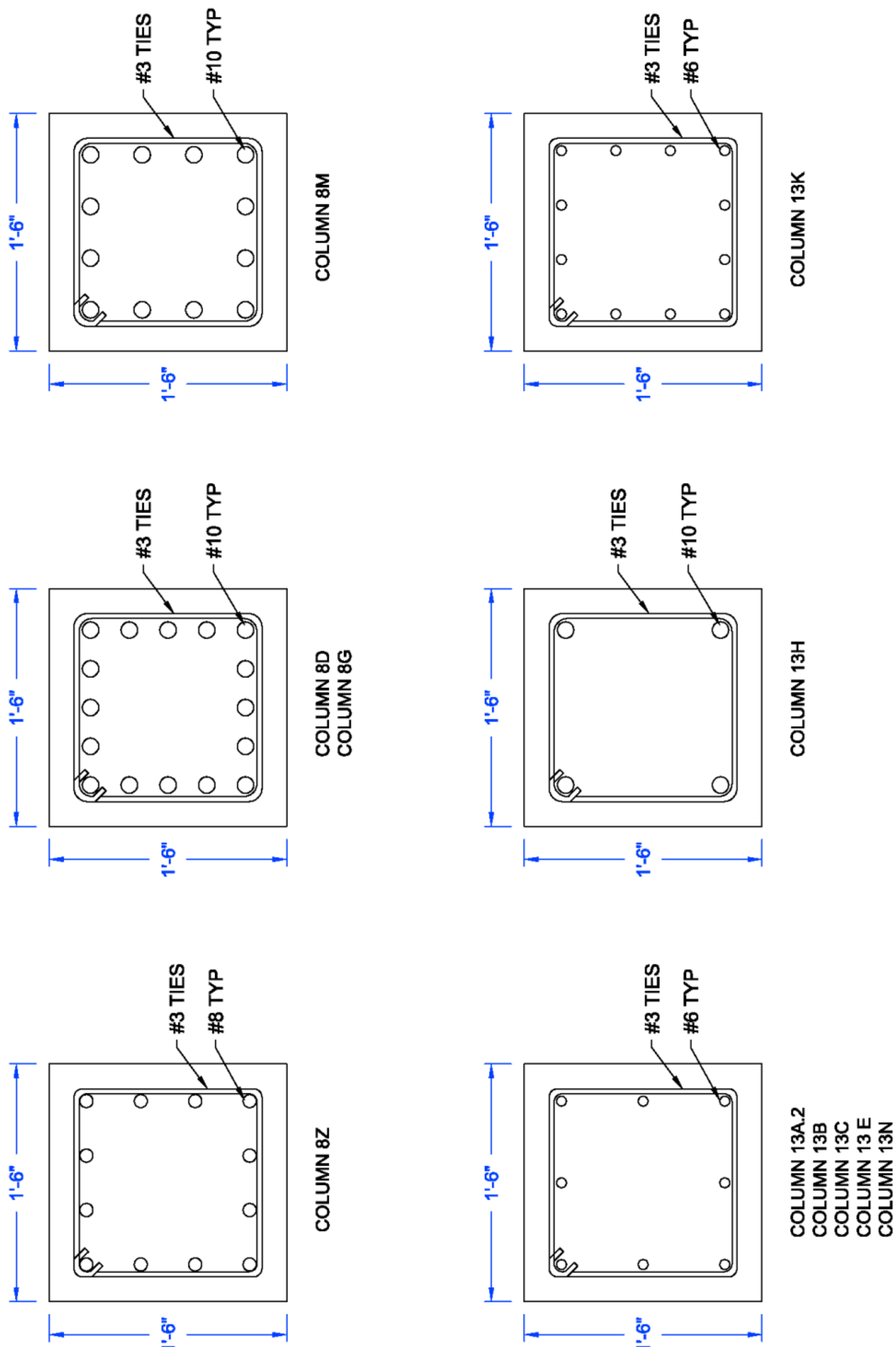


Figure 16: Final Column Designs

To ensure that the frames were adequate at resisting the lateral forces, a worst case girder and joist were checked for beam-column interaction. Through hand calculations, it was determined that both the joist and the girder were adequate. The interaction calculations may be viewed in Appendix B.7: Beam-Column Interaction Calculation.

In conclusion, the gravity system of the Peggy Ryan William Center also acts as the building's lateral system. The final gravity column size of 18" x 18" was used. The previously designed girders and joists were also found to be adequate in acting as the lateral system.

