ASCE Charles Pankow Foundation Architectural Engineering Student Competition

Team Registration Number **05-2013**

Our team submitted designs in the following categories:

Building Integration Design
Structural Systems
Mechanical Systems
Lighting/Electrical Systems
Innovative Construction Management and Construction Methods

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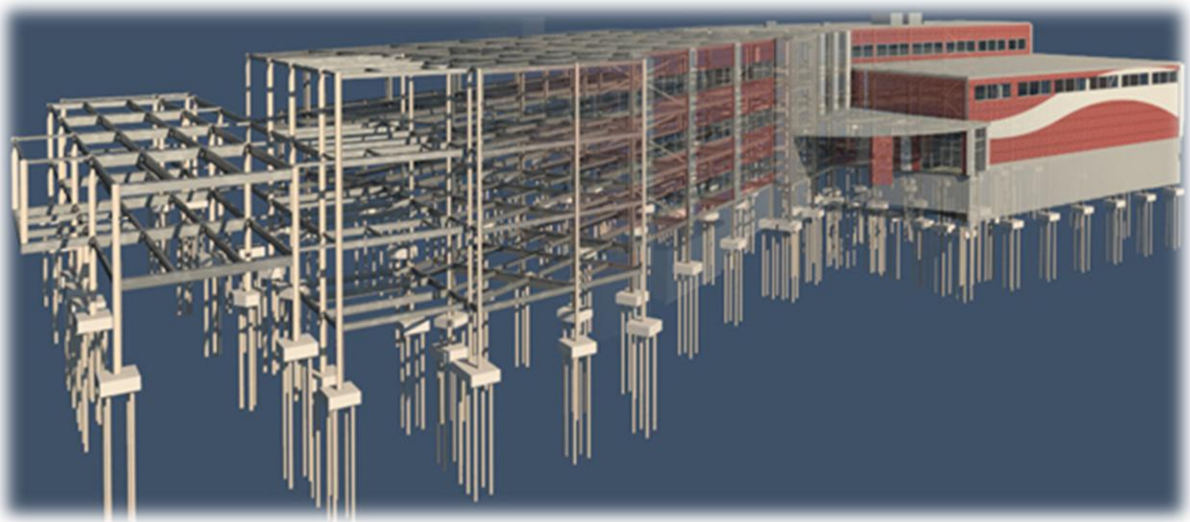
Project Goals

"Design and implement innovative strategies and structural systems that will build a better community and push project integration to new limits." With this discipline specific mission statement in mind, the structural systems, analysis procedures, and modeling of the High Performance Reading Elementary School were selected and executed based on their adherence to engineering standards and ability to utilize present and future advancements.



Rendering of Main Structural Systems

Community Based Criteria

According to the Reading School District 2011-2016 Strategic Plan [11], proficiency percentages on mathematics of the Pennsylvania System of School Assessment (PSSA) are 75% for the 3rd Grade and 60% for the 6th Grade. In the reading section, percentages are 55% for the 3rd Grade and 39% for the 6th Grade. On average, the PSSA scores drop as the grade level increases. In addition to Reading's low test scores, the city itself is ranked number five for highest crime rates by Congressional Quarterly Inc. [5] for cities with an 85,000 average population. To foster a better learning environment for the students and an inviting recreational space for the community, the structural systems were selected based on the following:

- Transparent and open spaces that induce productivity in learning and allows for a crime-free environment
- A structural frame that assists in defining educational spaces from community spaces
- A system that unites the architecture with the structural frame to create appealing and functional spaces

Design Based Criteria

To create a unified design process between the engineering disciplines and design a structure that employs safe and current advancements, the structural systems were also selected based on the following:

- Capability to integrate with other engineering disciplines
- A design process and modeling techniques that allow for Virtual and BIM based analyses

Codes & Standards

Local Codes

Prior to preliminary structural calculations, research was conducted to find which codes and guidelines the city of Reading, Pennsylvania followed. After investigating into Reading's official website [6] for local analysis procedures, a review of their Community Development Criteria page was performed. This review led to curiosities pertaining to one specific line item that was found. A roof design live load of 65 psf was called out under the Structural Design Criteria. After inquiries into this loading with Reading building officials, it was clarified that the 65 psf roof live load addressed and incorporated minimum snow loads. Reading lies within a Case Study (CS) region on the ground snow load charts of ASCE 7 [3] so the 35 psf ground load supplied by Reading officials was used in snow related analyses.

Structural Codes

During the same conversation with the building officials regarding the 65psf roof live load, it was found that Reading follows the IBC [9] Standard for structural design. In accordance with IBC [9], ASCE 7 [3] was used to find all building loads associated with our structure. For the design procedures of concrete structural elements, ACI 318 [1] and CRSI [7] were used. For the design of structural steel elements, the AISC Steel Manual [2] was used.

Snow Drift Code

Many spaces of the structure have heights that stop at different elevations of the building. To account for snow drift loads which have caused multiple roof failures in Pennsylvania, section 7.6 and 7.7 of ASCE 7 [3] was used for these spaces. With controlling leeward pressures, a drift surcharge of 70psf (see Structural Supporting Documentation page A-1 for complete calculation) was found for the gymnasium roof which spans 125' and is 14' below the abutting roof level.

IBC Fire Rating Standards

Fire rating requirements for structural members were found using chapters 5, 6, and 7 of IBC [9]. Equation 5-1 in section 506.1 was used to define our allowable area that can be used for construction classification. The frontage of our building in addition to our full sprinkler coverage allowed for an area and story increase that permitted the building to be classified as B type II Construction. Under these parameters, structural members are not required to use fire protective materials.

See the References section for all citations included in the Summary Narrative

Substructure

The probability of encountering sink holes due to unsuitable soils raised awareness as to which types of foundation systems should be selected. After research was conducted on the recommendations by the geotechnical report and close coordination with the Construction Management division of our team, it was decided that driven piles were the safest and most reliable decision due in majority to the vast amount of unsuitable soils on the site. This choice was also driven by schedule, cost, and the classification of the school as a High Performance building. Mitigating structural repairs in the future such as slab-on-grade cracking and wall separation helps to further define the criteria of a High Performance building.

Grade Beams

Grade beams tie into each pile cap to provide lateral bracing and are used as a part of the sub-grade foundation system. In accordance with ACI 318 [1], a typical grade beam supporting the prefabricated panels above was designed. Beams were designed with a depth of 22", a width of 16", and are reinforced with (5) #9's placed 3" from the bottom of the beam. The beam was increased to a depth of 24" to match the 24" depth of the pile caps to account for constructability on site. Checks for simplified deflections, flexural strength, shear reinforcing, steel strain, and axial support (see supporting documentation page A-2 for complete calculations) yielded a typical exterior grade beam as show in Supporting Document page S-201. To account for the possibilities of sinkhole formations at any location on the site, members were designed as continuous fixed-fixed beams with no support underneath their span. In addition, based on engineering judgment, beams were partially designed as beam-columns to handle 10% of the axial load that acted on adjacent columns.

To satisfy requirements for grade beam design, the following criteria were considered.

- Smallest cross-sectional dimension shall be greater or equal to the $\frac{\text{clear span between columns}}{20}$
- Closed ties shall be provided at spacing less than or equal to the $\frac{\text{smallest cross sectional dimension}}{2}$

Driven Piles and Caps

Structural columns are supported by pile caps ranging from three to six piles. Design recommendations and calculations (see supporting documentation page A-5 for complete calculations) were done in accordance to Chapter 13 of CRSI [7]. CRSI uses specific sections from ACI 318 [1] to check two way action punching shear and flexural reinforcement requirements. The typical exterior wall pile cap was sized for a 262 kip axial load which resulted in a cap depth of 24"and a cap length and width equal to 5'4". These caps are supported by (4) 8" diameter, 3,000 psi concrete piles each with a 0.25" steel encasing which consists of 60 ksf Hollow Structural Steel members. Four pile systems were chosen in order to account for the punching shear due to the column loads which a three pile system with the same depth and pile diameter cannot adequately resist. The decision to use a four pile system with a square pile cap also stemmed from coordination in respect to constructability with the Construction Management team. For interior column applications, pile caps supported by three piles with pre-determined dimensions given by CRSI [7] were implemented. The three 10" diameter pile system uses a pyramidal cap design with a length and width equal to 5'6" and 5'4". For cases where columns are in proximity to one another by about 3', a six pile system was used to support both columns. The design of these caps was simplified to follow the procedures specified by CRSI and this yielded a cap depth of 24" and a total length and width of 8'11" and 5'6".

To satisfy requirements for pile cap design, the following criteria were considered.

- Embed piles into caps by at least 6"
- Rebar clear cover to be at least 3"
- Center-to-center pile spacing minimum is 3' for piles up to 12" in diameter
- 1'6" is the minimum pile cap depth requirement

Superstructure

Steel Frame

As assumed by the geotechnical consulting firm, the superstructure is comprised of steel framing. Elementary schools can be supported by steel, concrete, wood, or the more traditional materials of masonry, but steel was selected for its ability to create open spaces that will assist in uniting the community. The selection of steel also conforms to the geotechnical recommendations for the bearing soil capacities as presented in the report.

The spaces of the school were designed for the indicated loads specified below in Tables 1 to 3.

Table 1: Minimum Required Live Loads (ASCE 7-05)

Space	Live Loads per ASCE 7-05 [psf]
Flat Roof	20
Green Roof	100
Classroom	40
Corridor on 1st Flr	100
Corridors above 1st Flr	80
Gymnasium	100
Stairs/Exits	100

Note: Live Loads subject to reduction except for Roof Live Load

Table 2: Calculated Dead Loads

Material	Dead Loads [psf]
Built-up Roof	20
Misc. (ducts, fixtures, etc.)	10
3 VLI Deck w/ 3.5" Concrete	63
3" Gypcrete [a]	30

[a] 3" Gypcrete only applies to classroom spaces for the radiant flooring

Table 3: Calculated Snow Load

Level	Snow Loads [psf]
Ground (Local Code)	35
Roof (east wing / west wing)	27 / 29.4

Note: Roof Snow Load found using ASCE 7-05 Eqn. 7-1

The main body of the framing consists of wide flange beams, girders, and columns. For the more elaborate spaces such as the special education room, Round HSS steel columns as pointed out in Figure 1 were used for their ability to have horizontal beam connections at any angle. These columns were seldom used in locations with abrupt changes in building orientation. For the roofs that differ in elevation and are adjacent to one another the supporting columns were designed with considerations to snow drift loads.

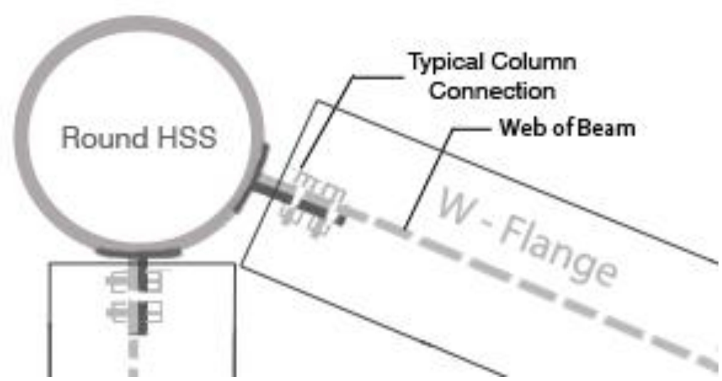


Figure 1: Connections for Round HSS Column

Building Separation

To account for movement from lateral loads and the change in building orientation between the west wing and the east wing of the school, two "separate buildings" were designed. Further explained in the section on Lateral Systems below, an expansion joint with a width of 1.5" creates the needed separation between the two buildings to account for calculated story drifts. The maximum roof drift in the E-W direction due to seismic loading for the west wing as calculated by our RAM model is 0.24 inches, and for the central wing is 0.27 inches. These drift values account for the C_d factor in ASCE 7 [3]. Exemplified by Figure 2, the separation also allows the structural decking to run in the X-direction in the west wing and run 30 degrees due North of the X-direction in the east wing of the building.

Creating two separate buildings reduces induced lateral forces in cases of accidental torsion as well. This layout makes for easier constructability of running the deck continuously over the beams with spacing controlled by allowable spans for construction without the need for shoring.

Furthermore, the west wing of the school will serve as the emergency hurricane shelter. By classifying an area as a “shelter” the seismic importance factor for the west wing changes from 1.25 for an elementary school to 1.5. This increases the seismic demands however the importance factor for wind loads remain unchanged at 1.15. Since the shelter in this case is essentially a separate building, the greater forces need only to be applied to the west wing which allows for the size of the lateral system to be reduced in the main building.

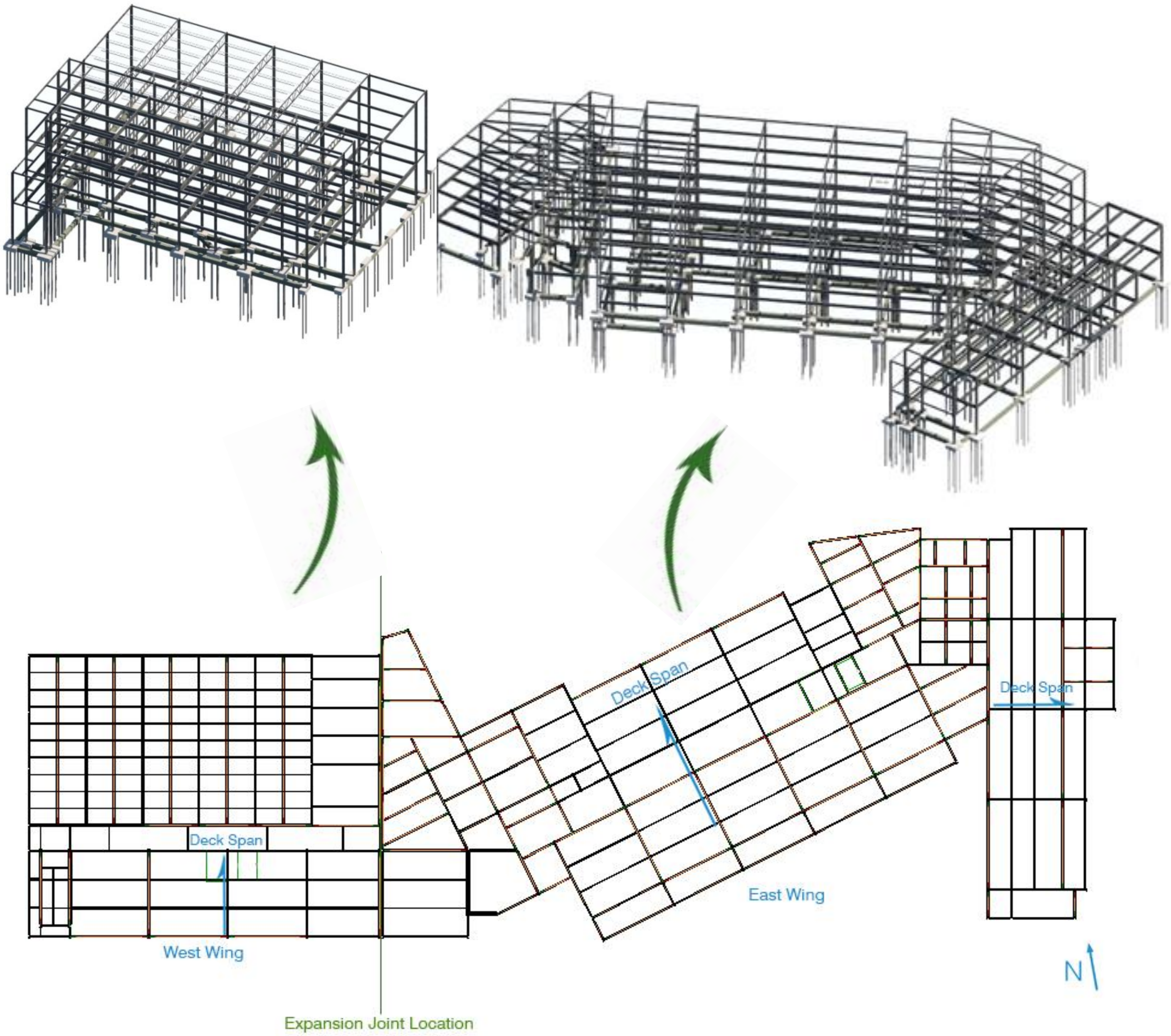


Figure 2: Structural Floor Plan and 3D Views

Lateral Analysis and Design

Main Wind-Force Resisting System

To do a suitable wind load analysis, simplifying assumptions were made in order to use the ASCE 7 [3] Chapter 6 Method 2 "Analytical Procedure". Changing the projected area of the building into several schemes to account for orthogonal wind loading was the first step. The scheme giving greater orthogonal projected areas was chosen as the controlling scenario (shown in Figure 3). The height of the elementary school falls below 60' which enables it to be classified as a low-rise building. In accordance with this, only figure 6-10 from ASCE 7 [3] Section 6.5.12 needed to be considered for wind loading cases. An important assumption for this process is the building enclosure classification. The building was assumed to be partially enclosed since there is no guarantee that user controlled spaces will always have the windows closed nor is there a guarantee the windows will not be breached in a strong storm.

Results from the analytical procedure yielded forces that were distributed to each floor level. Results that were found included windward, leeward, sidewall, and internal pressures. Forces from each were compiled together if they acted in either the north-south direction or east-west direction. The resulting base shear found from the sums (windward and leeward) for the gymnasium/shelter were 245 kips in the N-S direction and 199 kips in the E-W direction. For the main building the resulting forces were 476 kips in the N-S direction and 287 kips in the E-W direction. (See supporting documentation page A-7 for complete calculations and parameters.)

Figure 3: Assumed Rectangular Floor Plan for Wind Load Calculations

Seismic Force Resisting System

With the given Seismic Site Class C from the geotechnical report, chapters 11 and 12 of ASCE 7 [3] were used to find resulting story and base shear forces. Calculations done based on short and 1 second periods yielded an SDC (Seismic Design Category) equal to B. The 1 second period category was the controlling factor since the short period resulted in a SDC of A. The SDC was checked against USGS' U.S. Seismic Design Maps Web Application. USGS provides a web-based program that uses ASCE 7 [3] and user input to find the SDC Category. The SDC was found by accounting for and comparing USGS' output to hand based calculations. Results between the hand analyses and computer database values were found to differ in the short period. The design acceleration response for the short period (S_s) was specified as 26% in the USGS database. In the hand calculation procedure, a lower, but conservative S_s factor of 20% as compared to the actual 18% as given by Fig. 22-1 in ASCE 7 [3] was selected. Despite the differences between USGS output and the hand calculations, the same results were found. The USGS database as well as the traditional design method resulted in a worst case SDC equal to B and thus this value was used for further design of the lateral system.

To find values and factors for the equations associated with base shear, both structures were assumed to be Ordinarily Steel Concentrically Braced Frames. For the two main directions of analysis it was also assumed that both will have a maximum height of 42'. These factors were used to find a building period (Ta) equal to 0.33 seconds. The period gives reassurance to assumptions made considering the rule of thumb for a regularly shaped steel building which follows Period(T)=0.1N, where N is the number of stories. The period was later found to be equal to 0.44 seconds after computer analyses and this value is more realistic considering the shape and different heights of the frames. SDC B warrants that the analysis of a structure needs only to consider directions perpendicular to one another. The resulting base shear for both frames was applied to the N-S and E-W directions for design purposes and was found to be 153 kips in the gymnasium and 318 kips in the main structure (see Table 4). When broken down further the resulting story forces are:

Table 4: Seismic Variables

	West Wing	East Wing
Risk Category	IV	III
I_e	1.5	1.25
Site Class	C	C
R Factor[a]	3.25	3.25
SDC	B	B
Building Weight	2033 kip	5727 kip
Base Shear Coefficient, C_s	0.0738	0.0615
Base Shear	153 kip	318 kip

[a] Ordinary Steel Concentrically Braced Frames are used in both directions of analysis

<u>West Wing</u>
Roof = 49 kips
Story 3 = 84 kips
Story 2 = 20 kips
Base Shear = 153 kips

<u>East Wing</u>
Roof = 115 kips
Story 3 = 140 kips
Story 2 = 63 kips
Base Shear = 318 kips

(See supporting documentation page A-10 for complete calculations and parameters.)

Initial Seismic loads were established based on a one structure building and not two separated by an expansion joint as implemented in the actual design however the calculations were later revised to coincide with the building separation. Using RAM, the seismic forces found agree with the hand tabulated values where earthquake loads control in the E-W direction of the east wing and wind load controls in the E-W direction of the west wing and the N-S direction of both sections of the building.

Lateral Force Resisting System

To resist the controlling seismic and wind loads, X-braced bays between columns provide the needed stiffness for the structure. Trial and error layouts of the bracing in RAM based on engineering judgment and architectural constraints led to the use of four braced bays in the west wing and six braced bays in the east wing. The elements making up the braces are rectangular HSS members ranging from 4.5 x 4.5 x 3/8 to 6 x 6 x 5/8 as depicted in Figure 4.