Gravity System of the Pedestrian Bridge

Early on in the semester, sketches were produced to determine if the bridge should play off of a historic covered bridge or the Golden Gate Bridge. Through reasoning that may be viewed in the Architectural Breadth, the covered bridge option was chosen. Therefore, the redesign consisted of a box truss similar to the existing configuration. However, the two bridge supports were moved out to open up the space and more closely mimic a covered bridge. A Warren truss was used in the design. This particular truss was chosen for architectural reasons. Because the gravity loads dictate the two side trusses, those two trusses were designed first.

Gravity Loads

Gravity Loads were first calculated which included dead loads, live loads, snow loads, and snow drift loads. These load calculations are available in Appendix C.1: Gravity Loads on the Bridge. A layout then had to be chosen for the truss before panel point loads could be determined. The chosen layout may be viewed below in Figure 17 below. Layouts were also developed for the top and bottom trusses. Their layouts were based on the side Warren Truss design. Extra members were also added in order to ensure that the panel point loads were transferring into the proper locations. These layouts may be viewed in Appendix D.1: Bridge Trusses.

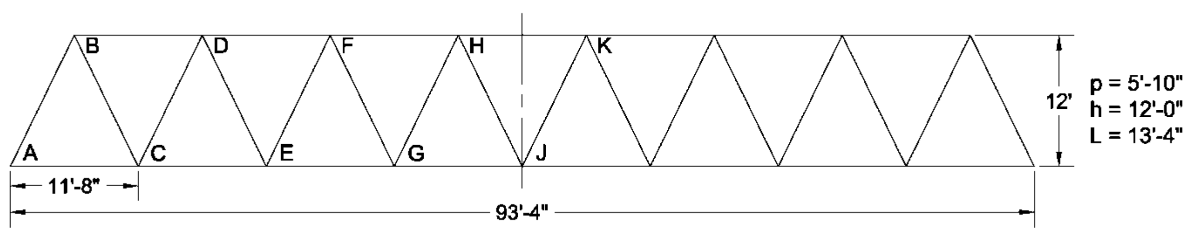


Figure 17: Truss Layout

Once the above layout was chosen, the panel point loads were calculated. The panel point load calculations may be viewed in Appendix C.2: Determination of Panel Point Loads and the final loads may be viewed in Appendix C.3: Panel Point Loads. Because the bridge must meet the requirements of ASCE7-10, the dead, live, and snow loads were kept separate so that load combinations could be considered. In the design of the two side trusses, only the gravity loads needed to be considered. The lateral loads would be taken by the top and bottom trusses, which will be designed time permitting. In considering gravity loads, only three different load combinations required consideration: 1.4 D; 1.2 D + 1.6 L + 0.5 S; and 1.2 D + 1.6 S + L. It was decided to factor the loads and then apply them to the truss. The loading conditions for these three load combinations are available in Appendix C.4: Panel Point Load Combinations. By factoring the loads first, it was determined that 1.4 D did not control the design.

Member Forces

Using the Indexing Method, the index for each member was then determined. The indices for the two different load combinations may be viewed in Appendix C.5: Member Indices. The indices, which are the vertical forces in the members, were then converted into axial forces through the use of geometry. The conversion may be seen in Appendix C.6: Conversion of Indices to Member Forces. The resultant forces may be viewed below in Figure 19 and Figure 20. A color coding key is available for reference in Figure 18 below.

Dead Loads
Live Loads
Snow Loads
Load Combination Loads
Index
Member Force

Figure 18: Color Coding Key

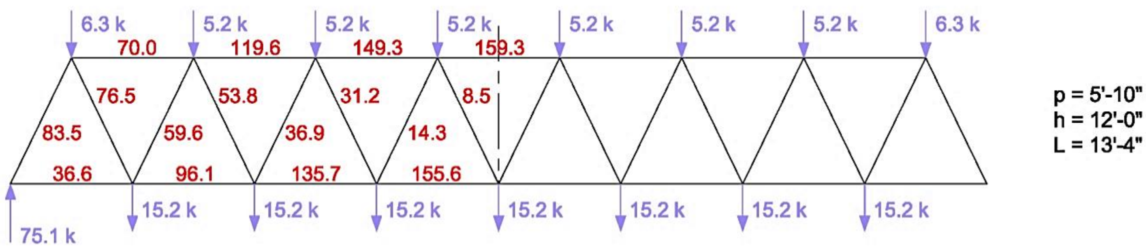


Figure 19: Member Forces for 1.2 D + 1.6 L + 0.5 S

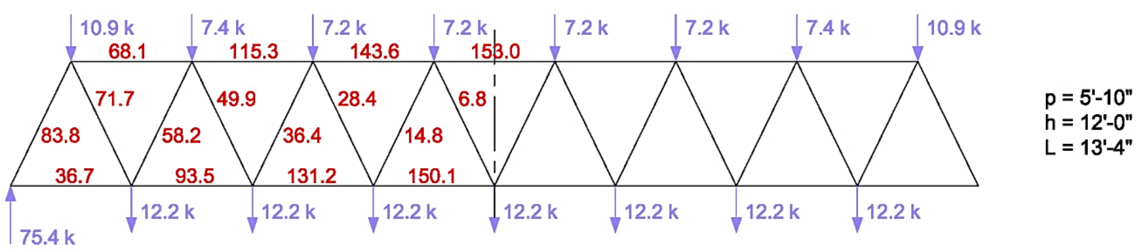


Figure 20: Member Forces for 1.2 D + 1.6 S + L

The member forces were then verified using the Method of Joints; those calculations may be viewed in Appendix C.7: Method of Joints. As seen below in Table 15, it was found that the Indexing Method is an accurate method of analysis.

Verify Forces for: 1.2 D + 1.6 L + 0.5 S			
Member	Indexing Method (k)	Method of Joints (k)	Variation (k)
AB	83.5	83.5	0.0
AC	36.5	36.6	-0.1
BC	76.5	76.5	0.0
BD	69.9	70.0	-0.1
CD	59.6	59.6	0.0
CE	96.0	96.1	-0.1
DE	53.8	53.8	0.0
DF	119.5	119.6	-0.1
EF	36.9	36.9	0.0
EG	135.6	135.7	-0.1
FG	31.1	31.2	-0.1
FH	149.2	149.3	-0.1
GH	14.2	14.3	-0.1
GJ	155.4	155.6	-0.2
HJ	8.4	8.5	-0.1
HK	159.1	159.3	-0.2

Table 15: Comparison Between the Indexing Method and the Method of Joints

Member Design

By loading all of the panel points (producing a uniform load), the worst case force in both the top and bottom chord was determined. Because the far left diagonal is a compression member, loading all of the panel points, also determined the worst case force in the diagonals. Since the top chord is a compression member, Table 4-4 of the Steel Manual was used to determine the size of the member. An HSS7x7x¼ was chosen for the top chord based on the worst axial force in the chord. For aesthetic purposes, the same size HSS was desired for the bottom chord. Using Table 5-5 of the Steel Manual, which is for the design of tension members, it was found that an HSS7x7x$^3/_{16}$ meets the strength requirements for the design of the worst case axial force in the chord. However, for constructability, an HSS7x7x¼ was chosen for the bottom chord. For the diagonals, it was desired to have an HSS size that was approximately half the size of the top and bottom chord. This would allow for a nice aesthetic of the truss. Therefore, using Table 4-4 of the Steel Manual, an HSS4x4x½ was chosen for the diagonals based on the worst case force in the far left diagonal. The design summary may be viewed in Appendix C.8: Member Design. The final truss design may be viewed in Figure 21 below.