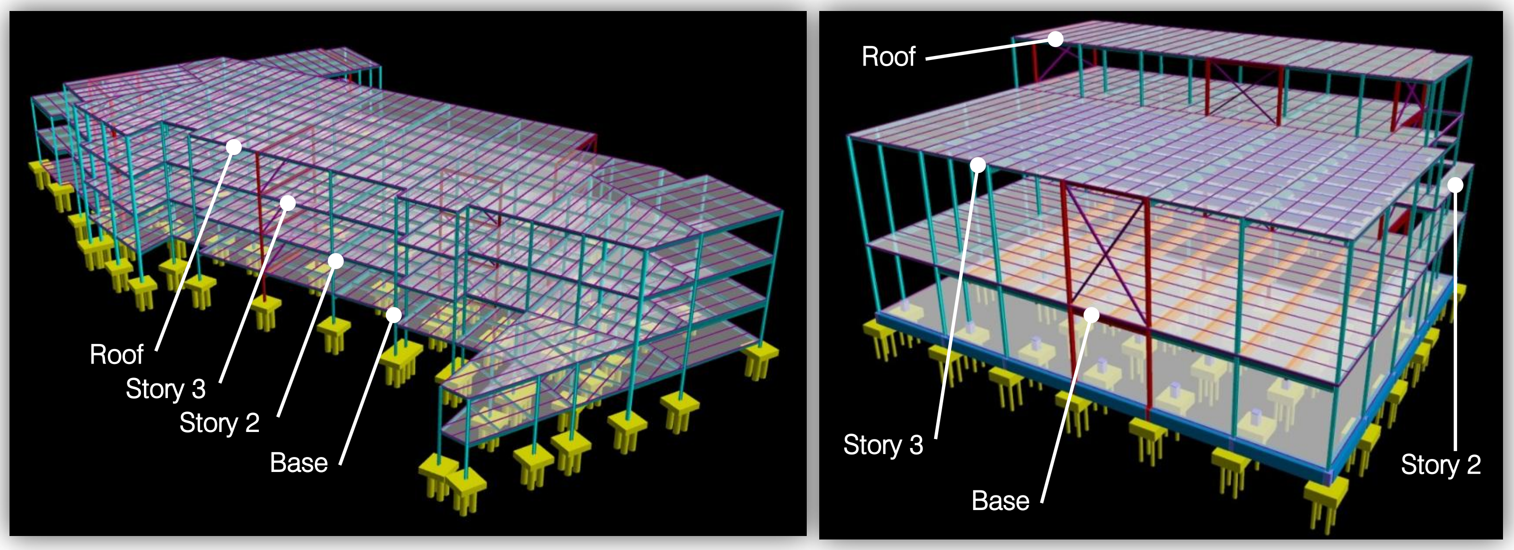


Figure 4: Lateral Bracing (in Red) for West and Main Structures

Applying cross-bracing allows the HSS members to support tensile and compression forces that are created as a response to seismic cyclic loading. The brace sizes were controlled by compression with a $KL/r < 200$. The length for this purpose was taken from column to column which is more conservative rather than column to midpoint. Diagonal bracing instead of moment frames was chosen because a moment frame solution would require heavier beam and column members and more expensive moment connections. The floor plan of the school also caters to placement of bracing without interrupting occupied bays. X-bracing was specifically selected over other types because of its efficiency to handle lateral loads.

Areas of Interest

Multi-Purpose Room and Natatorium

To fully integrate the community and educational spaces with one another without breaching the security between the two, it was decided to locate the Multi-Purpose Room above the Natatorium. To achieve the desired long spans, vibrational control between the spaces, and seamless coordination with the other systems, special consideration was given to the selection of the structural system that would support these spaces.

Long-Span Joist Girders

Trusses and girders are commonly used to support auditoriums, stadiums, convention centers, and any other long spanning structures. The use of joist girders and an exposed ceiling for the multi-purpose room as shown in Figure 5 is not only an economical approach; it is an educational approach that provides a clear view of the HVAC and Electrical systems. Exposing these systems introduces students to a ceiling assembly that may invoke early interest into engineering. The framing members supporting the built-up roof of this room are Vulcraft 48G10N10F specified joist girders. The selection of a 48" depth joist girder permits HVAC systems to run through the member and lighting fixtures to be attached to the bottom chords of the members.

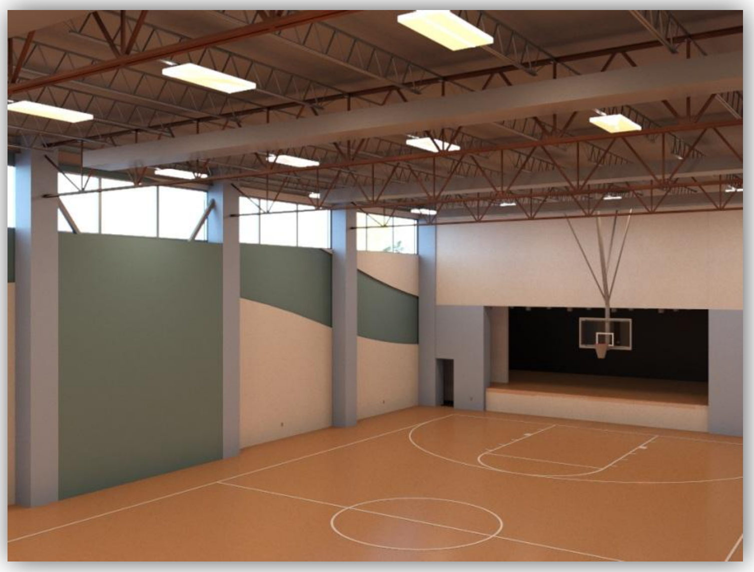


Figure 5: Multipurpose Room Showing Long-Span Joist Girders

Loads for the roof included a 20 psf roof live load, 32 psf built up roof, miscellaneous, and metal decking dead load, and a 27 psf roof snow load which was derived using the local 35 psf ground snow load. A snow drift surcharge was also taken into account since the roof elevation changes between the gym and classroom area (see calculations on page A-1). Supported by the girders is a 3NA22 roof deck which is a 3" deep metal deck with acoustical properties. This deck was specifically picked for its added sound control benefits. Perforations in the corrugated deck admit sound that will be absorbed by added acoustical insulation. This will control lunch and recreation time banter that would otherwise transmit to the adjacent hall. The 20' spaced girders are each braced by 16K2 bar joists that are spaced 6' apart.

To ensure requirements are met due to uplift since the multipurpose room will be used as a shelter as well, when being submitted for manufacture, an uplift pressure of 26 psf must be specified in accordance with sections 5.11 and 1004.9 of the Vulcraft Steel Joist and Steel Girders, Steel Joist Institute Catalogue [13]. This pressure was found by the wind calculations included on supporting documentation page A-7. Furthermore, the joist girders must be braced at the first bottom chord panel points.

Cellular Beams

Cellular or castellated beams have the capability to span 50 meters without intermittent supports, a featherweight make-up compared to wide flange beams of a similar strength to span ratio, reduced vibrational effects due to an increased depth, and lower fabrication costs compared to traditional joists. Cellular Beams do not only serve admirably in supporting the floor of the Multi-Purpose Room but also add an aesthetic appeal.

Cellular Beams are marked as an innovative architectural expression and are more of an unconventional approach for long spans. They are made by splitting one or two wide flange beams with a specially engineered cut pattern and then the two pieces are welded together to create a deeper yet lighter member as shown in Figure 6 below. The design for the span was checked three different ways because of the rarity in the use of these beams. Discrepancies that were found between computer analysis programs also warranted for three different checks. The beam was originally designed as a single member in RAM SBeam which determined an adequate size of L36 x 55/68. This nomenclature indicates the overall depth of the beam will be 36 inches while the two halves are made from a W24 x 55 for the top and a W24 x 68 for the bottom. Hand calculations were performed based on an AISC draft design guide [8] provided by a renowned structural engineer who specializes in steel design. The chosen size resulted inadequate by several k-ft which prompted additional calculations. After a period of trial and error, a composite member size of L36 x 55/76 was deemed suitable for the span (see supporting documentation page A-13). The selected member was additionally analyzed along with the west wing frame using RAM Structural System and was found to pass.

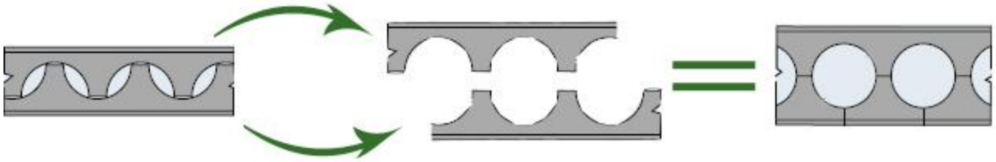


Figure 6: Process of Creating a Cellular Beam

To further help the vibrational control of the beams, a composite 3VLI deck with 2 studs per rib resulting in a total of 84 studs per beam, is connected to the top of the beams to add more resistance. The Cellular Beams will each have shear connections to supporting columns. The columns tie directly into supporting pile caps with piles driven to bedrock. Similar to the joist girders specified in the Multi-Purpose Room, this will transfer loads and vibrations from the recreational space above directly into the foundations below the finished basement floor. A finished rendering of the Pool area can be seen below in Figure 7.

Figure 7: Integrated Design of Cellular Beams

The placement of the Natatorium below the Multi-Purpose Room allowed for the integration that was desired by our design based goals. It solves the problem of adding the optional pool area to an already restricted site while still keeping the needed security separations in mind. Depth restrictions of this layout were a big concern that the design team faced. The Cellular Beams have 24” diameter spaces within them which allow for duct runs, sprinkler piping, and other vital systems to have adequate penetration space while providing an acceptable height clearance as shown in Figure 7. In addition to the supplied allowable space for other systems, the beams will not delay construction because of their pre-manufacturing process off site. As a final consideration for the beams in an indoor pool environment, humidity retardant epoxy paint is applied to the steel to combat the average humidity levels of 60% as well as corrosion due to the effects of chlorine.

Building Envelope

Prefabricated Panel System

To improve the construction time of the façade as well as provide a continuous enclosure to the building, a prefabricated panel system is used as shown in Figure 8. The panels come in customizable sizes as specified and designed by the engineering team. For the location of this elementary school, the panels must be able to resist leeward and windward loads of 21 psf and 19 psf respectively. However, in this situation, reinforcement is controlled by temperature and shrinkage. By placing 6x6 w4.5/4.5 welded wire mesh at the center of the two concrete sections that make up the enclosure, these requirements are satisfied (see supporting documentation page A-17). Since the panels in this particular situation will not be load bearing but instead are carried by the structural framing of our building, reinforcement for bending in the vertical direction was designed solely for the dead weight of the panels during construction. After calculations it was found that one #3 reinforcing bar with 2” of clear cover, continuous for the length of the panels will resist the resulting bending moments.

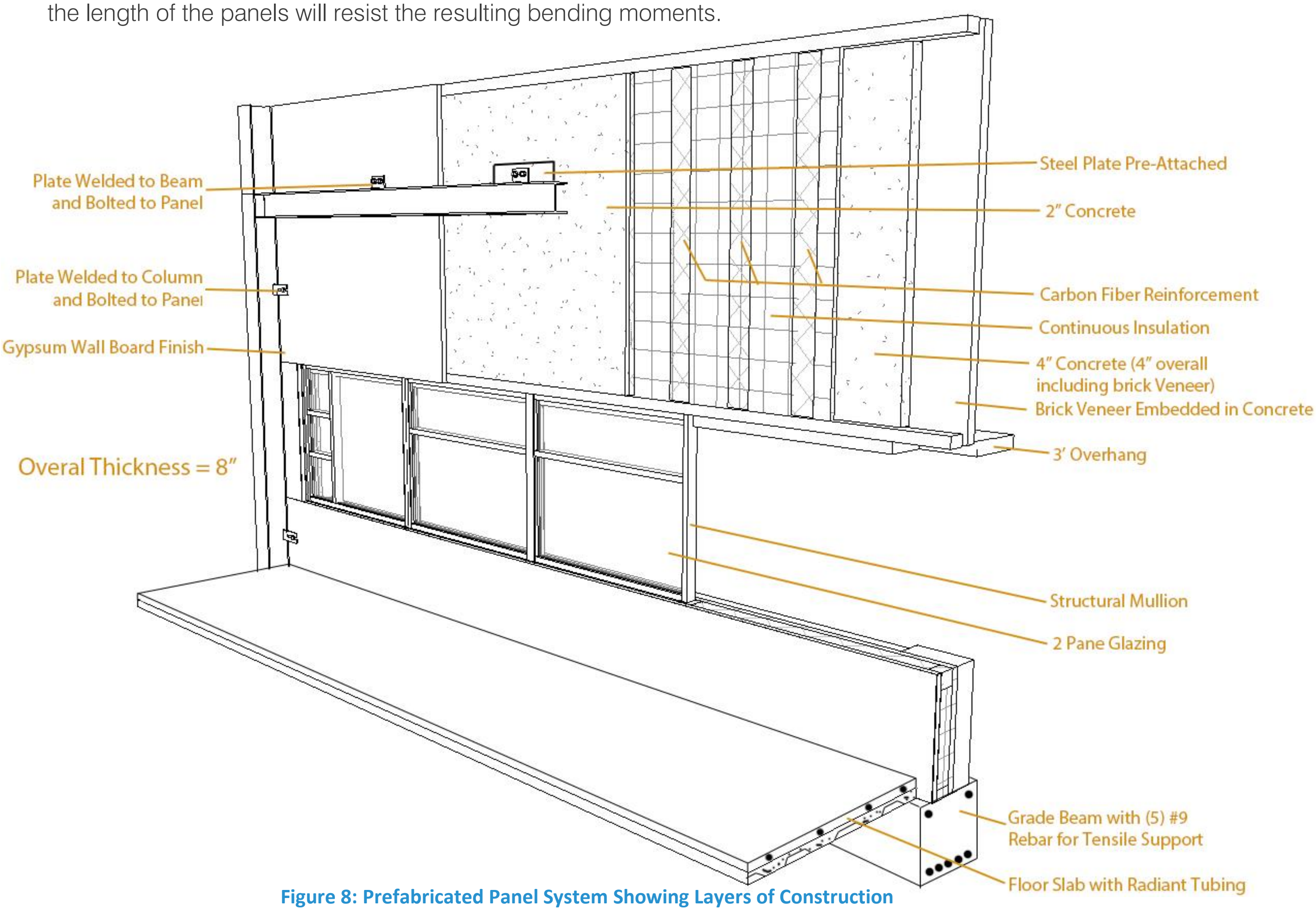


Figure 8: Prefabricated Panel System Showing Layers of Construction

These reinforcing bars will be used on all 4 edges of the panel to be certain the panel will contain enough strength regardless of the orientation it may mistakenly be lifted. It must be noted, the panels are to be shipped and erected in the vertical position. The carbon fiber truss system ties the two concrete sections together and causes the panel to act as one continuous section. This allows the entire 8" moment arm to be utilized during windward and leeward pressures. By using carbon fiber rather than steel reinforcing, the thermal bridge between the concrete is eliminated.

To ensure proper connection to the supporting beams and columns, angles using bolted connections were designed to come prefabricated with the panels. Typical panel heights include three, six, and nine feet with five feet of glazing that breaks up certain panels. To accommodate bending moments and connections between the nonstructural components, load resisting mullions are evenly spaced and welded between the panels that are broken up by the exterior glazing. Base plates for the mullions also come pre-installed to the top and bottom of the necessary panels. The glazing is then installed between the mullions at the construction site. This entire panel assembly when installed creates a vertically continuous façade that can resist lateral loads and will not have to be supported with added braces.

Virtual Modeling and Analysis

BIM

The industry of architectural engineering is steadily moving towards the use of technology for aid in design and construction. BIM, which is the epitome of this, is defined as the process involving the generation and management of digital representations of physical and functional characteristics of a facility. The transition from the older means of design to the paperless world requires the utmost amount of care and attention to detail.

Revit 2013 was the primary medium used to construct the structural model for visual and integration purposes. To get the full use out of Revit, measures were taken in classifying structural members into worksets and correctly laying them out in respect to one another. Worksets is a feature of Revit that allows materials, components, and assemblies to be organized into specific categories which can then be exported into programs such as Revit Navisworks for 4D scheduling. By itemizing every structural member into various worksets or "groups", it makes the scheduling for construction purposes easier. It also allows quick access to specific members; for instance a call-out on all steel angles used in the model can be generated.

Cross-Platform Modeling

Laying out the structural system correctly in Revit makes cross-program analyses more useful and accurate. The less the model has to be rearranged, the more accurate it is in determining clashes with other systems and limiting errors in connection distances during construction in the field. Two separate programs were used to either model the structure and/or analyze it. Revit 2013 [12] (Figure 9) was used to model the two separate structures that make up the single building while Bentley RAM Structural Systems [10] (Figure 10) was used for analysis purposes.

Figure 9: Revit 2013 Model of Multipurpose Room and Nearby Classrooms

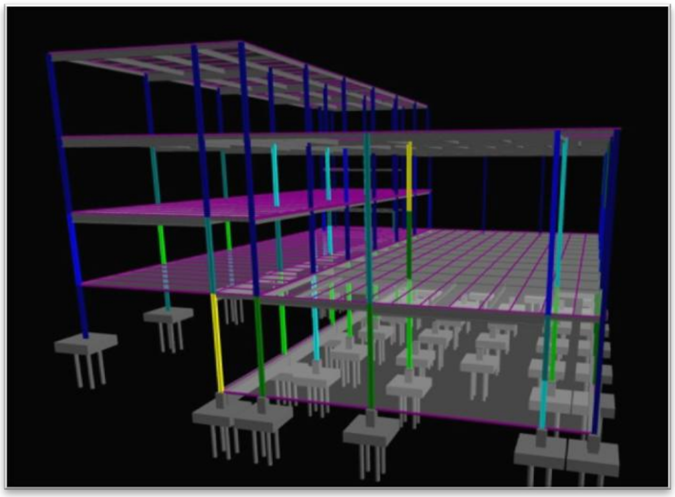


Figure 10: RAM Model of Multipurpose Room and Nearby Classrooms

The import and export features between the programs are not seamless. Issues between floor levels and types of members are examples of items that can become "lost in translation". One particular issue in the exporting step dealt with the change of our modeled joist girders in Revit becoming W Flanges in RAM. This problem called for extra attention in the design of the joist girders because of the improper treatment by computer software. Despite these issues, information given about the W Flanges that were sized instead of the joist girders helped in comparisons with the amount of loads and bending moment that each can handle. The use of these types of features such as member sizing in RAM can and has saved time for the design of the structure. Furthermore, because it was decided to analyze the frame as two separate buildings, importing the designs from RAM to Revit became a challenging task. Updating the members in Revit manually rather than using the add-in to import from RAM to Revit provided the reassurance necessary to ensure accuracy and completeness.

Structural Analysis

The two separate structures that frame the west wing and the main building were exported from Revit to RAM for computer analysis. Loads were created per ASCE 7 [3] and per Table 1 in the "Superstructure" section. RAM takes advantage of the codes by embedding criteria for limitations on deflection, earthquake loading, and wind loading. After applying the necessary loads and creating story diaphragms, the program was ran to analyze and update member sizes including all gravity and lateral members.

RAM considerations and assumptions

- Equivalent Lateral Force Method used for seismic loading
- All diaphragms assumed to be rigid [excluding the roof since the built up roof is a semi rigid diaphragm]
- Live Load Deflections limited to Length/360
- Deflections due to total Superimposed Load limited to Length/240
- Steel designed per AISC 360 [2] LRFD
- Concrete designed per ACI 318 [1]
- Concrete members only analyzed to obtain reaction forces to use for hand design and calculations

In early design iterations, Ram Structural Systems was used in our project to analyze steel members only since the program is not as user friendly when designing concrete elements. For concrete members, the sizes must be specified before performing an analysis which causes a need to perform hand calculations prior or run several iterations until the specified strength is reached whereas steel members "retain" the analysis data and can be individually selected and changed to perform as desired. Due to this the reactions from the designed steel elements were acquired through the program and then used to perform the appropriate hand calculations. Once the final sizes of the concrete members were determined through the hand calculations, a second analysis was performed in Ram Structural Systems to ensure requirements were met.

Conclusion

Buildings must be designed with the occupants in mind. The listed goals defined in the “Community Based Criteria” and “Design Based Criteria” sections ultimately cater to the kids, educational faculty, and the community. Through the design process, this main focus shaped the final decisions while the question of “Will this facility benefit the community?” refined it. This question considers all processes of phasing from schematic to construction and also helped the design team to relate any decisions back to the goals.

Though there were many considerations for the structural system with benefits and weaknesses to each choice made, a steel framing system was chosen to increase the adaptability and overall cohesiveness of the school as we wanted it to be an area to bring the community together. A concrete structure with shear walls tends to make it difficult to move or combine rooms in the future and limits the visibility throughout since “punching” openings would call for special procedures and detailing. Steel excelled in meeting these requirements. Our building was separated into two structures to accommodate for the abrupt building shape change and use of our west wing as a natural disaster relief shelter. This allowed the main structure to be designed for a lower Importance Factor (I_e) of 1.25 for seismic while the west wing needed to be designed for a seismic Importance Factor of 1.5. The Importance Factor for snow load also decreased to 1.1 for the east wing while the west wing required an Importance Factor of 1.2. The joint also indicates the boundary between the community area and the educational facilities.

It was learned through the process of working closely with all of the other disciplines that there are some areas which warrant much more cross disciplinary collaboration than others. Two of the main items included the building enclosure and pool area which affect almost every aspect of the design industry including mechanical, lighting and construction (see Integration report pages 8 and 13 respectively). This necessitated close coordination to be sure to handle lighting requirements, heat loss values, cost and schedule, visual aesthetics and ultimately how the prefabricated panels were going to be attached to the framing while resisting gravity and lateral loads. Other major areas of concern tended to generate in areas where there were factors from existing conditions or site restrictions which limited the ability for normal design methods. This urged our team to think in a creative manner. This was particularly relevant in the pool area where depth of excavation was limited as well as clear height (finished floor to lowest obstruction) was restricted due to the nature of the pool facility.

The choice of using Cellular Beams to support the Multipurpose Room floor, which is above the Natatorium, stemmed from the restrictions associated with ducts, sprinkler piping, plumbing and lighting. The use of this specific type of beam allows these systems to be incorporated into the same vertical dimension as the steel structure. Further investigation revealed this to also be a more economical solution due to their relatively light weight, customizable properties, increased vibration controls, and ability to span long distances. Cellular Beams help to add an interesting aesthetic to the pool area, however having them exposed in the humid chlorine prone environment justified the need for a moisture protective coating.