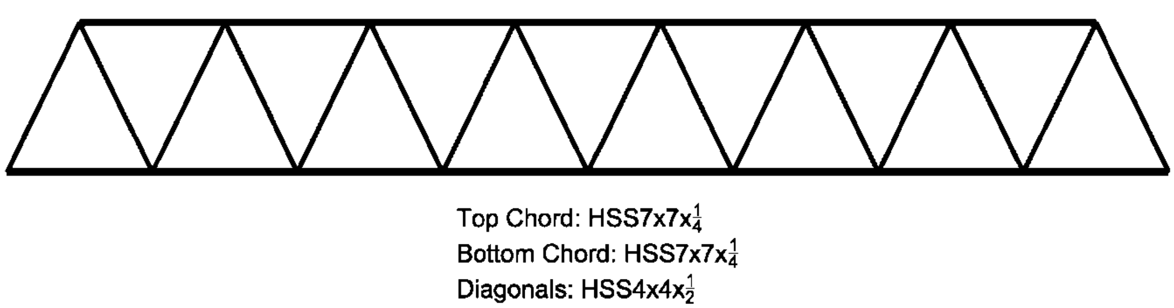


Figure 21: Final Design of the Side Trusses

Architectural Breadth – Bridge Façade Redesign

Initial Sketches

In the beginning of the semester, sketches were done to determine which redesign concept would be chosen. The first sketch was the covered bridge option. This option was inspired by New York's historical covered bridges, in particular that of the Newfield Bridge. The Newfield Bridge was built using a lattice truss. That truss type created an interesting diamond pattern on the interior, which may be seen in Figure 22 below. It was desired to mimic this diamond pattern in the bridge redesign. Therefore, if this redesign would be chosen, a truss type would be selected that allows for the incorporation of the diamond pattern. The covered bridge redesign would also consist of moving the supports closer to either adjacent building, thus creating a longer span to give the illusion of the bridge only being supported by either building. A gable roof would also be considered if this redesign option was chosen. The façade would incorporate some of the materials of the façade of the Peggy Ryan Williams Center. The first sketch of the covered bridge redesign concept may be seen in Figure 23 below.

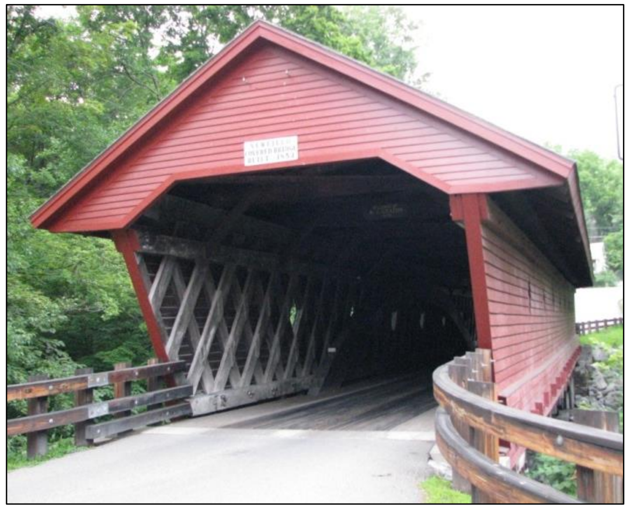


Figure 22: The Newfield Bridge | *Photo taken 07-31-13*

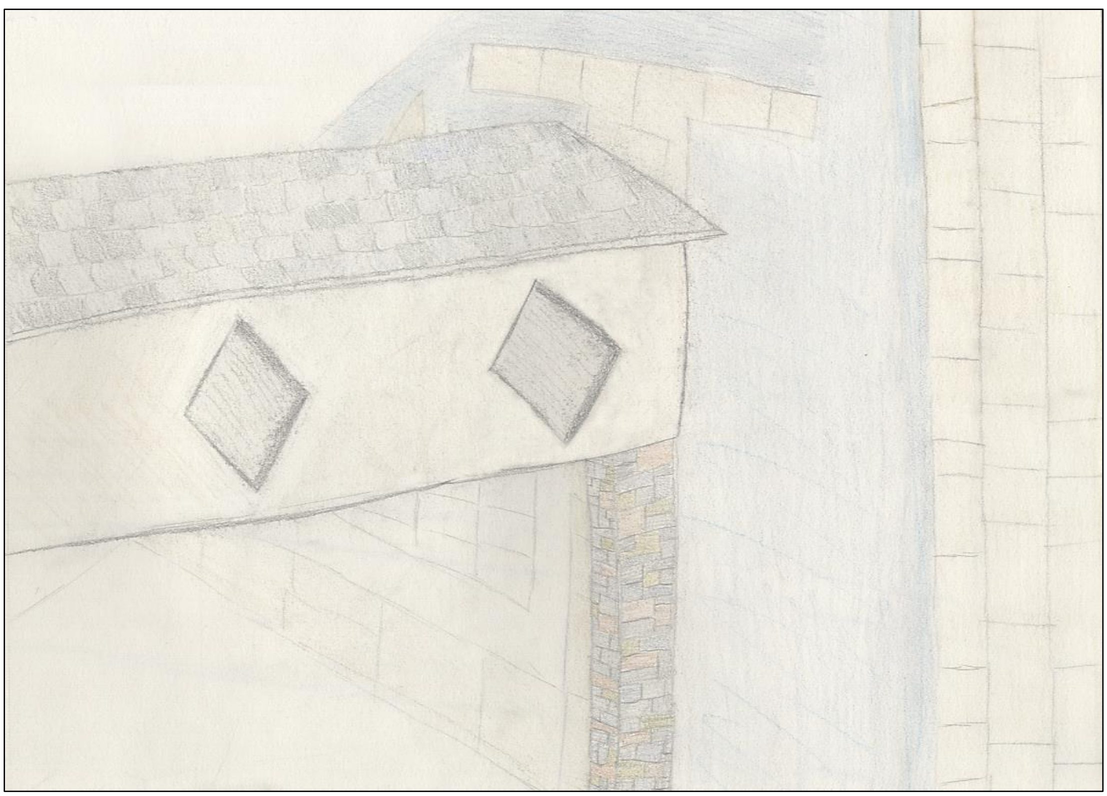


Figure 23: Covered Bridge Redesign Concept

The second redesign option reflected on the original name of the building, “The Gateway Building.” Therefore, this redesign would reflect upon the Golden Gate Bridge. The redesigned box truss would be very similar to the original design, including a glass façade. However, instead of having the two original pier supports, the bridge would instead be suspended from two towers (similar to those of the Golden Gate Bridge). The towers would likely be located where the current supports reside. The size of the towers would need careful consideration so that they allow the bridge to stand out, yet not overpower the Peggy Ryan Williams Center. The second sketch, of the Golden Gate Bridge redesign concept, may be seen in Figure 24 below.

Figure 24: Golden Gate Bridge Redesign Concept

In comparing these two sketches, both redesigns would provide a good learning opportunity of bridge design while carefully considering the bridge’s impact on its surroundings. While the Golden Gate Bridge option would be very interesting and provide a learning opportunity of suspension bridges, its box truss would not change much, if at all, from the original design. That option may also not appear in harmony with the existing adjacent buildings, due to the two large towers. The covered bridge option would allow for a complete redesign of the box truss and more options for the façade of the bridge. Careful consideration would need to be taken to ensure that this bridge does not look out-of-place next to the existing buildings. Through completing the two sketches it became evident that the covered bridge option was desired. This option would provide a wide range of opportunities for the bridge redesign. Upon choosing the covered bridge option, a third sketch was done. That sketch shows a more complete view of the bridge and the adjacent buildings. With the use of careful material consideration, it was determined that this redesign would tie in nicely with the Peggy Ryan Williams Center. The third sketch may be viewed below in Figure 25.

Figure 25: Covered Bridge Option - Redesign Concept Chosen

Truss Design

As previously stated, in choosing the covered bridge redesign option, a truss would be selected that would achieve a diamond pattern. Originally, a Double Intersection Warren Truss was chosen for the redesign. The diamond pattern of that truss would mimic that of the lattice truss of the Newfield Bridge. However, it was decided that a Warren Truss would be used in the redesign. The Warren Truss would be more economical since the truss would require fewer steel members. In order to preserve the concept of mimicking the Newfield Bridge, an applique would be applied to the façade of the bridge that suggests that a Double Intersection Warren Truss is within. The diamond design applique would not only mimic the Newfield Bridge; but also, the diamond pattern ties into the irregular roof angles of the Peggy Ryan Williams Center. In order to allow natural light into the bridge, every other diamond of the bridge would be a large window. The trusses were created using Autodesk Revit 2014. The complete box truss may be viewed below in Figure 26. The individual side, top, and bottom trusses may be viewed in Appendix D.1: Bridge Trusses.