Overall our design intent to have flexibility in the use of the building as well as meet pre-established project criteria to bring together and help rebuild the Reading area has been met by inviting residents to a space of entertainment, safety, and education. The final product is not only the result of designing a school; it is the result of the innovation and integration that was required to build a better community.

“To design and implement innovative strategies and structural systems that will build a better community and push project integration to new limits.”

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Referenced Equations

Building Area Modifications (IBC 2009 Eqn. 5-1)

$A_a = \{A_t + [A_t * I_f] + [A_t * I_s]\}$

A_a	=Allowable building area per story,	A_t	=Tabular building area per story
I_f	=Area increase factor due to frontage,	I_S	=Area increase factor due to sprinkler protection

Flat Roof Snow Loads (ASCE 7-05 Eqn. 7-1)

$p_f = 0.7 * C_e * C_t * I * p_g$

C_e	= Exposure factor from table 7-2	I	= Importance factor from table 7-4
I_f	= Thermal factor from table 7-3,	p_g	= Ground snow load

Snow Loading and Snow Drift

Gym Roof snow Drift

Ground snow Load 35 psf (Reading codes)

Exposure factor C_e Category B = 1.0 (partially exposed per table 7-2)

Thermal factor C_t 1.0 (Heated/insulated structure)

Importance factor I – Elem School – 1.1 (per table 7-4)
Shelter – 1.2

Flat Roof snow load

$= .7 C_e C_t I p_g$

$= .7(1)(1)(1.1)(35) = 27$ psf (for east wing)

$= .7(1)(1)(1.2)(35)$
$= \underline{29.4}$ psf (for west wing)

Drift on lower Roof

h_d = Larger of windward or Leeward

Leeward $= .43 (L_{u_1})^{1/3} (p_g + 10)^{1/4} - 1.5$

$= .43(40)^{1/3}(35+10)^{1/4} - 1.5$

$= 2.31'$

L_{u_1} = length of upper roof in direction of wind

Windward $= .75\, h_d$

$= .75[.43(L_{u_2})^{1/3}(p_g+10)^{1/4} - 1.5]$

$= .75[.43(60)^{1/3}(45)^{1/4} - 1.5]$

$= 2.14'$

L_{u_2} = length of lower roof in direction of wind

$h_d = 2.31'$ controls

Drift width (w) $= 4\, h_d$ ($h_d < h_c$) fig 7-8 ✓

$= 4(2.31)$
$= 9.24'$

Snow density $\gamma = .13 p_g + 14$

$= .13(35) + 14 = 18.55$ pcf

Max drift surcharge (p_d) $= h_d \gamma$
$= 42.85$ psf

height of flat roof snow
$= 29.4$ psf $/ 18.55 = 1.585$ ft

max drift load $= 29.4 + 42.85 = 72.25$ psf

Grade Beam Design

Grade Beam Design | ACI 318-08 | Page 1 of 3

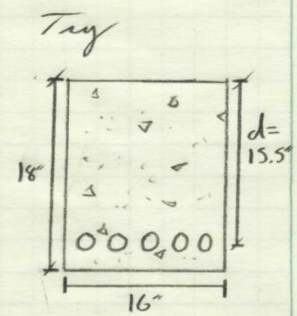
Notes [ACI 21.12.3]

- Smallest cross-sectional dimension shall be ≥ clear spacing between columns ÷ by 20.
- Grade beams can be seperate from slab-on-grade
- Closed ties shall be provided at spacing ≤ ½ smallest cross sectional dimention

Typical Exterior Grade Beam

Length = 26′7″ Width = $\frac{26'(12)+7''}{20} = 16''$

Dead Loads



Self Weight: 150 pcf (1.5′)(1.33′) = 0.30 Klf

Pre-Fab Panel: 150 pcf (0.5′)(12′) + 8 psf (12′) = 3.5 Klf

DL Total = 3.8 Klf

Live Loads

Classroom: 40 psf (13′) = 0.5 Klf

LL Total = 0.5 Klf

$\omega_u = 1.2(3.8) + 1.6(0.5) = 5.3$ Klf

Simplified Defection Check

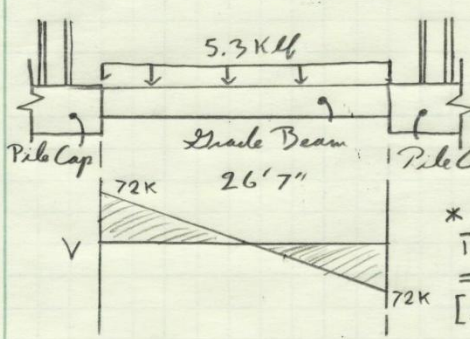
[ACI Table 10-1]

Beams, both ends cont.

$\frac{l}{21}$ = min $h = \frac{26.58' \times 12}{21}$

= 15.2″ ✓

GOOD IF SINK HOLE FORMS BENEATH



$* \frac{53}{104} \omega_u l_n = 72$K [AISC]

M_{mid} [ACI 8.3] $= \frac{\omega_u l_n^2}{14} = \frac{5.3(26.58')^2}{14} = 267$ K-ft

- End support used to account for all exterior grade beams

Grade Beam Design ACI 318-08 Page 2 of 3

Flexural Strength

$\varepsilon_s \geq \varepsilon_y$ assumed

Try (5) #8

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{0.79(5)\,60{,}000}{0.85(3{,}000)\,16''} = 5.8''$$

$$M_n = A_s f_y \left(d - \frac{a}{2}\right) = (0.79)\,60{,}000\left(15.5'' - \frac{5.8''}{2}\right) = 2986\text{ K-in or } 248\text{ K-ft} < 267\text{ K-ft}$$

NO GOOD

Try (5) #9

$$a = \frac{1(5)\,60{,}000}{0.85(3{,}000)(16'')} = 7.35''$$

$$M_n = 1(5)\,60{,}000\left(15.5' - \frac{7.35''}{2}\right) = 3548\text{ K-in or } 296\text{ K-ft} > 267\text{ K-ft}$$

Use (5) #9

ε (Strain Check)

$\beta_1 = 0.85 \quad f'_c \leq 4{,}000$ psi

$$c = \frac{a}{\beta_1} = \frac{7.35''}{0.85} = 8.65'' \qquad \varepsilon_s = \varepsilon_t = \frac{\varepsilon_u}{c}(d-c) = \frac{0.003}{8.65''}(15.5'' - 8.65'')$$

$$= 0.0024 \ll \varepsilon_t = 0.005$$

(Tensile)

NO GOOD

* Increase $h = 24''$

$d = 21.5''$

$$\text{New } M_n = 1(5)\,60{,}000\left(21.5'' - \frac{7.35''}{2}\right) = 5348\text{ K or } 446\text{ K-ft}$$

$$\text{New } \varepsilon_s = \varepsilon_t = \frac{0.003}{8.65}(21.5'' - 8.65'') = 0.0045 < \varepsilon_t \quad \text{OK}$$

$$\phi\,[\text{ACI Fig. 9.3.2}] = 0.65 + 0.25\,\frac{\varepsilon_t - \varepsilon_y}{0.005 - \varepsilon_y}$$

$$= 0.65 + 0.25\,\frac{0.0045 - 0.00207}{0.005 - 0.00207} = 0.85 = \phi$$

$$\phi M_n = 0.85(446) = 379\text{ K-ft} > 267\text{ K-ft}$$

GOOD

Shear Strength

$$V_s = \frac{V_u}{\phi} - V_c = \frac{72 - 21.5\left(\frac{72}{13.29' \times 12}\right)}{0.75} - 2(1)\sqrt{3{,}000}\,(16'')(21.5'')$$

$$= 45.35\text{K} \leq 8\sqrt{f'_c}\,b_w d = 151\text{K} \quad \text{OK}$$

Check $V_s = 45.35 \leq 4\sqrt{f'_c}\,b_w d = 4\sqrt{3{,}000}\,(16'')(21.5'') = 75.4\text{K}$ OK

Grade Beam Design ACI 318-08 Page 3 of 3

Shear Reinforcement Spacing

$S_{max} = \frac{d}{2} = 10.75''$, Use 10"

* Grade Beam closed ties $\leq \frac{1}{2}$ 16" = 8"

$Av_{min} = 50 b_w s / f_{yt} = 50 (16'')(8'') / 60{,}000 = 0.11\ in^2$

Use (2) #3 stirrup

$S = A_v f_{yt} d / V_s = 0.22(60)(21.5)/45.35 = 6.26''$ Use (2) #3 @ 6" @ 2" from support

ϕV_n for $s = 8'' \Rightarrow 0.75\left(37.7K + \frac{0.22(60)21.5}{8''}\right)$

$= 55K$

Use (2) #3 @ 8" @ 35" from support and stop @ 120" from support for $0.5\ \phi V_c = 17K$

Lateral Support

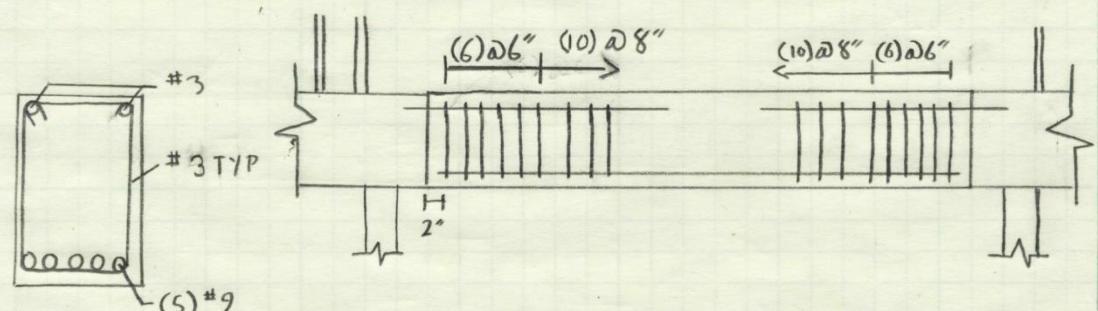
*Design Beam for 0.1Pu from adjacent column

$P_u = 262K$

$0.1 P_u = 26K$ $V_c = 2\left(1 + \frac{0.1P_u}{2{,}000 A_g}\right)\lambda\sqrt{f'c}\, b_w d$ $\lambda = 1$ for Beams (Compression)

$= 2\left(1 + \frac{26K}{2{,}000(24 \times 16)}\right)(1)\sqrt{3{,}000}\,(16'')(21.5'')$

$= 37.6$ ✓ GOOD FOR COMP



*Follows Table A.7 [ACI] w/o #4 Ties Typ

Pile and Pile Cap Design

Pile Cap Design | CRSI 2008 | Page 1 of 2

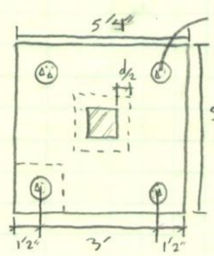
Notes
- Embed piles into caps by 6"
- Rebar clear cover is 3" minimum
- Pile spacing minimum is 3'
- Grade 60 steel used for rebar
- Area of reinforcement based on $d = D - d_c$, D = total depth

① *Typical Exterior Pile Cap Design*

$d_c = 10''$ for steel, $8''$ for conc

$P_u = 262K/4 = 65.5K = V_u$ (Ignore Pile Cap Self Weight)

Try



$D = h = 24''$
$d = 24'' - 8'' = 16''$

*1'6" is minimum cap depth for rebar developement length (l_d) per Design of Reinforced Concrete by M.L. Gambhir

Two Way Action

[Around Column]: $b_o = 4(12'' + d) = 4(12'' + 16'') = 112''$ Use $f'_c = 3{,}000$ psi

$\phi V_c = 4\sqrt{f'_c}\, b_o d$ [Controls]

$= 4\sqrt{3{,}000}\,(112)(16)/1{,}000 = 392K > P_u = 262K$ and $V_u = 65.5K$ O.K.

[Around Piles]: $b_o = 2(8'' + 16'') = 48''$

$\phi V_c = 4\sqrt{3{,}000}\,(48)(16)/1{,}000 = 168K > V_u = 65.5K$ O.K.

Flexure [ACI 10.5.4 + 7.12.2.1]

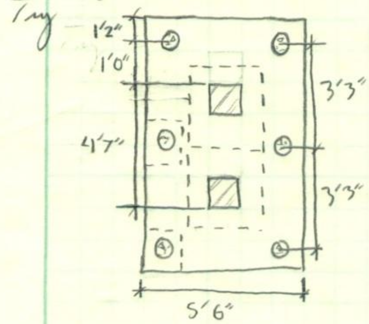
$M_u = 2(65.5K)(1.5' - 0.5') = 131$ ft-K

Try $\rho = 0.18\% \Rightarrow A_{s\,min} = 0.0018bh$
$= 0.0018(12'')(24'') = 0.5184$ in²/ft $\boxed{\text{Use } 9\#5 @ 6\tfrac{1}{3}'' \text{ each way}}$

② *Two Column Pile Cap Design*

Equivalent $P_u = 322K + 71K/4 = 98K = V_u$

Try



$D = h = 24''$
$d = 16''$

Two Way Action

$b_o = 4(16'' + 16'') = 128''$ [Around Column]
$\phi V_c = 448K > P_u = 393K$ O.K.

$b_o = 3(8'' + 16'') = 72''$ [Around Pile]
$\phi V_c = 252K > V_u = 98K$ O.K.

Flexure

$M_u = 2(98K)(1.625' - 0.67') = 187$ ft-K

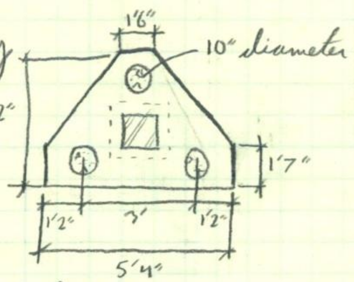
$A_{s\,min} = 0.5184$ in²/ft $\boxed{\text{Use } 15\#5 @ 6\tfrac{3}{4}'' \text{ Long};\ 9\#5 @ 6\tfrac{1}{3}'' \text{ Short}}$

Pile and Cap Design   CRSI 2008 & ACI 318   Page 2 of 2

③ Typical Interior Pile Cap Design

$P_u = 262K/3 = 87.3K = V_u$

Try



$D = h = 24"$
$d = 16"$

Two Way Action

[Around Column]: Same as Exterior Pile Caps, but thicker so... O.K.

[Around Pile]: $b_o = 2(10"+16) = 52"$

$\phi V_c = \frac{168K}{48}(52) = 182K > V_u$ O.K.

Flexure

Use recommended rebar size from CRSI, 4#6 or Equivalent $\boxed{6\#5 \text{ 3-Ways}}$

④ Pile Design

8" Pile w/ Steel Encasing

$P_u = 65.5K$

Bearing Capacity of Concrete: $\phi(0.85 f'c A_1)$

$= 0.9(0.85)(3)(\pi)(4)^2$

$= 115.4K > P_{u_{8"}} = 65.5K$

$> P_{u_{10"}} = 87.3K$

O.K.

* 8" and 10" Concrete Pile can handle P_u, so use 1/4" thick steel encasing as recommended by Geotech * minimum

Wind Loading

Wind Load Analysis ASCE 7-05 pg 1 of 3

MWFRS Analytical procedure (assuming simplification on pg 2)

Reference in ASCE 7-05

Basic wind speed (v) : 90 mph (Fig 6-1)
Kd : Wind directionality factor : 0.85 (table 6-4)
Category ~~III~~ IV → Elementary school + Shelter (table 1-1)
Importance factor : I = 1.15 (for both) (table 6-1)
Exposure Category B → Urban Area (6.5.6.2)
Topigraphic factor : $K_{ZT} = 1$

Gust factor :

Natural frequency (na) $\frac{75}{h} = \frac{75}{42} = 1.786$ (e6-17)
* Lower bound

$\frac{100}{h} = \frac{100}{42} = 2.38$ (e6-18)
* upper bound

Since natural Frequency is well above 1, It is considered Rigid per § 6.2 therefore gust factor may be taken as 0.85 per § 6.5.8.1

Building Enclosure Classification : Partially enclosed
* can not be sure windows will all be shut or not breach

Internal Pressure Coefficient : $GC_{pi} = \pm .55$ (Figure 6-5)

External pressure Coefficient : (Figure 6-6)

Windward wall $C_p = 0.8$
Side wall $C_p = -0.7$
Leeward wall (N-S wind) $C_p = -0.5$
Leeward wall (E-W wind) $C_p = -0.3$

L/B (N-S wind) $= \frac{146}{375} = .39$

L/B (E-W wind $= 2.57$

GC_{PF} Low Rise (Fig 6-10)
Roof < 0-5
Surface

1	2	3	4	5	6	1E	2E	3E	4E
.40	-.69	-.37	-.29	-.45	-.45	.61	-1.07	-.53	-.43

* Reference Fig 6-10 for surface locations

Velocity Pressures

K_h : Case 1 per notes on table 6-3
* Interpolate between 40' + 50' for 42'

$\frac{.81 - .76}{50-40}(42-40) + .76 = .77$

$q_h = q_z = .00256\, K_z K_{zT} K_d V^2 I$ § 6.5.7.2
$= .00256(.77)(1)(.85)(90^2)(1.15)$
$= 15.607$ psf

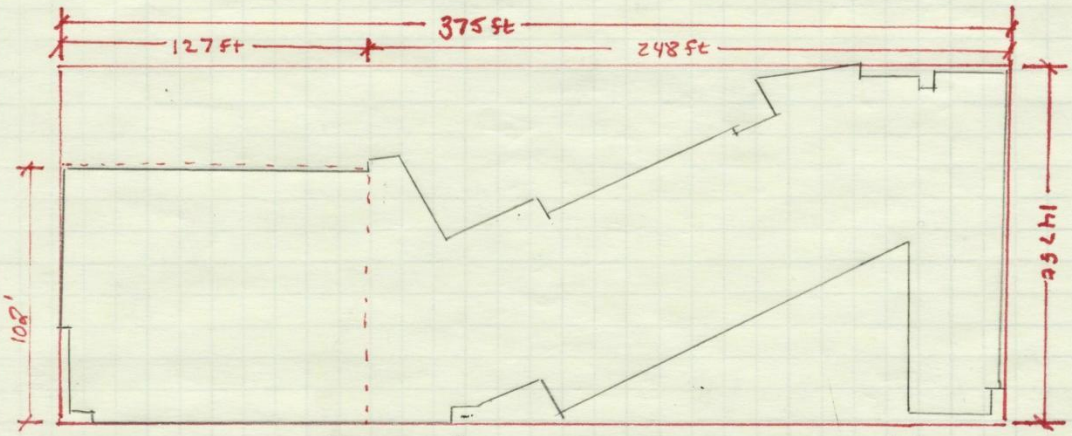
Vind Load Analysis ASCE 7-05 pg 2 of 3

Reference

Design Wind load Low Rise §6.5.12.2.2

$p = q_h [GC_{pf} - GC_{pi}]$

Surface	PSF	Surface	PSF
1	6.24 ± 8.58	6	-7.02 ± 8.58
2	-10.77 ± 8.58	1E	9.52 ± 8.58
3	-5.77 ± 8.85	2E	-16.70 ± 8.58
4	-4.53 ± 8.58	3E	-8.27 ± 8.58
5	-7.02 ± 8.58	4E	-6.71 ± 8.58



ASSUMED PROJECTED BUILDING SURFACES FOR SIMPLIFIED CALCS

a = 0.1 Least horizontal Dim or .4 h (smallest of)

Gym Main

10.2 or 16.8 14.7 or 16.8

Check: > 4% LHD ✓

> 3' ✓

* Force Calculations on attached sheet

For comparison to Seismic, wind must be multiplied by 1.6 factor

N-S MWFRS: 446.75(1.6) = $\boxed{714.8 \text{ Kip}}$

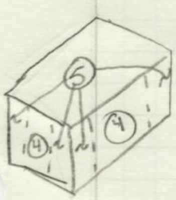
E-W MWFRS: 179.23(1.6) = $\boxed{286.8 \text{ Kip}}$

Components + Cladding $h < 60'$ §6.5.12.4.1

$p = q_h [(GC_p) - (GC_{pi})]$

$q_h = 15.607 \text{ psf}$ (pg 1) + § 6.5.6.3

$GC_{pi} = \pm .55$ (fig 6-5)



Walls

Effective wind area = span(span/3) = $28\left(\frac{28}{3}\right) = 261 \text{ ft}^2$

GC_p may be reduced by 10% for Roof slope < 10° (6-11A note 5)

for ④ GC_p = +.75, -8.5 for ⑤ GC_p = +.75 -.9

Corner zone 5

$p = 15.607[.75(.9)+.55] = 19.12 \text{ psf}$

$15.607[-.9(.9)-.55] = -21.23 \text{ psf}$

Zone 4

$p = 15.607[.75(.9)+.55] = 19.12 \text{ psf}$

$= 15.607[-.85(.9)-.55] = -20.52 \text{ psf}$

N-S Direction MWFRS Gym

GC_{pi} 0.55 h= 42
q_z = 15.607 a= 10.2

	GC_{pf}	Area (ft^2)	=(q_z)x(GC_{pf})x(Area) (kips)		=(q_z)x(GC_{pi})x(Area) (kips)	Wall facing	Worst Case (kips)
1	0.40	4477.00	27.95	+/-	38.43	South	66.38
2	-0.69	1.00	-10.77	+/-	8.58	(psf for uplift)	-19.35
3	-0.37	1.00	-5.77	+/-	8.58	(psf for uplift)	-14.36
4	-0.29	4477.00	-20.26	+/-	38.43	North	-58.69
5	-0.45	4284.00	-30.09	+/-	36.77	West	-66.86
6	-0.45	4284.00	-30.09	+/-	36.77	East	-66.86
1e	0.61	856.00	8.15	+/-	7.35	South	15.50
2e	-1.07	1.00	-16.70	+/-	8.58	(psf for uplift)	-25.28
3e	-0.53	1.00	-8.27	+/-	8.58	(psf for uplift)	-16.86
4e	-0.43	856.00	-5.74	+/-	7.35	North	-13.09

	N-S Force	E-W Force
	153.66	0.00
with 1.6 wind factor =	245.86	

E-W Direction MWFRS Gym

	GC_{pf}	Area (ft^2)	=(q_z)x(GC_{pf})x(Area) (kips)		=(q_z)x(GC_{pi})x(Area) (kips)	Wall facing	Worst Case (kips)
1	0.40	3427.00	21.39	+/-	29.42	West	50.81
2	-0.69	1.00	-10.77		8.58	(psf for uplift)	-19.35
3	-0.37	1.00	-5.77		8.58	(psf for uplift)	-14.36
4	-0.29	3427.00	-15.51	+/-	29.42	East	-44.93
5	-0.45	5334.00	-37.46	+/-	45.79	South	-83.25
6	-0.45	5334.00	-37.46	+/-	45.79	North	-83.25
1e	0.61	856.00	8.15	+/-	7.35	West	15.50
2e	-1.07	1.00	-16.70		8.58	(psf for uplift)	-25.28
3e	-0.53	1.00	-8.27		8.58	(psf for uplift)	-16.86
4e	-0.43	856.00	-5.74	+/-	7.35	East	-13.09

	E-W Force	N-S Force
	124.33	0.00
with 1.6 wind factor =	198.92	

N-S Direction MWFRS Main Building

GC_{pi} 0.55 h= 42
q_z = 15.607 a= 14.7

	GC_{pf}	Area (ft^2)	=(q_z)x(GC_{pf})x(Area) (kips)		=(q_z)x(GC_{pi})x(Area) (kips)	Wall facing	Worst Case (kips)
1	0.40	9181.00	57.32	+/-	78.81	South	136.12
2	-0.69	1.00	-10.77	+/-	8.58	(psf for uplift)	-19.35
3	-0.37	1.00	-5.77	+/-	8.58	(psf for uplift)	-14.36
4	-0.29	9181.00	-41.55	+/-	78.81	North	-120.36
5	-0.45	6174.00	-43.36	+/-	53.00	West	-96.36
6	-0.45	6174.00	-43.36	+/-	53.00	East	-96.36
1e	0.61	1234.80	11.76	+/-	10.60	South	22.35
2e	-1.07	1.00	-16.70	+/-	8.58	(psf for uplift)	-25.28
3e	-0.53	1.00	-8.27	+/-	8.58	(psf for uplift)	-16.86
4e	-0.43	1234.80	-8.29	+/-	10.60	North	-18.89

	N-S Force	E-W Force
	297.73	0.00
with 1.6 wind factor =	476.36	

E-W Direction MWFRS Main Building

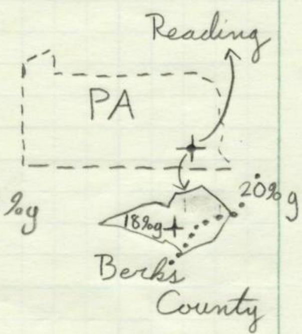
	GC_{pf}	Area (ft^2)	=(q_z)x(GC_{pf})x(Area) (kips)		=(q_z)x(GC_{pi})x(Area) (kips)	Wall facing	Worst Case (kips)
1	0.40	4939.20	30.83	+/-	42.40	West	73.23
2	-0.69	1.00	-10.77		8.58	(psf for uplift)	-19.35
3	-0.37	1.00	-5.77		8.58	(psf for uplift)	-14.36
4	-0.29	4939.20	-22.35	+/-	42.40	East	-64.75
5	-0.45	10416.00	-73.15	+/-	89.41	South	-162.56
6	-0.45	10416.00	-73.15	+/-	89.41	North	-162.56
1e	0.61	1234.80	11.76	+/-	10.60	West	22.35
2e	-1.07	1.00	-16.70		8.58	(psf for uplift)	-25.28
3e	-0.53	1.00	-8.27		8.58	(psf for uplift)	-16.86
4e	-0.43	1234.80	-8.29	+/-	10.60	East	-18.89

	E-W Force	N-S Force
	179.23	0.00
with 1.6 wind factor =	286.76	

Seismic Loading

Seismic Loading | ASCE 7-05 | Page 1 of 3

SDC (Seismic Design Category)



Risk Category III [Table 1-1]

I_e = 1.25
1.5, for Hurricane [Table 1.5-1]

S_S = 20%g (Reading lies outside of the 20%g line in the 18% region, but use 20% to be conservative)

S_1 = 6%g [Fig. 22-1, 22-2]

Check 11.4.1: $S_S > 0.15$ and $S_1 > 0.04$, so not permitted to use SDC=A

Seismic Site Class = C [Pg. 13 of Geotech Report]

F_a = 1.2 [Table 11.4-1], Short Period

F_v = 1.7 [Table 11.4-2], 1 sec. Period

$S_{MS} = F_a S_S = 1.2(0.2) = 0.24$ $\quad$ $S_{M1} = F_v S_1 = 1.7(0.06) = 0.102$

$S_{DS} = \frac{2}{3} S_{MS} = 0.16$ $\quad$ $S_{D1} = \frac{2}{3} S_{M1} = 0.068$

Based on S_{DS}, SDC=A [Table 11.6-1]

Based on S_{D1}, SDC=B [Table 11.6-2]

Use SDC=B

C_s

Table 12.2-1: Steel Ordinarily Braced
$R=3.25$, Ω_0 = 2, C_d = 3.25, Height Limit = NL

* Equivalent Lateral Force Permitted per Table 12.6-1 *

$T_a = C_T h_n^x = 0.02(42)^{0.75} = 0.33$ sec

C_T = 0.02 [Table 12.8-2]

x = 0.75 [Table 12.8-2]

h_n = 42' (Max height of building)

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)} = \frac{0.16}{\left(\frac{3.25}{1.25}\right)} = 0.0615$$

$$\text{or } = \frac{0.16}{\left(\frac{3.25}{1.5}\right)} = 0.0738$$ (Gymnasium Hurricane Shelter)

Seismic Loading | ASCE 7-05 | Page 2 of 3

Check T vs. T_L : $T_L = 6$ secs [Fig. 22-12]

$C_s \le \frac{S_{D1}}{T\left(\frac{R}{I_e}\right)}$ for $T_a \le T_L \Rightarrow \frac{0.068}{0.33\left(\frac{3.25}{1.25}\right)} = 0.079 > 0.0615$

> 0.0738

$C_s \ge 0.01 \Rightarrow 0.0615 > 0.01$

$0.0738 > 0.01$ O.K. O.K.

Total Weight

Snow Load (Flat Roof): $p_f = 0.7 C_e C_t I_s p_g$ [Eqn. 7-1]

$p_f = 0.7(1.0)(1.0)(1.1)(35\text{ psf})$ $= 27$ psf

$C_e = 1.0$ [Exp B, Partial Exposure]

$C_t = 1.0$ [All Structures]

$I_s = 1.1$ [Table 1-1]

$p_g = CS =$ Local Code $= 35$ psf

*$p_f < 30$ psf, Do not use 20% snow load in W = Dead Load + 25% Storage Live Load + 20% Snow Load [12.7.2] *

West Wing (Gymnasium & Shelter)

Roof

Concrete DL = 56.25 psf (12,400 ft² − 7,440 ft²) = 281K

Deck DL = 2 psf (4,960) = 10K

Misc DL = 8 psf (4,960) = 40K

Wall DL = 45 psf (7')(328') = 103K

Total DL Roof = W_R = 434K

3rd Floor

Concrete DL = 56.25 psf (12,400) = 698 K

Deck DL = 25K

Misc DL = 99K

Wall DL = 45 psf (14') (448') = 282K

Total DL 3rd = W_3 = 1,104K

2nd Floor

Concrete DL = 56.25 (4,960) = 280K

Deck DL = 10K

Misc DL = 40K

Wall DL = 45 psf (14') (328') = 207K

Total DL 2nd = W_2 = 537K

* 1st Floor lies on grade so do not count W_1 *

Seismic Loading | ASCE 7-05 | Page 3 of 3

Main Building

Roof

Concrete DL = 56.25 psf (15,015 ft²) = 845 K

Deck DL = 30 K

Misc DL = 120 K

Wall DL = 45 psf (7')(546') = 172 K

Total DL Roof = W_R = 1,167 K

3rd Floor

Concrete DL = 56.25 psf (36,400 ft² - 12,400 ft²) = 1,350 K

Deck DL = 48 K

Misc DL = 19 K

Wall DL = 45 psf (14') (42' + 38' + 136' + 100' + 142' + 42' + 120' + 138') = 478 K

East Green Roof = 45 psf (5,040 ft²) = 227 K

Total DL 3rd = W_3 = 2,122 K

2nd Floor

Concrete DL = 56.25 psf (24,000 ft²) = 1,350 K

Deck DL = 48 K

Misc DL = 19 K

Wall DL = 478 K

Total DL 2nd = W_2 = 1,895 K

C_{vi}

West Wing

$\Sigma w_i h_i^K = 434(42') + 1{,}104(28') + 537(14') = 56{,}658$

*K=1 for $T_a \leq 0.5$ secs

$C_{v2} = \frac{537(14')^1}{56{,}658} = 0.13$

$C_{v3} = \frac{1{,}104(28')^1}{56{,}658} = 0.55$

$C_{vR} = 0.32$ $\Sigma = 1.0$ ✓

Base Shear: $V = C_s W$ [Eqn 12.8-1]

$= 0.0738(2{,}075K)$

$= \boxed{153K}$

Main Bldg

$\Sigma w_i h_i^K = 1{,}167(42') + 2{,}122(28') + 1{,}895(14') = 134{,}960$

$*C_{vi} = \frac{w_x h_x^K}{\Sigma w_i h_i^K}$

$C_{v2} = \frac{1{,}895(14')^1}{134{,}960} = 0.20$

$C_{v3} = \frac{2{,}122(28)}{134{,}960} = 0.44$

$C_{vR} = 0.36$ $\Sigma = 1.0$ ✓

Base Shear: $V = 0.0615(5{,}184)$

$= \boxed{318K}$

$\boxed{F_2 = 20K}$ $\boxed{F_3 = 84K}$ $\boxed{F_R = 49K}$ $\boxed{F_2 = 64K}$ $\boxed{F_3 = 140K}$ $\boxed{F_R = 115K}$

Story Forces

Cellular Beam Design

Cellular Beam Calculations pg 1 of 4

LL = 100 psf ASCE 7-05 (Non Reducible)
DL: Deck span = 10' o.c → Try 3.5" conc on 3 VLI
Gauge 20 max span = 11'-9" ✓ (3 span condition) Vulcraft catalog
Superimposed Live load max = 158 psf
Misc DL = 10psf (MEP Allowance)
100 + 10 = 110psf < 158 psf ✓

Use 3VLI20 with 3.5" topping = 63psf

$w_u = [(63+10)(1.2) + (100)(1.6)](10) = 2.48$ klf

Cellular Beam data
Span = 60' spacing = 10'-0"
Trial Beam W24x58 (top) W24x78 (Bot) ⟹ LB 36 x 55/78
Loading as Noted above
Deflection Limit: Live = L/360 Total = L/240

Assume fully braced by deck above
Steel: F_y = 50 ksi studs = Ø3/4", h = 4.5" F_u = 65 ksi
Conc: f'c = 4 Ksi NC t_c = 3.5" → 6.5 total t
* Beam checked for const strength by Ram SBEAM → Passes

Beam Properties

Top		Bottom	
d = 23.6"	I_x = 1350 in⁴	d = 23.9	I_x = 2100
t_w = .395"	S_x = 114	t_w = .440	S_x = 176
b_f = 7.01	Z_x = 134	b_f = 8.99	Z_x = 200
t_f = .505	A = 16.2	t_f = .68	A = 22.4

Resultant shape LB 36 x 55/78
D_o = 24" $e = S - D_o = 7.25"$
S = 31.25" $Loss = D_o/2 - [(D_o/2)^2 - ((S-D_o)/2)^2]^{.5} = .561"$
$d_{t\,top\,Net} = (d_{top} - (D_o/2 + loss))/2 = 5.52"$
$d_{t\,bot\,Net} = (d_{bot} - (D_o/2 + Loss))/2 = 5.67"$

$d_g = d_{t\,top\,net} + d_{t\,bot\,net} + D_o = 35.19"$
$y = ((.5D_o)^2 - (.225D_o)^2)^{.5} = 10.72"$
$d_{t\,top\,crit} = D_o/2 - y + d_{t\,top\,net} = 6.80"$
$d_{t\,bot\,crit} = D_o/2 - y + d_{t\,bot\,net} = 6.95"$

Check Limits of applicability
Design procedures for web post buckling only apply if $1.00 < S/D_o < 1.5$
and $1.25 < d_g/D_o < 1.75$.
$S/D_o = 1.302$ ✓
$d_g/D_o = 1.46$ ✓

Cellular beam calculations   pg 2 of 4

SECTION properties of top+ bottom Tees + Beam

Top tee @ ctr of opening

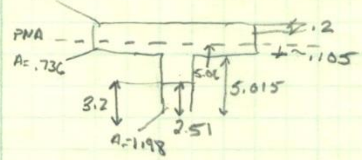
	Area	d (in)	I_x (in^4)	d - ENA = dena	Adena^2
Flange	3.54	detopnet − tf/2 5.27	.075	.99	3.47
Stem	1.98	$\frac{d_e\ topnet - t_f}{2}$ = 2.51	4.15	−1.77	6.20
	5.52		4.225		9.67

$ENA = \frac{\Sigma Ad}{\Sigma A}$
$= 4.28''$

$A_{tee} = 5.52$
$y = 4.28''$
$I_x = 13.895$
$I_y = \frac{(7.01)^3(.505)}{12} + \frac{(5.52-.505)(.393)^3}{12}$
$= 14.522$

$S_{x\,top} = I_x/(d_{topnet} - ena) = 11.21$
$S_{x\,bott} = I_x/ena = 3.25$
$r_y = \sqrt{I_y/A} = 1.62$
$r_x = \sqrt{I_x/A} = 1.59$

PNA : A ABOVE = A Below ; $5.52/2 = 2.76 < A_f$ ∴ PNA In flange



$2.76/b_f = 2.76/7.01 = .394$
$P_{NA} = d_{top} - .394 = 5.12$

$Z_x = (2.76)(.2) + (2.76)(5.0675 - 3.2) = 5.71$

Bottom Tee @ center

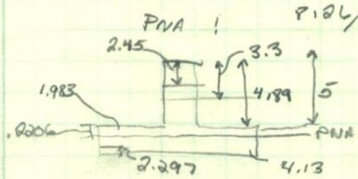
	AREA	d in	Ix	d-ENA	Adena^2
FLAnge	6.11	5.33	.235	.72	3.17
Web	2.15	2.50	4.29	−2.06	9.12

$ENA = \frac{\Sigma Ad}{\Sigma A}$
$= 4.51$

$A = 8.26$
$I_x = 16.815$
$I_y = 41.21$
$S_{x\,Top} = 15.8$
$y = 4.51$

$S_{x\,bot} = 3.73$
$r_y = 2.23$
$r_x = 1.43$
* equations same form as above

PNA ! $8.26/2 = 4.13 < A_f$ so in flange



$Z_x = (4.13)(.2297) + 4.13(5 - 3.3)$
$= 7.97$

Section Properties @ Critical Sections

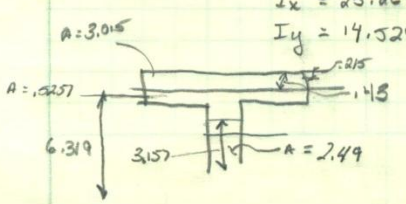
-Top

	A	d	Ix	dena	Adena^2
Flange	3.54	de topcrit − tf/2 6.548	.075	1.408	7.018
Web	2.49	(de topcrit − tf)/2 3.14	8.211	2	9.96

ENA = 5.14

$A_{tee} = 6.03$
$y = 5.14$
$I_x = 25.264$
$I_y = 14.529$

$S_{x\,top} = 15.22$
$S_{x\,bot} = 4.92$
$r_y = 1.55$
$r_x = 2.04$



$Z_x = (.215)(3.015) + (3.015)(6.319 - 3.157)$
$= 9.53$

Cellular beam Calculation.

Bott Tee @ Crit

	Area	d	I_x	dena	$Adena^2$
Flange	6.11	6.51	.235	1.13	7.8
Web	3.614	3.09	8.61	2.29	15.8

ENA = 5.38

$A = 9.124$ $S_x\ top = 26.34$
$I_x = 38.72$ $S_x\ bot = 7.19$
$I_y = 41.22$ $r_y = 2.12$
$y = 5.38$ $r_x = 2.06$

Net section

$A_{net} = 5.52 + 8.26 = 13.78$

$\bar{y}_{bs} = \frac{(5.52)(35.19) - (5.52 - 4.28) + 8.26(5.57 - 4.51)}{13.78}$

$= 17.07$

$y_{ts} = 35.19 - 17.07 = 18.12$

$d_{eff} = 35.19 - ((5.52 - 4.28) + (5.57 - 4.51)) = 32.89$

$I_{x_{net}} = 13.895 + 5.52(35.09 - 17.07 - 1.24)^2 + 16.815 + 8.26(17.07 - 1.06)^2$

$= 3702\ in^4$

Critical section

$A_{crit} = 6.03 + 9.124 = 15.154$

$\bar{y}_{bs} = \frac{6.03(35.19 - (6.8 - 5.14)) + 9.124(6.85 - 5.38)}{15.154}$

$= 14.19$

$y_{ts} = 35.19 - 14.19 = 21$

$d_{eff} = 35.19 - ((6.8 - 5.13) + (6.85 - 5.38)) = 32.22$

$I_{xnet} = 25.264 + 6.03(35.09 - 17.07 - 1.66)^2 + 38.72 + 9.124(17.07 - 1.43)^2$

$= 3909$

Composite section @ crit section

$n = E_s/E_c = 29e^6 / 57000\sqrt{f'c} = 8.044$

$b_{eff} = \min\left[\frac{span}{4}, spacing\right] = 180''$ or $\boxed{120''}$

$A_{conc} = b_{eff}(t_c) = 120(3.5'') = 420\ in^2$

$A_{tr} = \frac{A_{conc}}{n} = 52.21$

$K_c = 52.21 / (52.21 + 15.154) = 0.775$

$e_c = h_r + t_c/2 = 3 + 3.5/2 = 4.75$

Cellular Beam Calculation

Assume NA in Conc

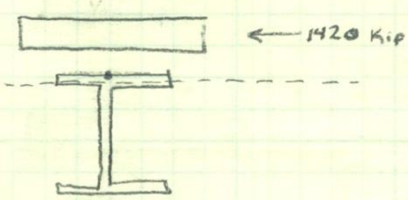
$$y_{cc} = \frac{A_{crit}\, x_{tc}}{A_{ctr}}\left(\sqrt{1 + \frac{2A_{ctr}}{A_{net}(t_c)}\left(y_{c\,steel} + \frac{t_c}{2}\right)} - 1\right)$$

$= 6.89 \quad \therefore$ NA in Steel

(deck + topping = 6.5")

$$V_c = .85 f'_c\, b_{eff}\, t$$
$$= .85(4)(120)(3.5)$$
$$= 1428\ K$$

$$x = 6.89 - 6.5 = .39''$$



$$M_n = .85 f'_c\, b_{eff}\, t_c\left(\frac{d_{eff}}{2} + \frac{t_c}{2} + 3\right) + 2F_y\, b_f\, x\left(\frac{d}{2} - \frac{x}{2}\right)$$

$$M_n = 1428\left(\frac{32.22}{2} + \frac{3.5}{2} + 3\right) + 2(50)(7.01)(.39)\left(\frac{32.22}{2} - \frac{.39}{2}\right)$$

$$= 2844.9$$

$$\phi M_n = .9 M_n$$
$$= 2560\ ft\text{-}K$$

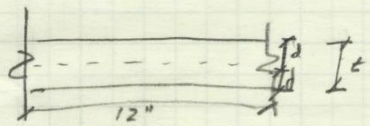
$$\frac{wL^2}{8} = \frac{2.48(60^2)}{8} = 1116\ K\text{-}ft < 2560\ K\text{-}ft \quad \underline{OK.}$$

* Design itterations omitted to achive submission length requirements

** Beam only checked for flexural strength to acheive submission length requirements. First itteration failed in vierendeel Bending. This beam passes vierendeel bending in both Ram Structure and Ram SBEAM while first itteration failed in Ram Structural.