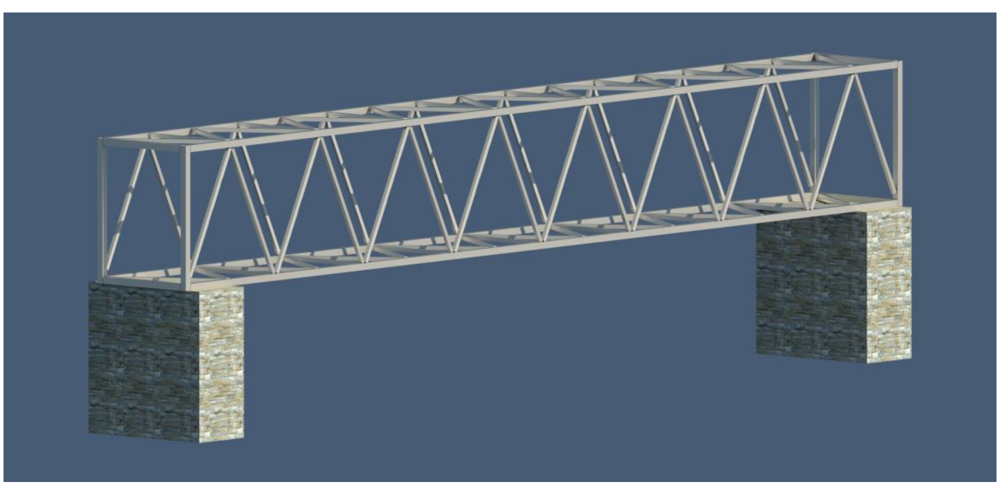


Figure 26: Bridge Box Truss

Gable Roof

Initially, the gable roof was chosen to more closely mimic a covered bridge. However, in order to attempt to not take the covered bridge idea too literal, other roof options were considered. A shed roof was taken into consideration. But, this option was eliminated due to both sides of the bridge being readily exposed to the public. Therefore, neither side would allow the shed roof design. A flat roof design was also considered. The downside of the flat roof design was that it appeared to be too similar to the original design. Also, the bridge did not give off the same vibe as it did with the gable roof. As a result, it was decided to use a gable roof for the bridge redesign.

Façade Diamond Design

The main façade of the redesigned bridge consists of a diamond pattern. This façade feature was inspired by the Yotel Building which is located in New York City. The façade of that building features various polygonal shapes which reflect the light and cast shadows when they are washed with light. Figure 27 below shows the Yotel Building's façade.

Figure 27: Yotel Building Façade | Photo Courtesy of ebayink

This type of façade was inspired by the decision to also complete a Lighting Breadth of the bridge. Various patterns and shapes were explored for the façade design. In the end, a diamond and half diamond pattern was chosen. This pattern would tie into the truss design and play off of the numerous irregular angles of the Peggy Ryan Williams Center. In order to avoid the façade becoming too busy, only alternating rows of diamonds have the half diamond extrusion. This pattern, like that of the Yotel Building, will allow light to be reflected and interesting shadows to be created when the façade is washed with light. An up-close render of the diamond pattern façade may be seen below in Figure 28.

Figure 28: Diamond Facade Pattern

Materials Chosen

Aluminum | Diamond Patterned Façade

- The aluminum ties in with the aluminum panels on the Peggy Ryan Williams Center
- Green tint was chosen to take the LEED status literally and to connect the bridge to the lush green landscape surrounding it.

Limestone | Lattice Applique

- The limestone lattice mimics the limestone panels on the PRWC.
- Limestone allows for a clear distinction of the lattice design.

Bluestone | Façade of the Supports

- Bluestone was used on the perimeter of the lower level of the PRWC. Therefore, by using it at the bottom of the bridge, it continues the pattern started by the building.
- Bluestone also helps the bridge supports to look like they are a part of the adjacent building and not simply there for the bridge.

Slate | Roof

- The slate again ties back to the bluestone used on both the building and the bridge.
- The slate also matches the overall color scheme and feel of the bridge's exterior.

Façade Comparison

The redesigned bridge was created using Autodesk Revit 2014. In the following four figures, the differences between the existing bridge façade and the redesigned bridge façade may be seen.

Figure 29: Existing Bridge Façade | Front View

Figure 30: Redesigned Bridge Façade | Front View

Figure 31: Existing Bridge Facade | Prospective View

Figure 32: Redesigned Bridge Facade | Prospective View

Architectural Breadth Conclusion

The architectural redesign of the bridge produced a design which is very different from the original design and commands attention, yet it is still in harmony with its surroundings. Through the redesign, a new truss type was used; the façade was designed for both architectural features and lighting features, and materials were selected to reflect upon the Peggy Ryan Williams Center. The bridge successfully mimics the Newfield Bridge, with its Warren Trusses and diamond design applique on the façade, without taking the covered bridge inspiration too literally.

Lighting Breadth – Exterior Lighting of the Bridge

As mentioned in the Architectural Breadth section, the façade of the bridge was inspired by the Yotel Building in New York City. In order to accomplish a similar lighting affect, the lighting fixture ColorGraze Powercore 30x60, manufactured by Philips Color Kinetics, was selected for the exterior lighting of the bridge. This luminaire was chosen for its high performance; including its linear form which enables the grazing of the façade and its superior ability at highlighting the texture of facades. Another reason for the selection of this luminaire was for its outdoor weather rating. The 30x60 was chosen because it allows for a more uniform effect. This product has been used in various successful façade projects such as the John E Jaqua Academic Center for Student Athletes at the University of Oregon. This luminaire has well defined color changing ability as well. When paired with an adequate control system, such as Philips LED Lighting Systems Controllers, the façade can be lit under colored light. This can become a pleasant visual element to the new bridge. In the following figures, the various lighting effects may be viewed. Figure 33 shows the original white LED. Figure 34 shows the façade grazed with green LED light, which further accents the green façade material of the bridge. Figure 35 illustrates a blue LED light graze, which is one of Ithaca College's school colors. The specification sheet for the luminaire is available in Appendix E.1: Luminaire Specification Sheet.