Precast Panel Reinforcement Design

precast panel Design 1 of 2



length 28'
width 9'

Wind pressures
windward 19.12 psf
leeward 21.23 psf

minimum Reinforcement for shrinkage + temperature =

.0018 bwd per § 10.5.4 ACI 318-08

for 4" panel $.0018(4)(12) = .0864$

use 6x6 W4.5x4.5 = .09"

wind moment

$w = 40.35$ plf

$\frac{wl^2}{8}$ assume 1 way top to bottom span

$\frac{40.35(9)^2}{8} = 408 \text{ ft-lbs} \left(\frac{12\text{ in}}{1\text{ ft}}\right)\left(\frac{1\text{ Kip}}{1000\text{ lbs}}\right) = 4.9 \text{ K-in}$

$a = \frac{A_s f_y}{.85 f'_c b}$

$a = \frac{A_s(60)}{.85(5)(12)}$

$a = 1.176 A_s$

$4.9 \text{ k-in} \leq \phi M_n = \phi A_s f_y \left(d - \frac{a}{2}\right)$

$= .9(A_s)(60)\left(2 \text{ in} - \frac{1.176 A_s \text{ in}}{2}\right)$

$4.9 \text{ K-in} = 54 A_s(2 - .588 A_s)$

$4.9 \text{ K-in} = 108 A_s - 31.75 A_s^2$

$A_s \geq .046 \text{ in}^2$ 6x6 W4.5/4.5 = .09 in^2 good

$a = 1.176(.09) = 0.106$

$c = \frac{0.106}{.85} = .125$ $\varepsilon_s = \frac{.003}{.125}(2 - .125) = .045 > .005$

deflection due to wind

$$\Delta = \frac{5wL^4}{384EI} < L/360$$

$E = 57000\sqrt{f'c}$

$= 57000\sqrt{5000}$

$= 4030508$

$I = \frac{1}{12}bh^3$

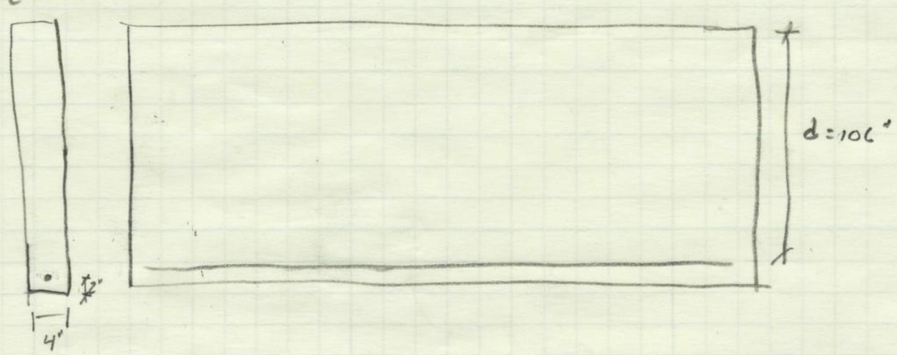
$= \frac{12}{12}(4)^3 = 64$

$$= \frac{5(40.35)(9^4)(144)}{384(4030508)(64)}$$

$= .0019'' < \frac{9(12)}{360} = .3$ ✓

+ Reinforcing for self weight during construction

Self weight/ft $\frac{4''}{12}(9')(150\text{pcf}) = 450\text{plf}$



$$M_u = \frac{450(28)^2}{8} = 44.1 \text{ K-ft}$$

$a = \frac{A_s(60)}{.85(5)(4)}$ $a = 3.529A_s$

$44.1 = 54A_s\left(10.6 - \frac{3.53A_s}{2}\right)$

$95.3A_s^2 - 572.4A_s + 44.1 = 0$

$A_s = .1007$ use (1) #3 $A_s = .11$

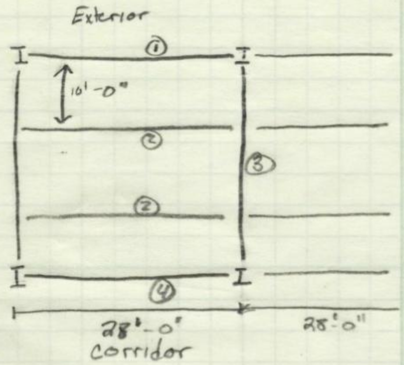
$a = 3.529A_s = .3882''$

$c = \frac{.3882}{.85} = .4567$ $\varepsilon_s = \frac{.003}{c}(d-c) = .69 > .005$ ✓

Member Spot Checks

Spot Checks   Pg 1 of 2

Typical classRoom



DL 108 psf (includes topping for radiant heat + 3.5" slab)
LL = 100 psf

Factored psf = $1.6(100) + 1.2(55) = 289$ psf

① $289\ \text{lb/ft}^2 \left(\frac{10'}{2}\right) = 1448$ plf $= w_u$

Max $M = \frac{wL^2}{8} = \frac{1448(28)^2}{8} = 141904$ ft-lb

$= 142$ ft-k

use 14 x 26 $M_n = 151$ ft-k ✓ Good

② $289(10') = 2890$ plf $= w_u$

Mom $= \frac{2890(28)^2}{8} = 283220$ ft-lb

$= 284$ k-ft

use W18 x 40 = 294 kft ✓ good

③ 289 psf$(28) = 8092$ plf

$\frac{6328(30)^2}{8} = 910350$ ft-lb

$= 910$ k-ft

use 27x84 = 915 ft-k ✓ good

④ $289(11) = 3179$ plf

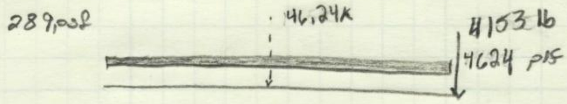
$\frac{3179(28)^2}{8} = 311542$ ft-lb

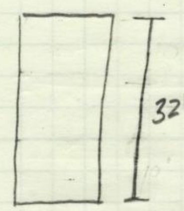
$= 312$ k-ft

use W 21 x 44 = 358 ✓ good

pg 2 of 2

Cantcleaver Class Room East side





From Roof

$[20(1.2) + 32.2(1.6)](11)(\frac{10}{2}) = 4153$ lbs (½ of span goes directly to column O-20)

$M = 46.24k(5) + 4.2k(10)$
$= 273.2$ k ft

use 18x40 $M_u = 294$ k-ft ✓good

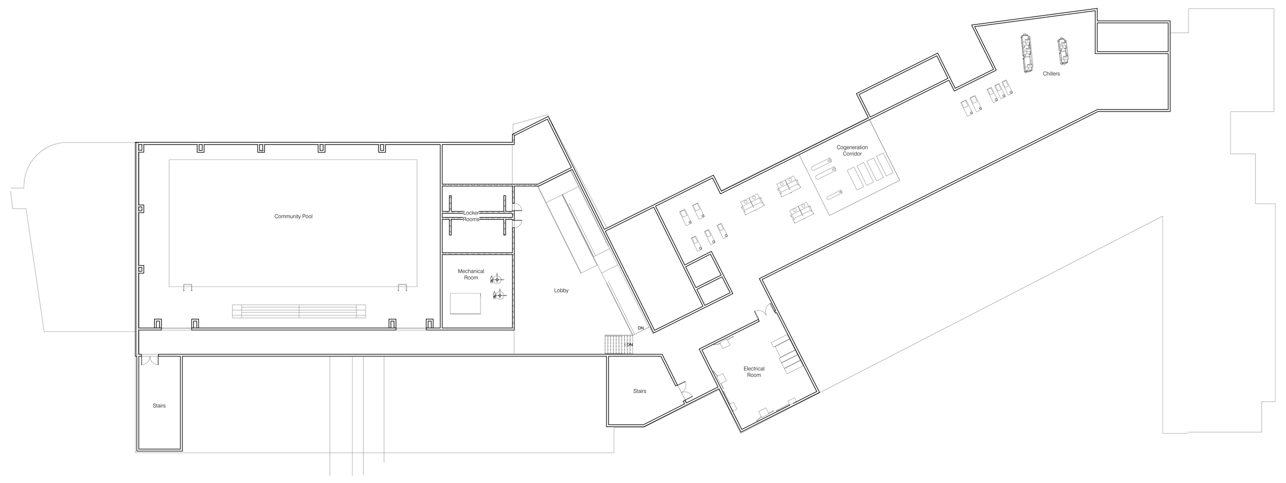
Stage Support

LL = 125 psf
Misc DL = 20psf
DL = 44 psf (3.5" slab)

$[1.2(64) + 1.6(125)]12.5' = 3460$ plf

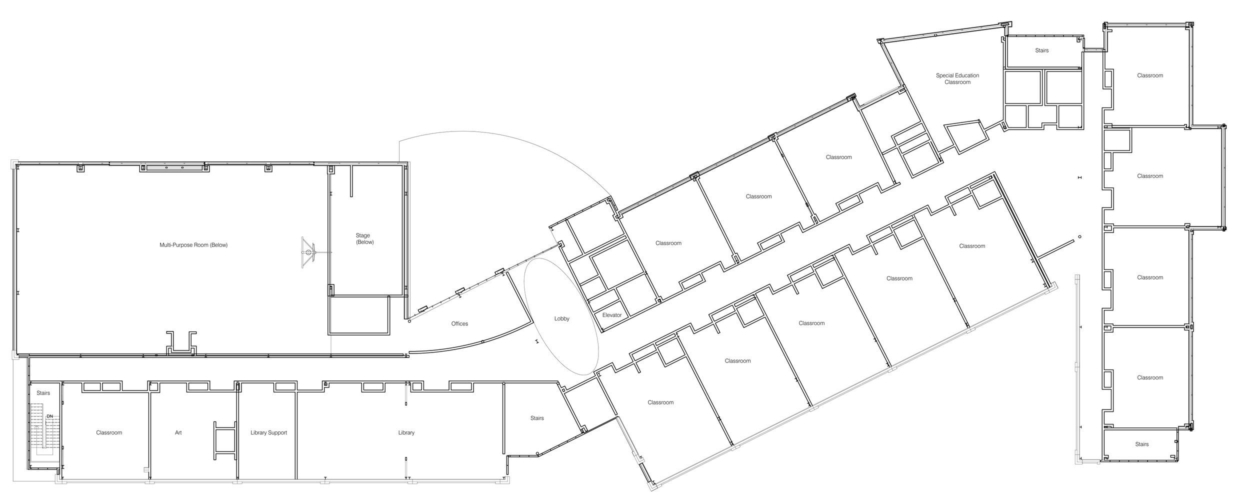
$$M_u = \frac{3460(24)^2}{8} = 249120 \text{ ft-lb}$$
$$= 250 \text{ k-ft}$$

use w16x40 = 274 k-ft ✓



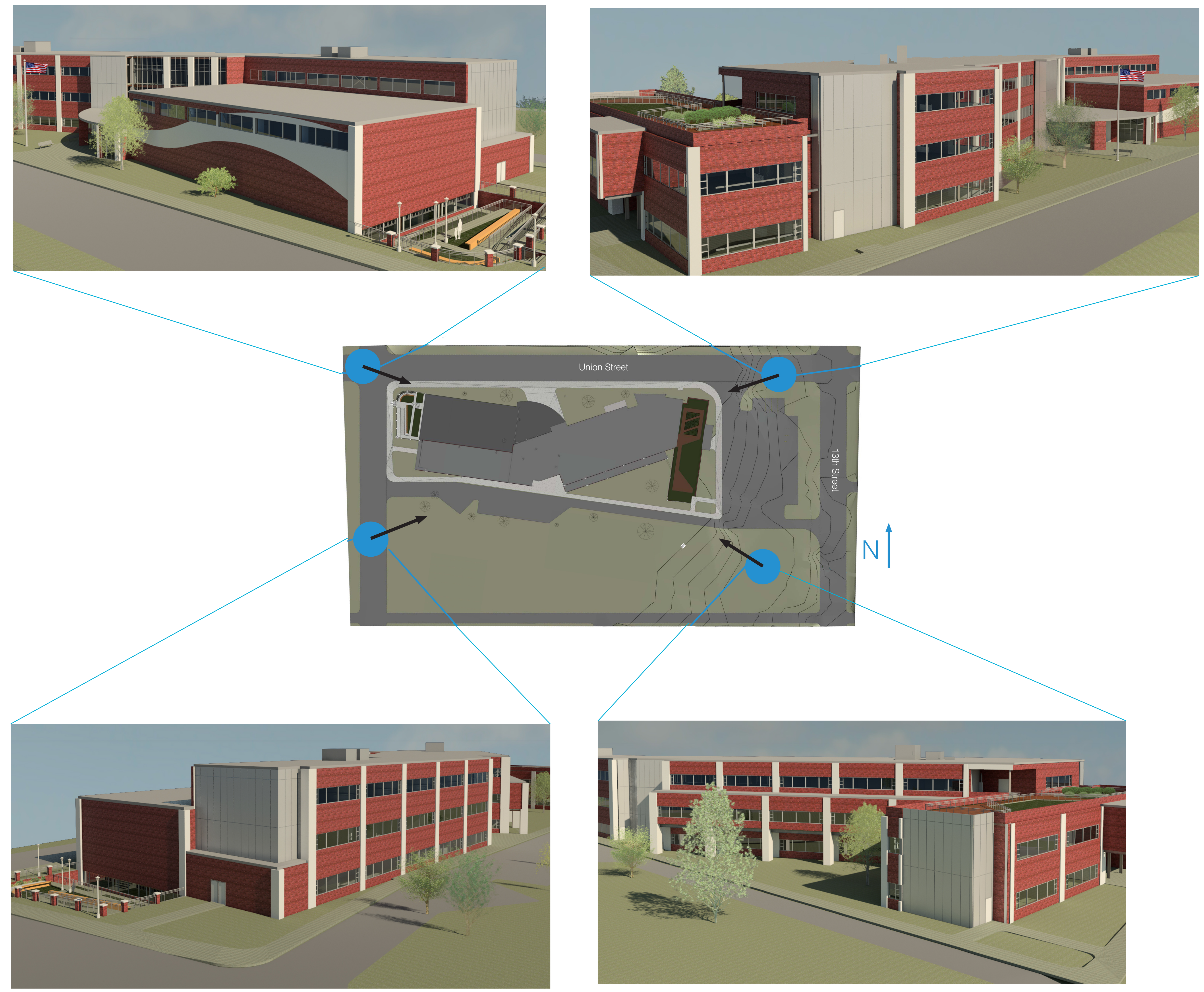
Basement Architectural Plan N

First Floor Architectural Plan N



Second Floor Architectural Plan N

Third Floor Architectural Plan N



Building Overview

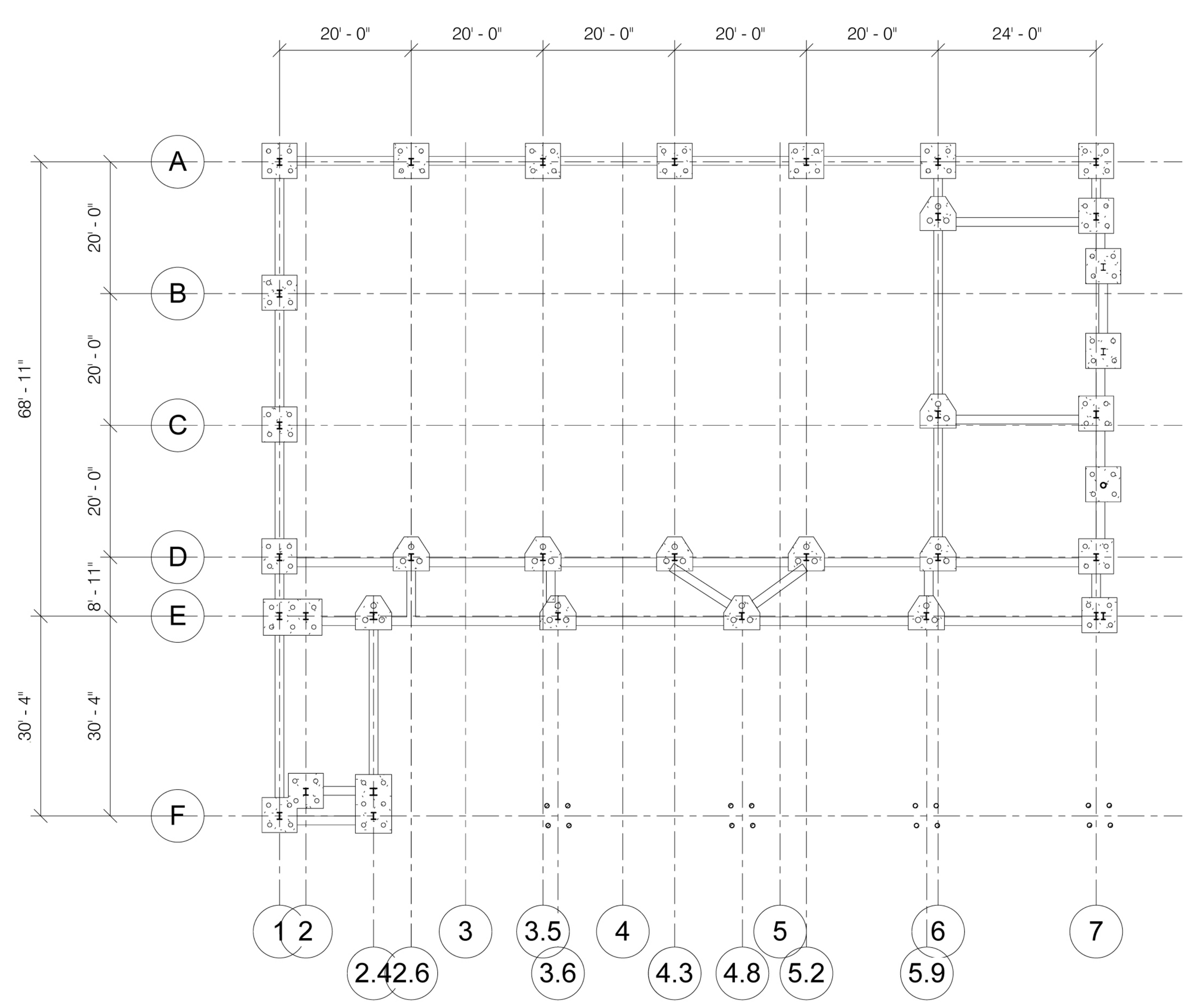
G 100

Team Registration Number 05-2013

ASCE Charles Pankow Foundation Student Competition

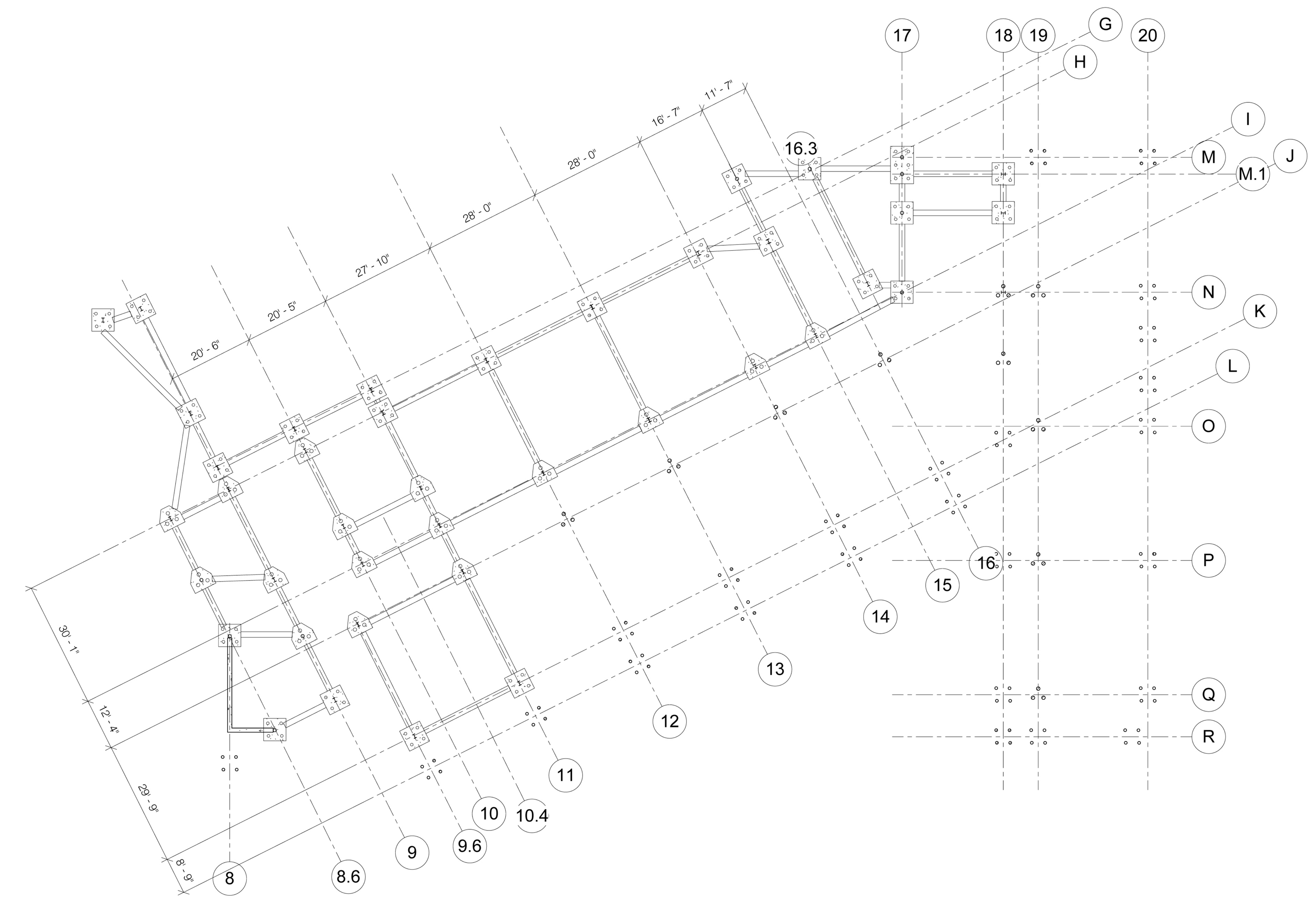
Notes

- West Wing Foundation Plan is at Elevation 347'- 0"
- East Wing Foundation Plan is at Elevation 353' - 4"