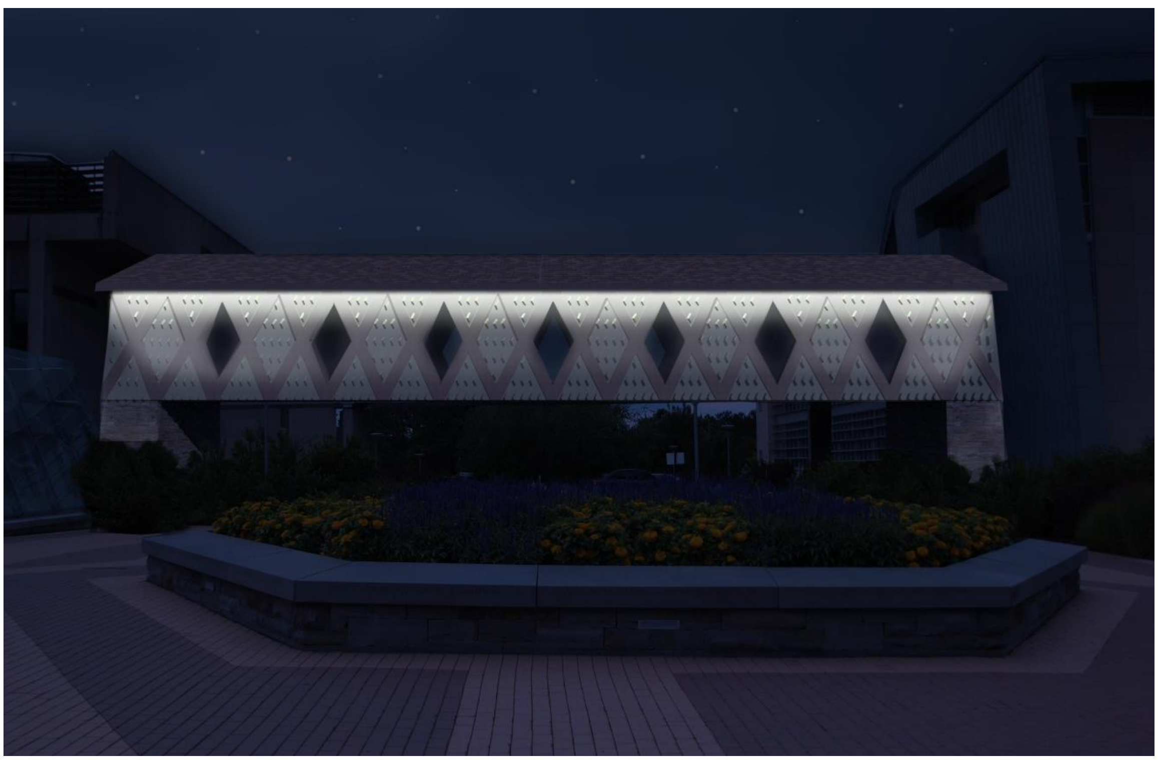


Figure 33: Exterior Lighting of the Bridge with Original White LED Light

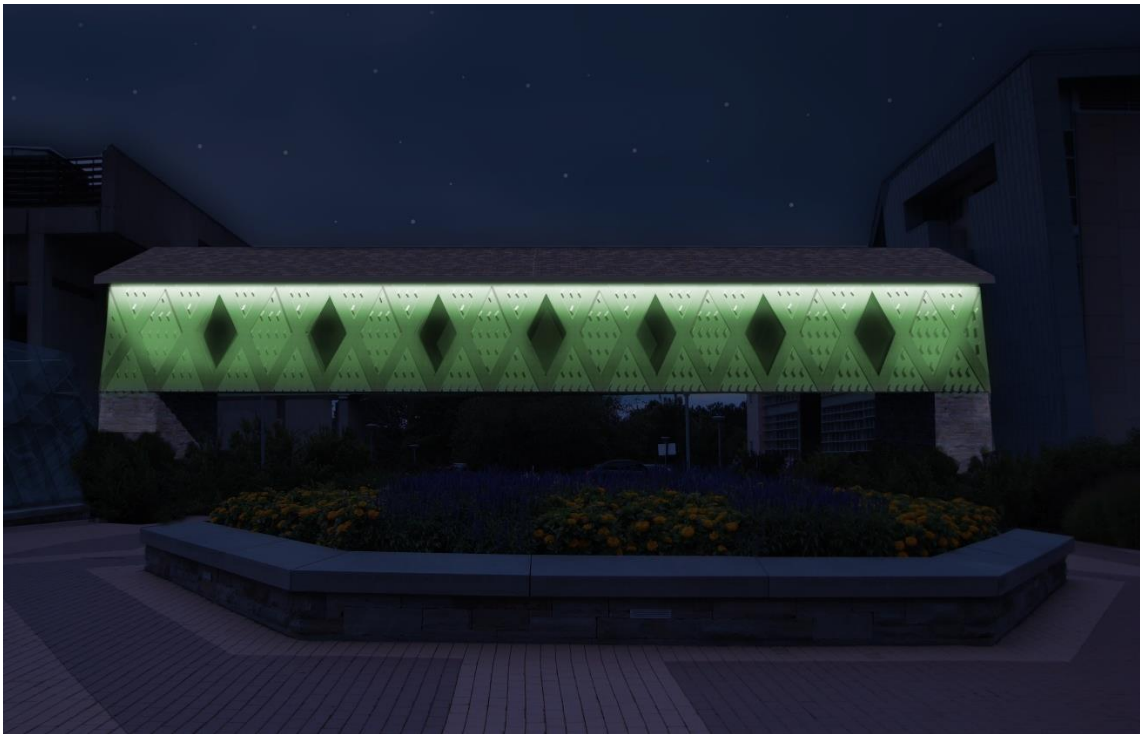


Figure 34: Exterior Lighting of the Bridge with Green LED Light

Figure 35: Exterior Lighting of the Bridge with Blue LED Light

Conclusion

This report provided an overview of both the analysis and the redesign of the Peggy Ryan Williams Center. The existing gravity and lateral system of the building were analyzed during the fall semester. Both of the systems were found to be adequate. A scenario was then created in which the schedule was no longer critical, thus allowing concrete to be explored as the building material. Through further investigation, it was determined to redesign the gravity system of the building to be a one way concrete slab system with pan joists, girder, and columns. The pan joists were designed using the CRSI Manual. After a joist system was selected, the same system depth (24-½") was used throughout the building. The beams and girders were then designed using spBeam. Finally, the loads were carried down through the building and the columns were designed using spColumn. Through the use of this system, the floor system depth was decreased by 6-⅝". This allowed for a larger floor to ceiling height, thus opening up the interior spaces. A few girders and columns were able to be removed from the existing framing of the building. Therefore, pending the owner's preference, a more open floor plan may be utilized.

The next section of the report focused on determining if the new gravity system would be adequate to act as both the building's gravity system and its lateral system. Seismic and wind loads were calculated per ASCE7-10, and RISA was used in order to determine the stiffness of the concrete frames. Once the forces were distributed accordingly, spColumn was used to design/analyze the columns. In the end, it was determined that the building's gravity system was adequate to act as the building's lateral system as well.