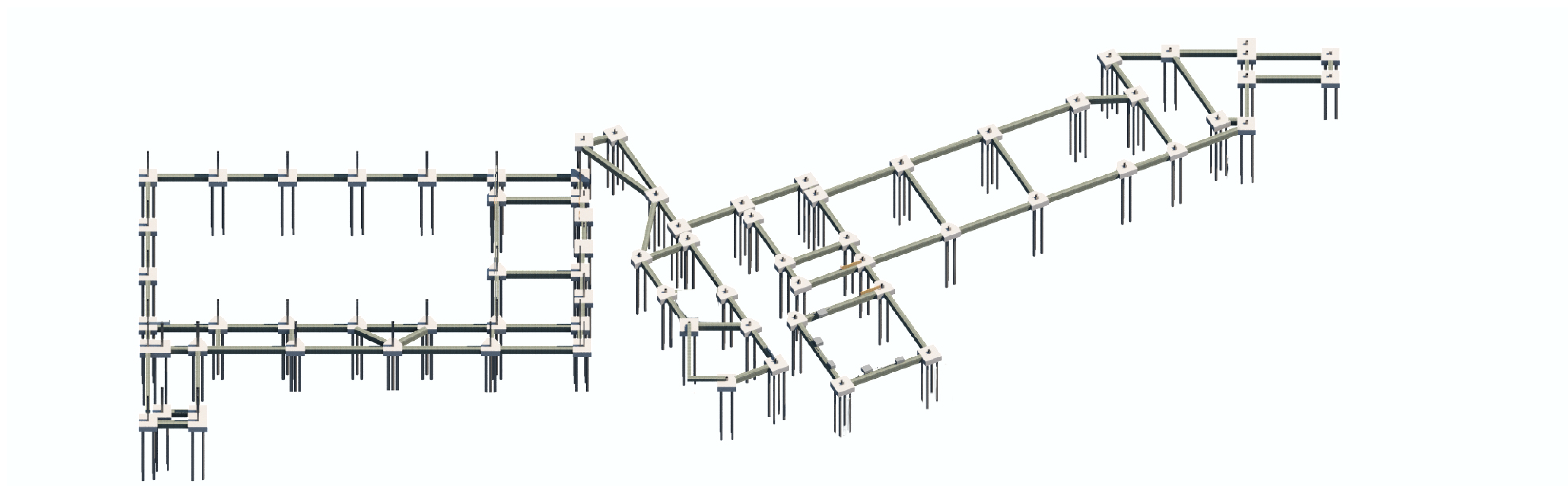
West Wing Foundation Plan

Scale : 1/16"=1'0"



East Wing Foundation Plan

Scale : 1/16"=1'0"



3D Isometric Plan of Pool Lvl and Basment Lvl Foundations

Structural - 100

Team Registration Number 05-2013

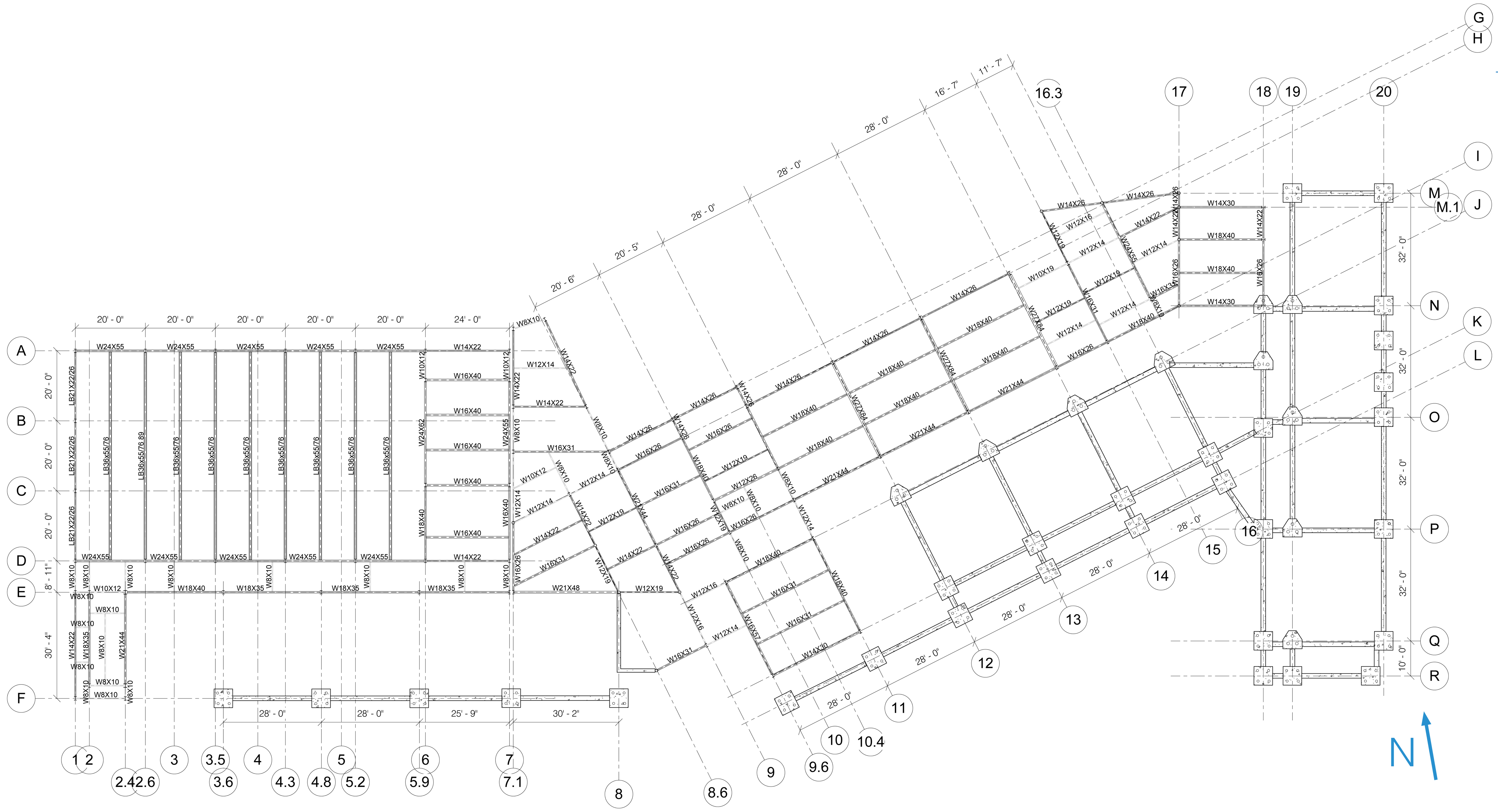
ASCE Charles Pankow Foundation Student Competition

Foundation Plan

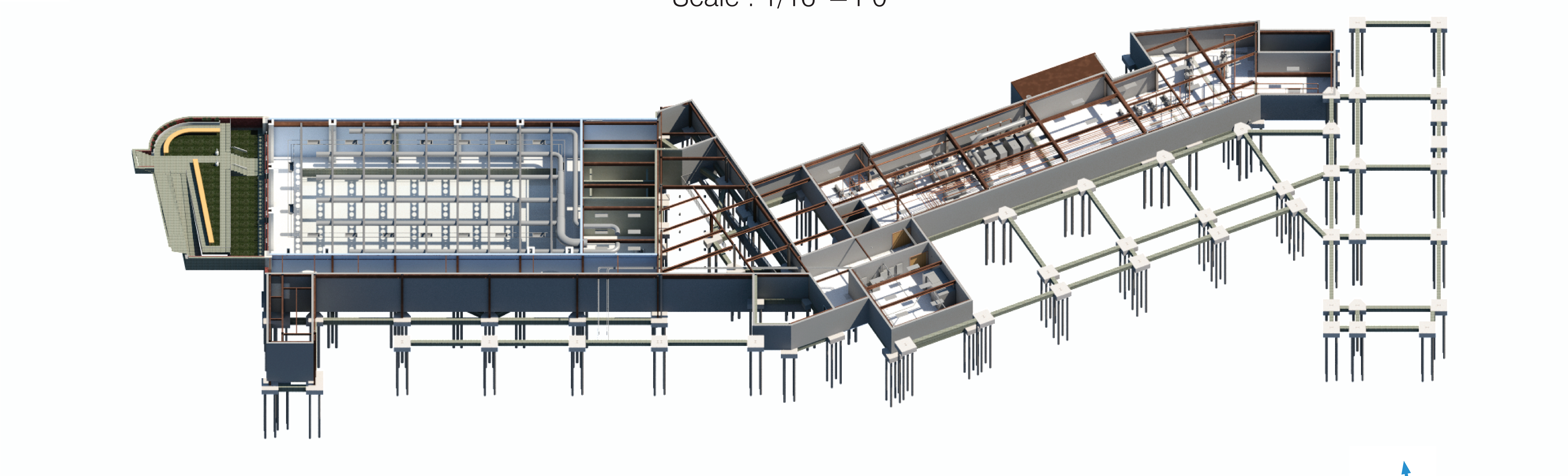
First Floor Framing Plan

Notes

- First Floor Top of Steel is at Elevation 364'-4.5"



First Floor Framing Plan
Scale : 1/16" = 1'0"

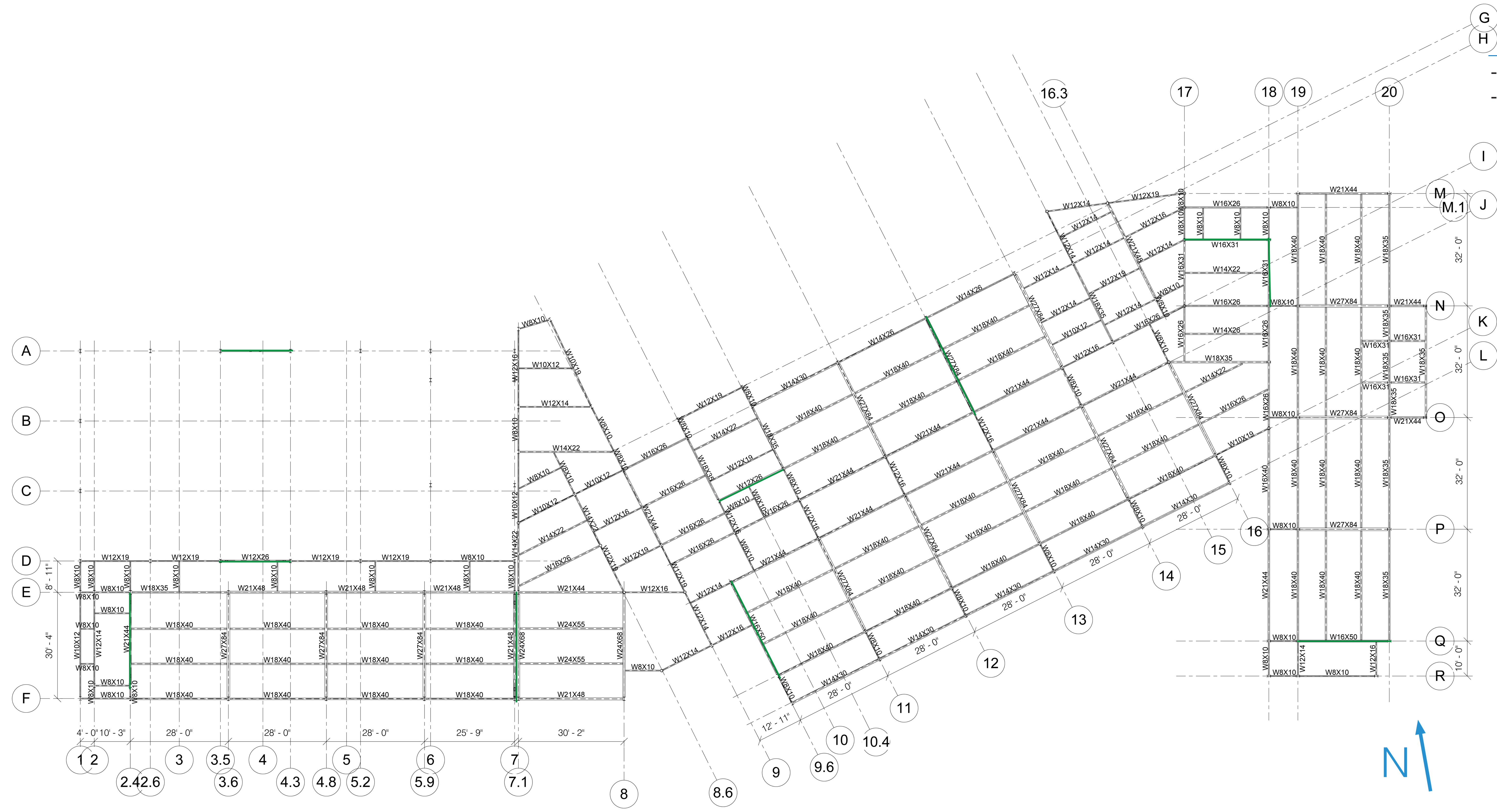


3D Isometric Plan of First Floor

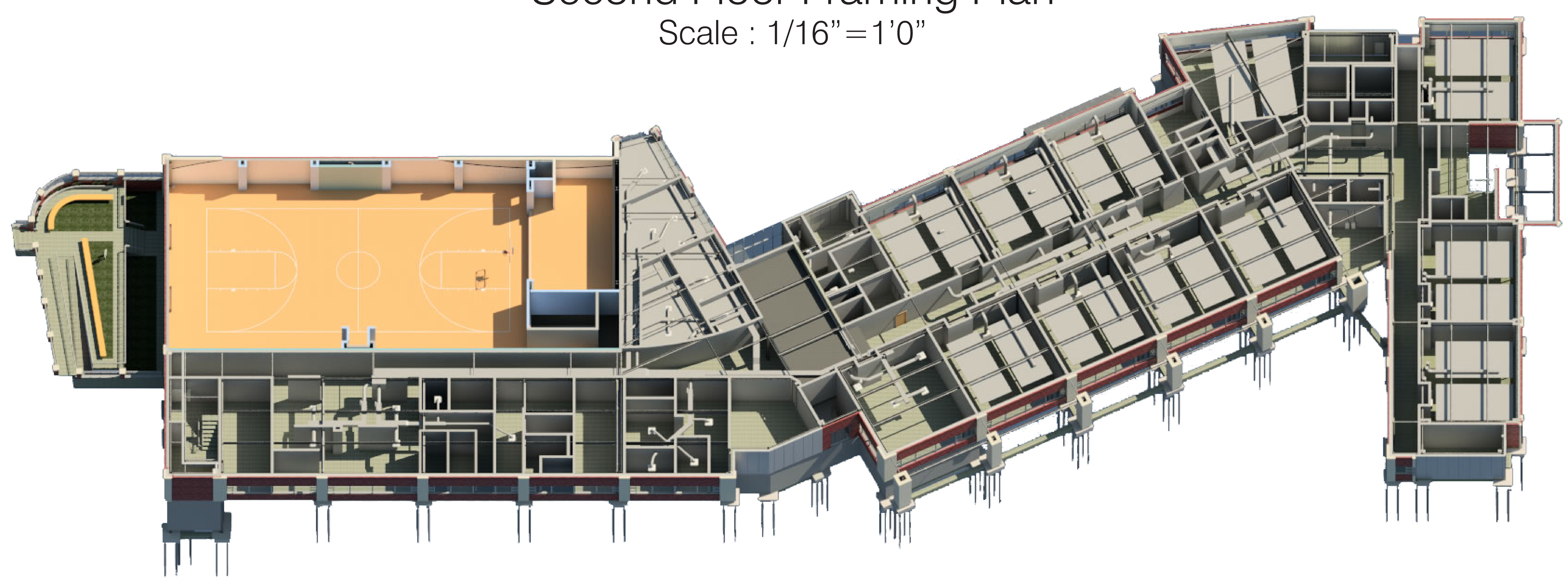
S 101
Team Registration Number 05-2013
ASCE Charles Pankow Foundation Student Competition

Notes

- Second Floor Top of Steel is at Elevation 378'-4.5"
- Green Lines Shown on Second Floor Framing Plan Denotes Lateral Bracing Member Locations



Second Floor Framing Plan
Scale : 1/16" = 1'0"



3D Isometric Plan of Second Floor

Second Floor Framing Plan

S 102

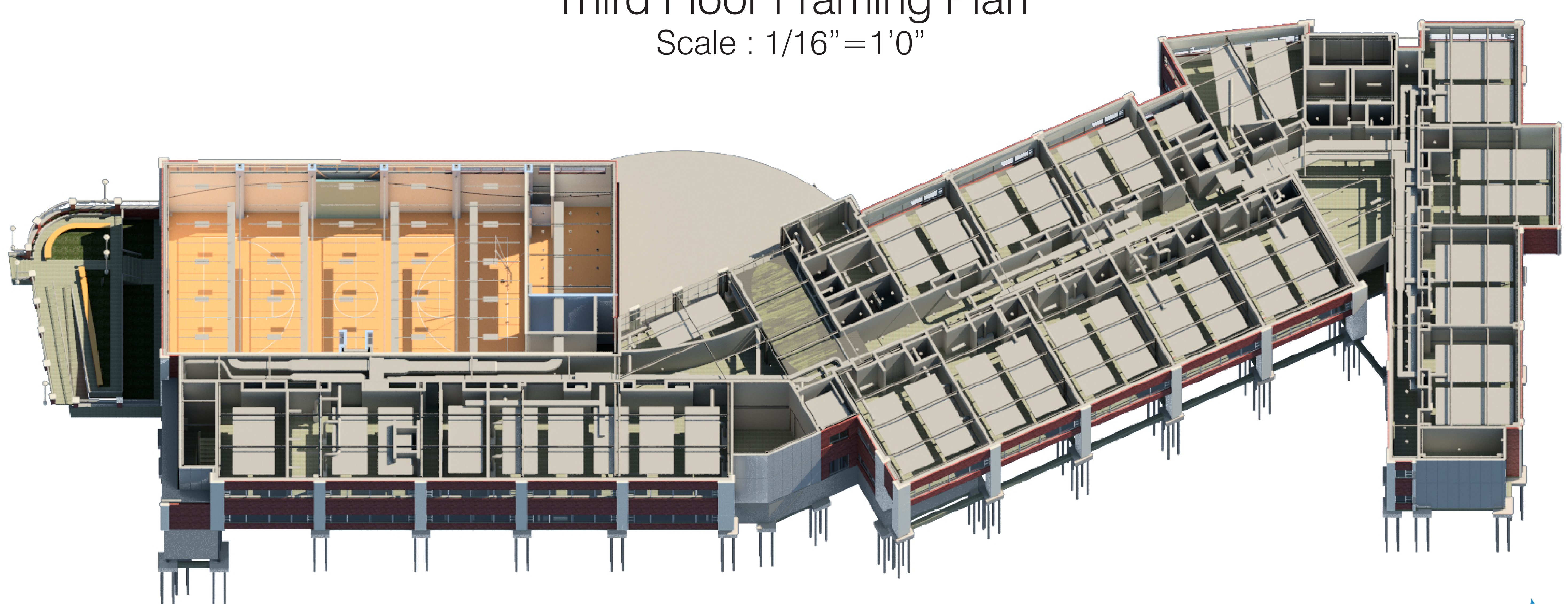
Team Registration Number 05-2013

ASCE Charles Pankow Foundation Student Competition

Notes

- Third Floor Top of Steel is at Elevation 392’-4.5”
- Green Lines Shown on Third Floor Framing Plan Denotes Lateral Bracing Member Locations

Third Floor Framing Plan
Scale : 1/16”=1’0”



3D Isometric Plan of Third Floor

S 103
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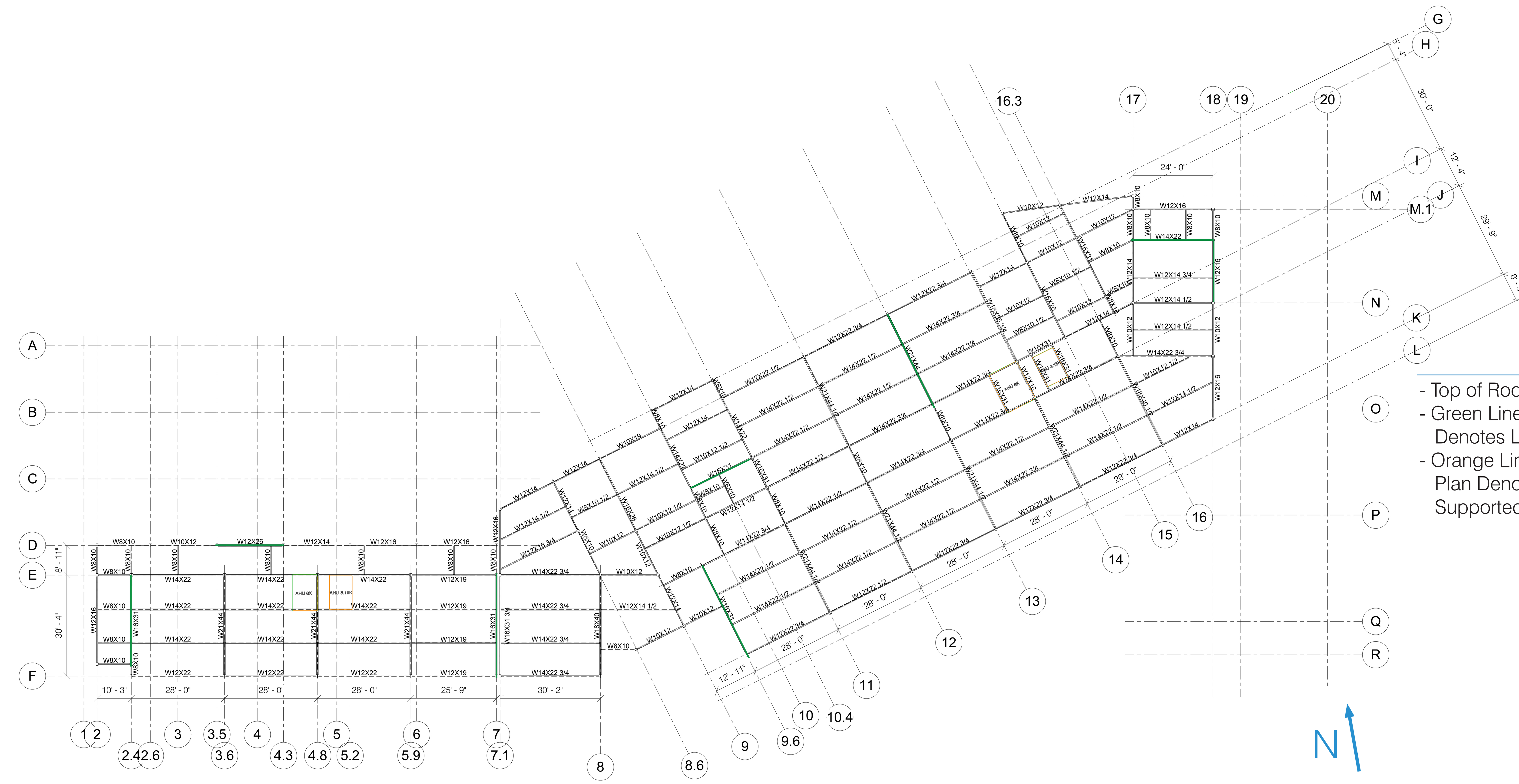
Third Floor Framing Plan

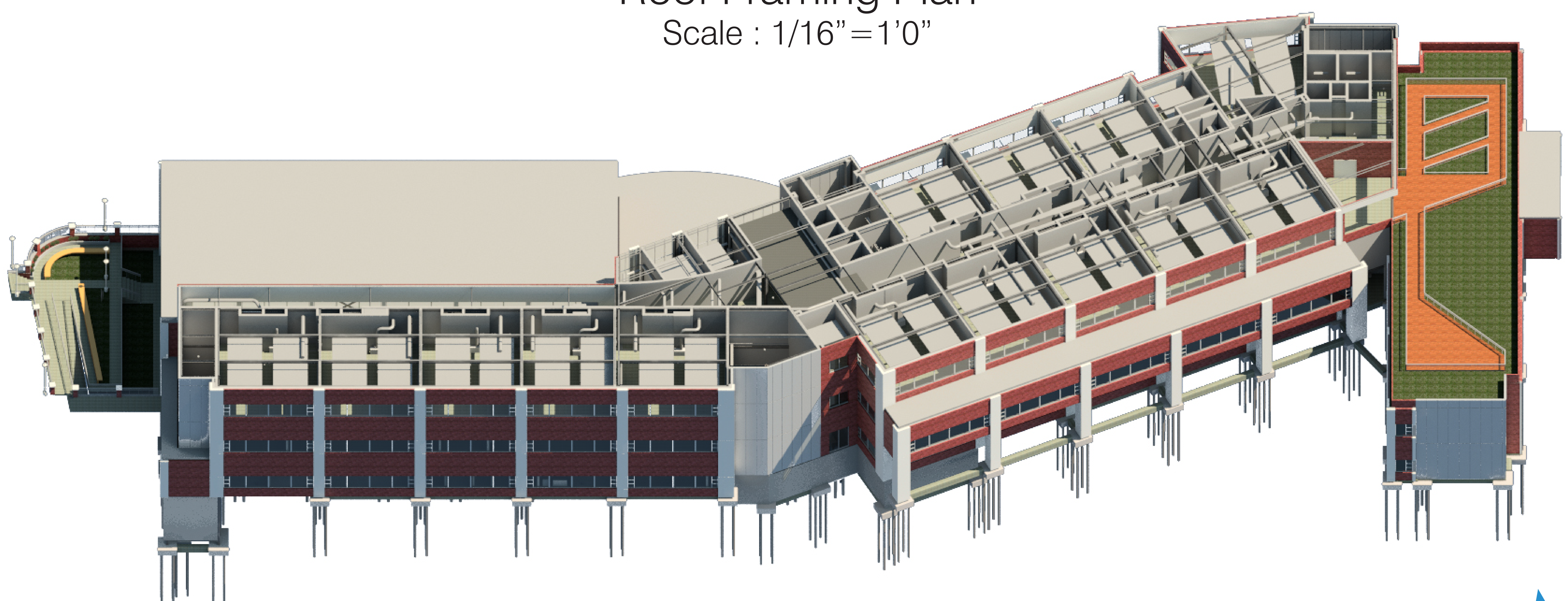


Notes

- Top of Roof is at Elevation 407’- 4.5”
- Green Lines Shown on Roof Framing Plan Denotes Lateral Bracing Member Locations
- Orange Lines Shown on Roof Framing Plan Denotes Outline of Structurally Supported AHU’s

N

Roof Framing Plan
Scale : 1/16”=1’0”



3D Isometric Plan of Roof and Green Roof

N

S 104

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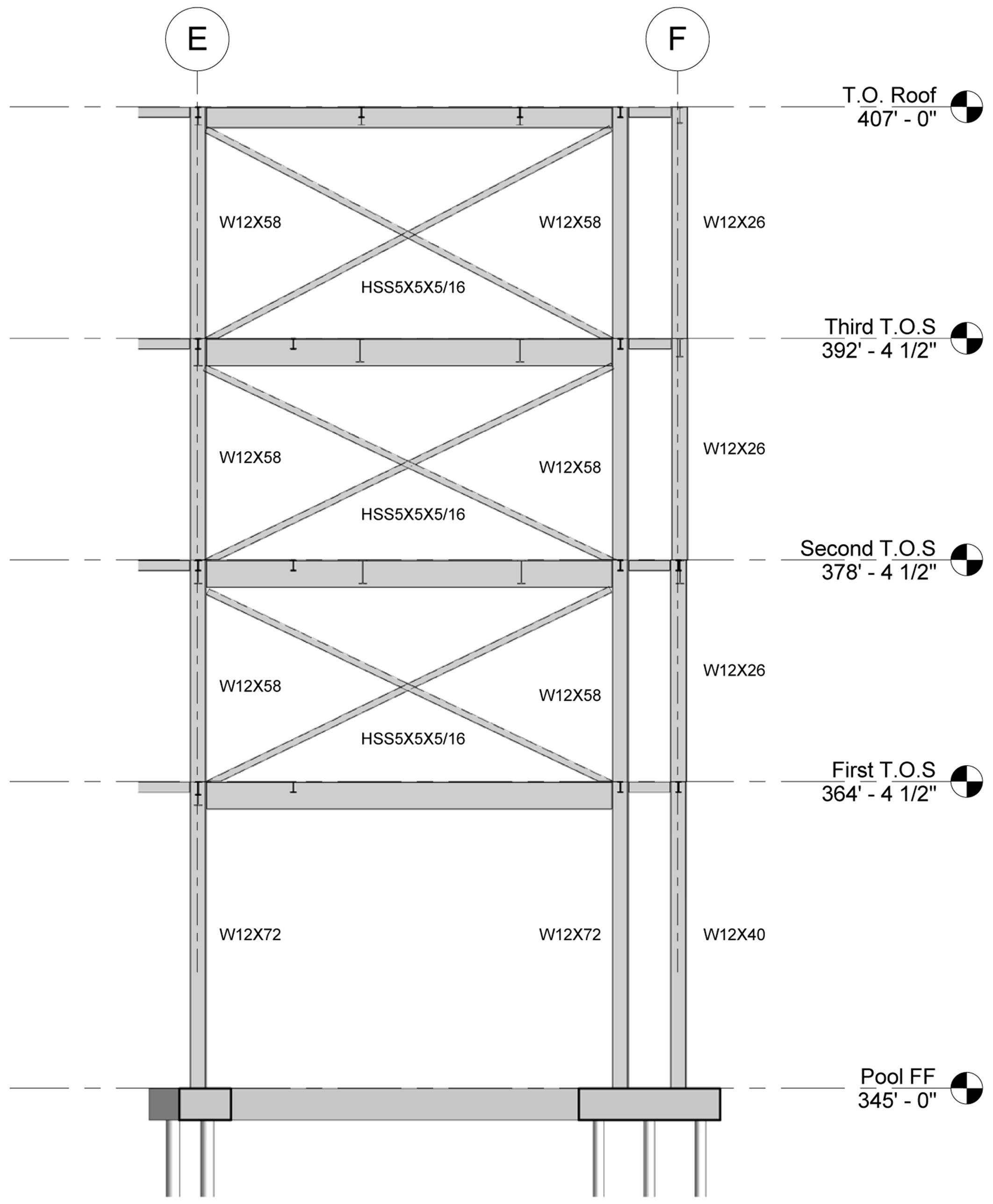
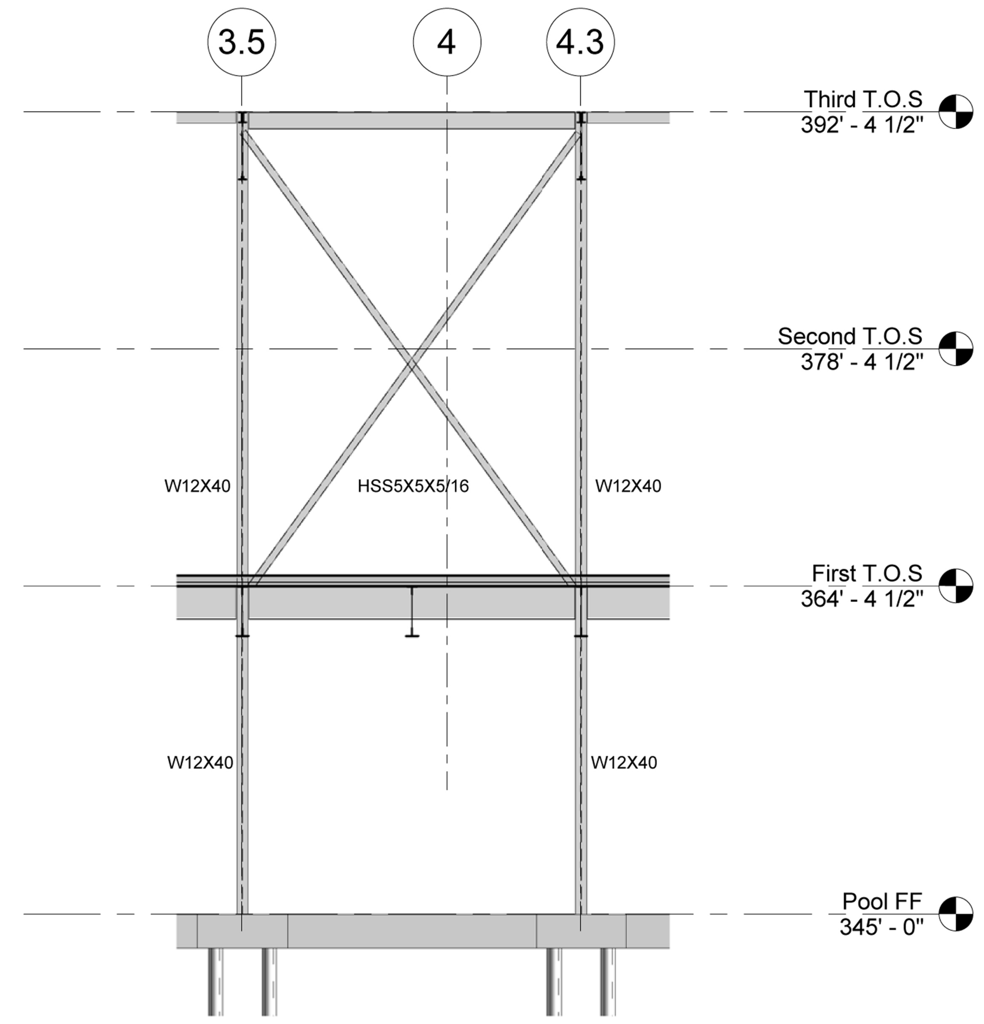
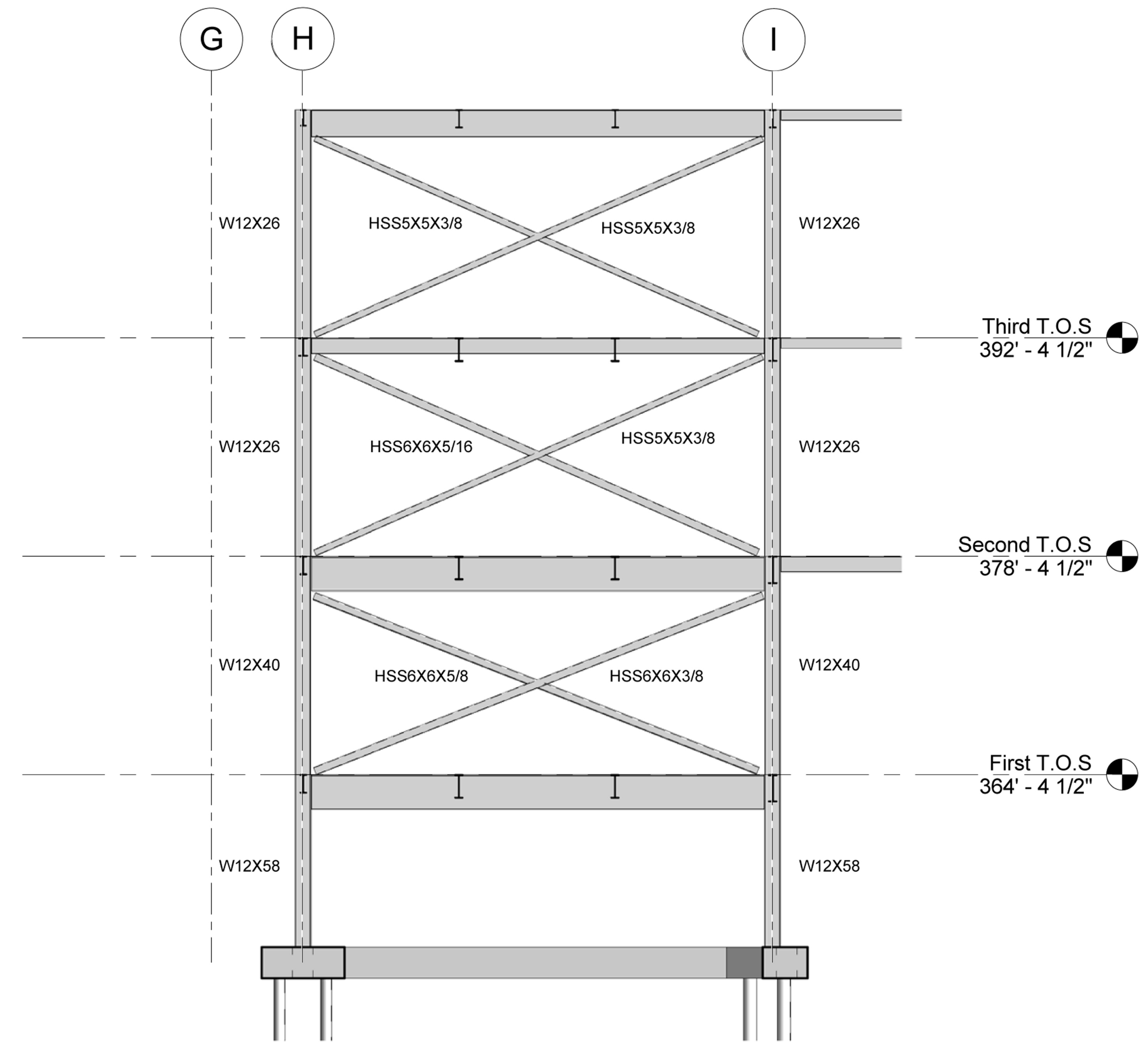
Roof Framing Plan



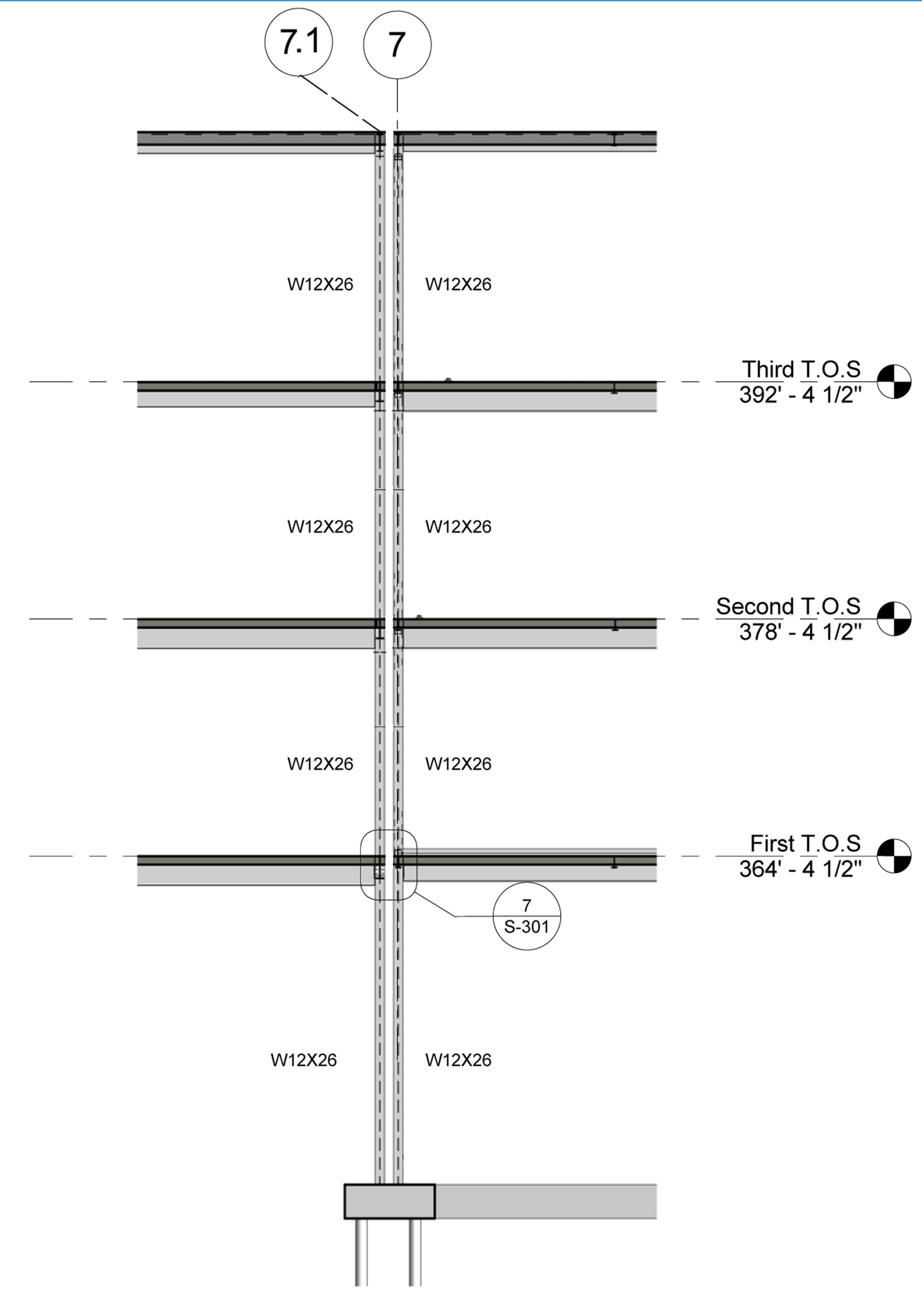
1 Stairwell Bracing
S-301 1/8" = 1'-0"



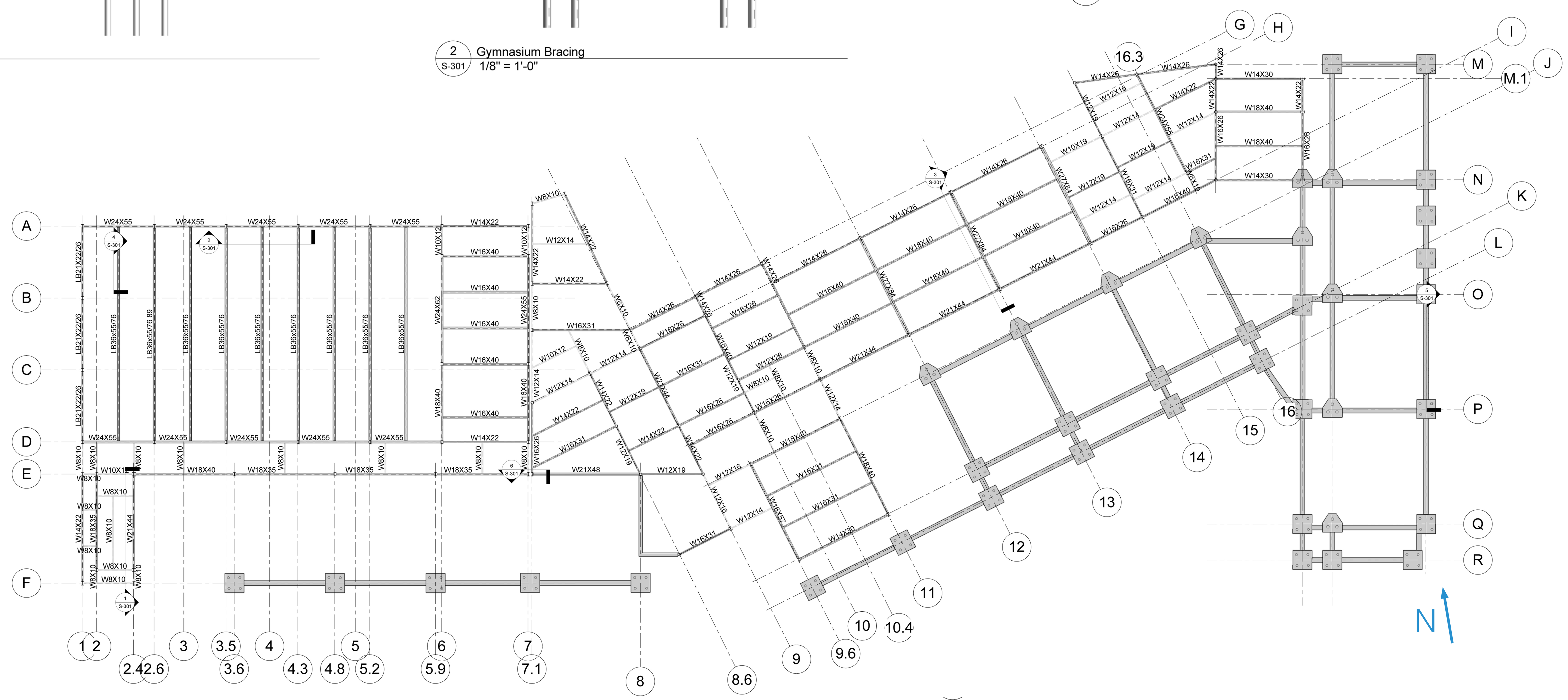
2 Gymnasium Bracing
S-301 1/8" = 1'-0"