The pedestrian bridge was the focus of the remaining portion of the report. Two inspirational concepts were considered for the redesign, a reflection on the building's original name, "The Gateway Building," and a reflection of New York's historical covered bridges. It was decided to redesign the bridge using the covered bridge inspiration. Next, a Warren Truss was designed for the side truss of the redesigned bridge. The bridge was designed through the use of the historical Indexing Method and the Steel Manual. This structural redesign opened the door for both an architectural breadth of the bridge façade and a lighting breadth of the exterior of the bridge.

A detailed Autodesk Revit model served as the main feature of the architectural breadth. The Revit model shows the careful consideration that was taken into both the design and the materials chosen for the bridge façade. The bridge was then rendered to allow a nice comparison to be seen between the existing bridge and the redesigned bridge. While both designs allow the bridge to be cohesive with its surroundings, the redesign allows the pedestrian bridge to stand out more from its surroundings.
In order to complement the structural redesign of the bridge and the architectural breadth, an exterior lighting breadth was performed. The inspiration for the façade of the bridge and in turn the lighting of it was the Yotel Building in New York City. A luminaire from Philips Color Kinetics was selected in order to achieve a similar lighting affect. The report showcases various lighting effects which are possible with the high performance luminaire.

The structural depth of the bridge, architectural breadth, and lighting breadth all provided a great learning experience and a peak into the design process that goes into creating large scale bridges.

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