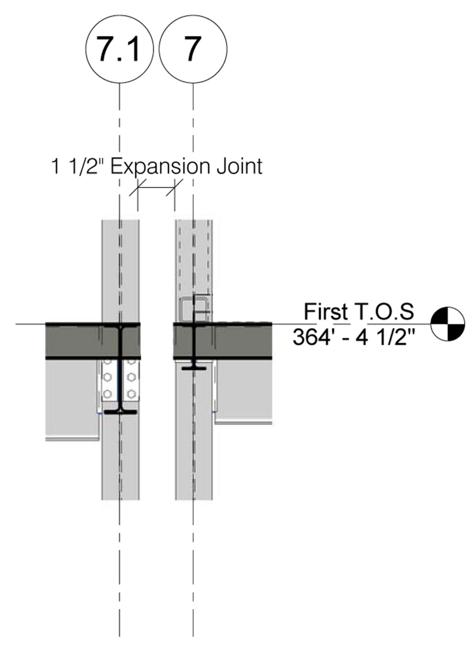
3 Main Wing Classroom Bracing
S-301 1/8" = 1'-0"



6 Building Seperation Expansion Joint
S-301 1/8" = 1'-0"



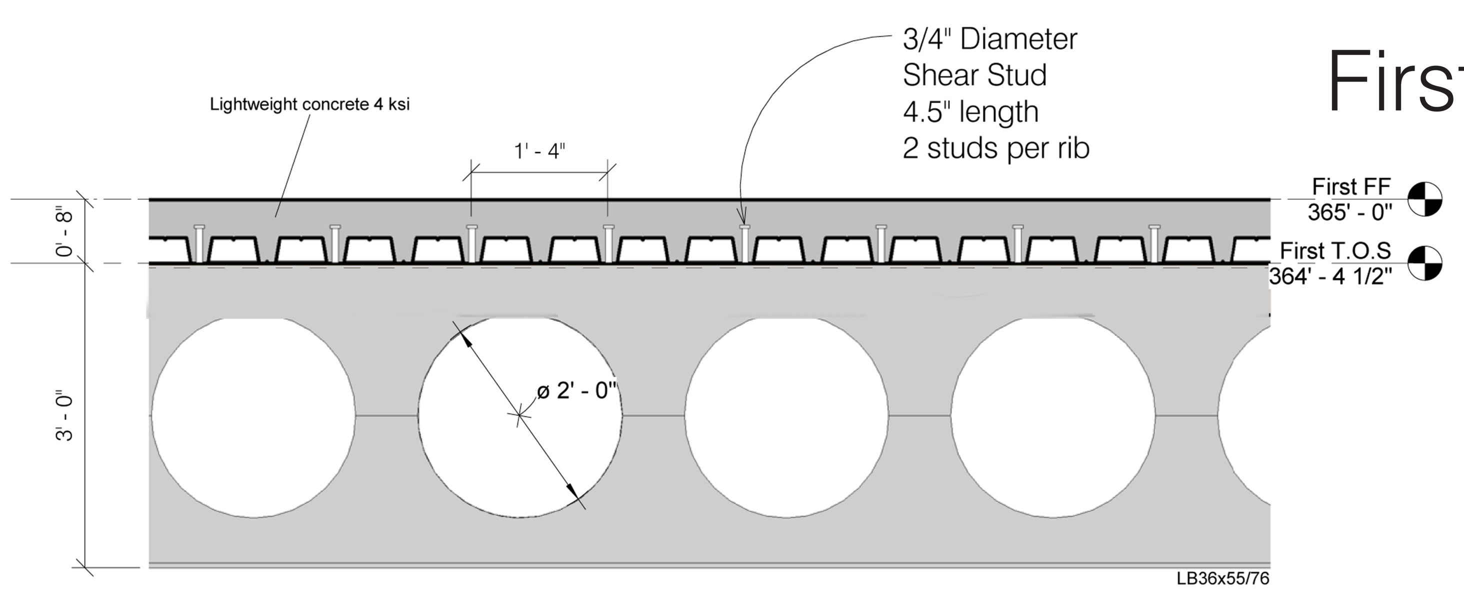
First Floor Framing Plan



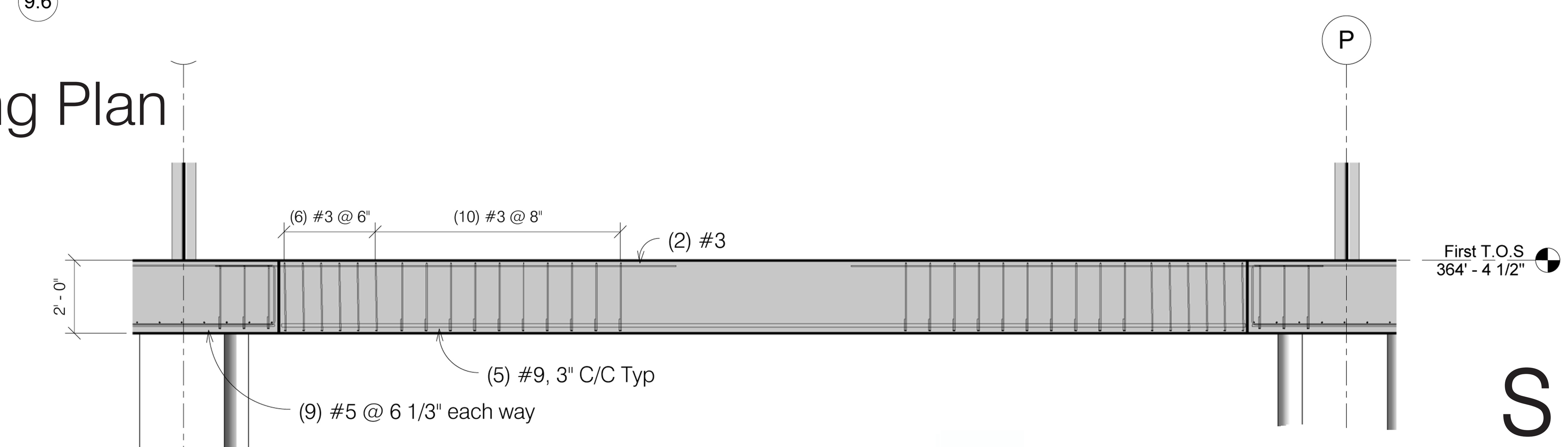
7 Callout of Building Seperation Expansion Joint
S-301 1/2" = 1'-0"

Notes

- Pile Caps Shown on Not to Scale First Floor Framing Plan Support Columns That End at the First Floor Level



4 Cellular Beam Detail
S-301 3/4" = 1'-0"



5 Grade Beam and Pile Cap Detail
S-301 3/8" = 1'-0"

Sections and Details

S 201

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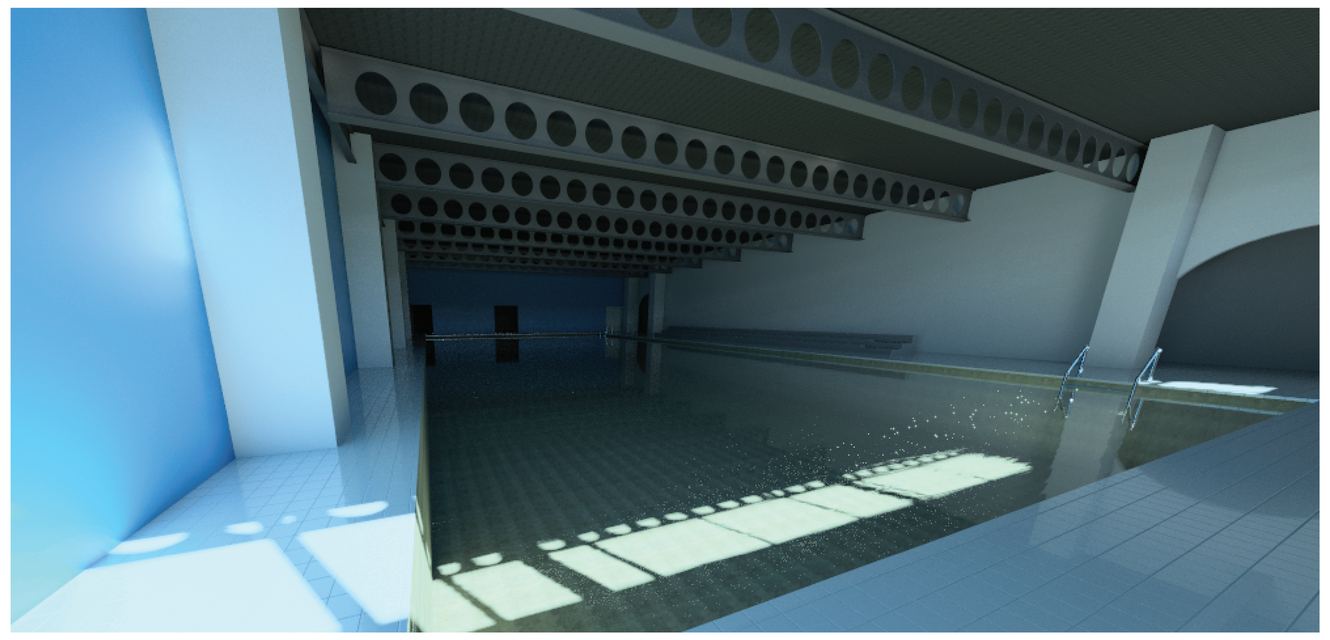
Pool Integration Timeline

1



Rendering showing Cellular Beam layout after coordinating and accomodating for mechanical and lighting systems

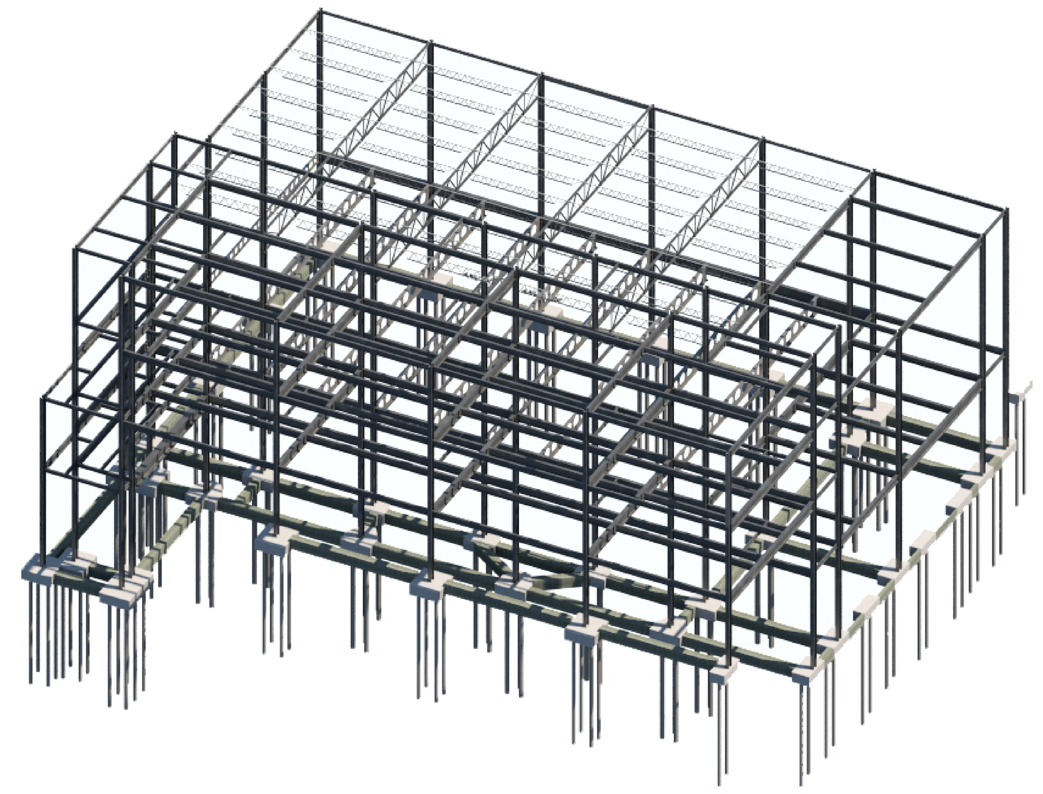
2

Rendering showing Cellular Beam, duct, and lighting integration.

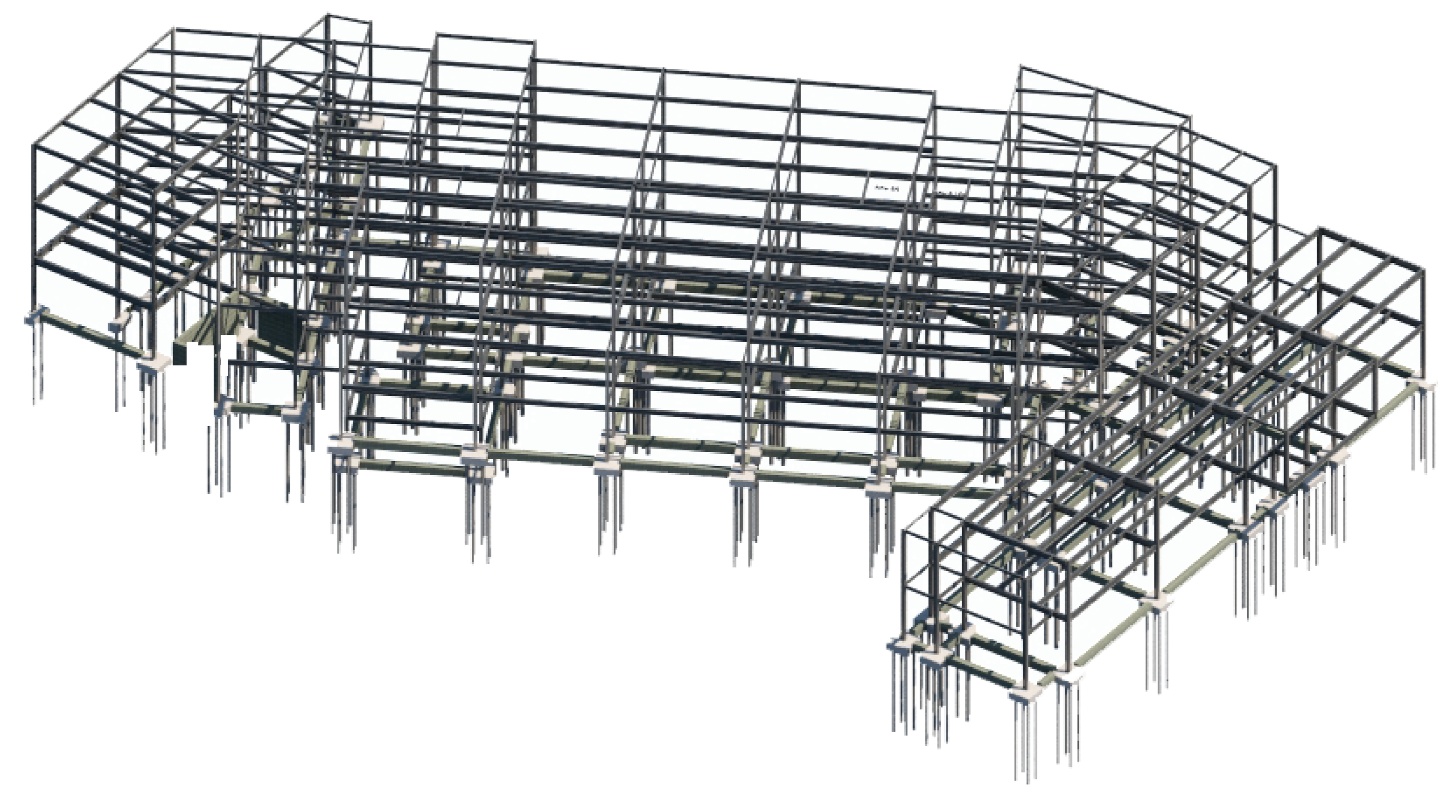
3

Rendering of final community pool area after lighting levels and hieght optimization was coordinated between all disciplines

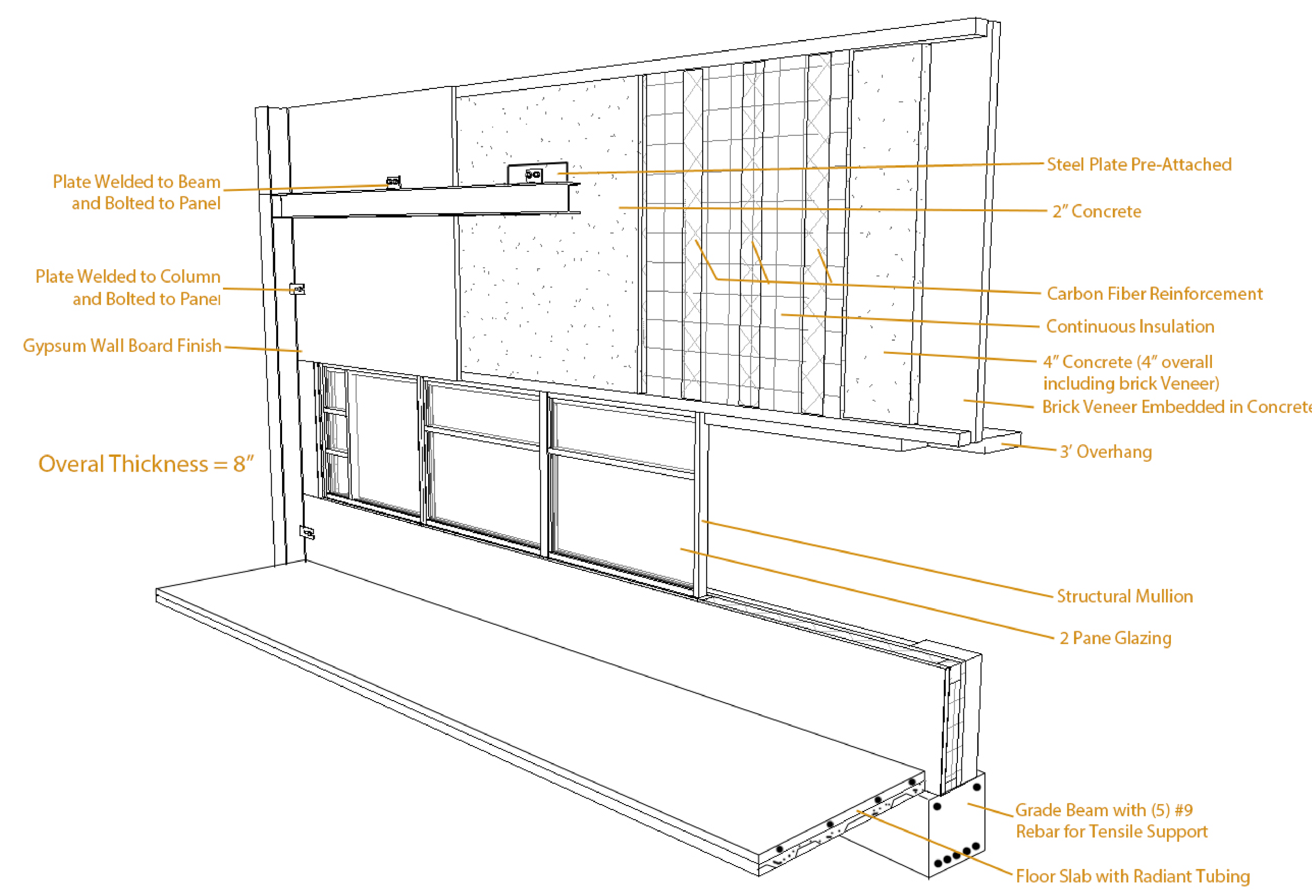
Structural Images



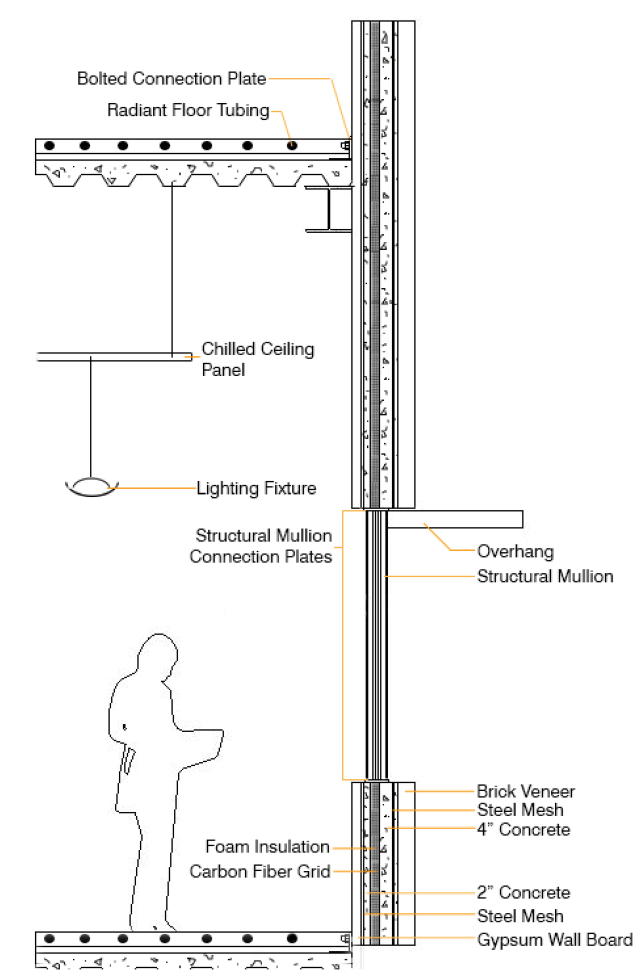
West Wing Structural Frame



East Wing Structural Frame



Pre-Fabricated Panel 3D Isometric Section



Pre-Fabricated Panel Integration Detail

S 202

Team Registration Number 05-2013

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