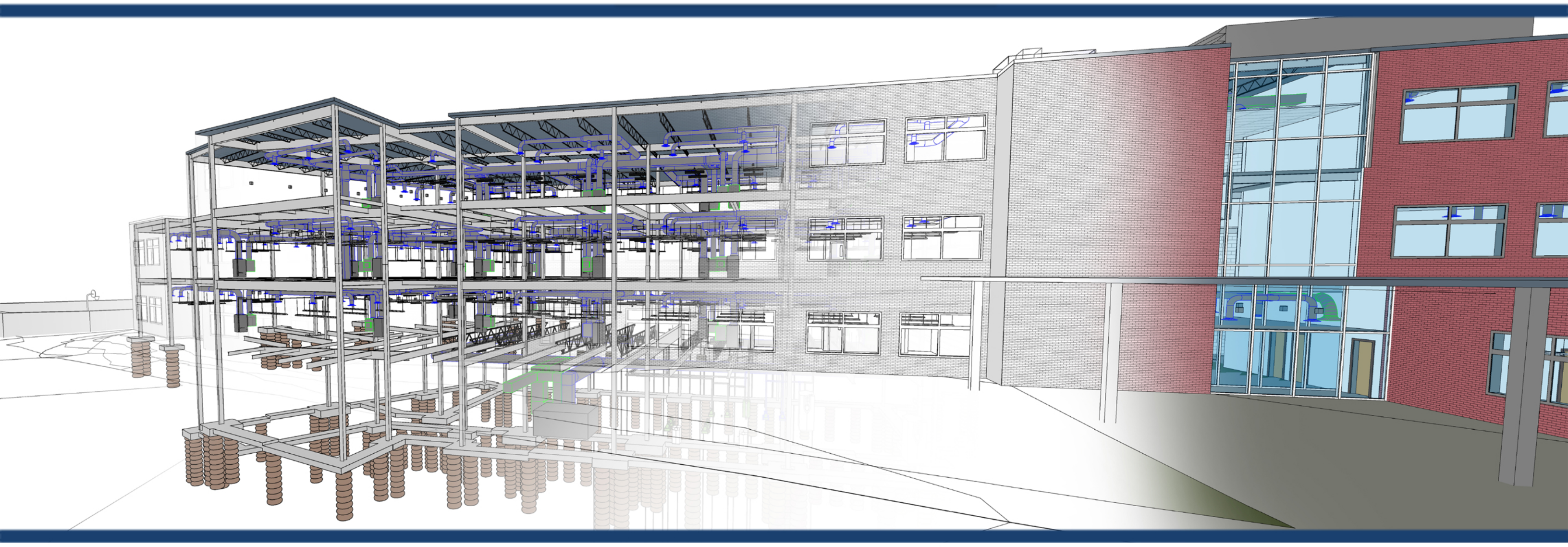
Reading
Elementary
School

AEI Team 10-2013

Structural



February 22, 2013

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SECTION 1: STRUCTURAL NARRATIVE

Project Goals

As a main part of our team's submission for the structural systems component of the 2012 ASCE Charles Pankow Foundation Annual Architectural Engineering Student Design Competition, we wanted to emphasize not only the Pankow Foundation goals, but also our individual team goals as well. The competition description discusses the term "high-performance building" as related to the Energy Independence and Security Act of 2007. This text defines a high-performance building as "a building that integrates and optimizes on a life cycle basis all major high performance attributes, including energy conservation, environment, safety, security, durability, accessibility, cost-benefit, productivity, sustainability, functionality, and operational considerations." These attributes and their relation to our structural systems have been a driving force behind our entire design to ensure our proposed Reading Elementary School is indeed a "high-performance building."

In addition to this, another driving force has been our team-specific goals. When the competition program was first made available, our structural team listed five main goals that we wanted to accomplish through the design. These goals are as follows:

1. Design an innovative and cost-effective foundation.
2. Optimize the design of a gravity structural system.
3. Design and implement an innovative and efficient lateral force-resisting system.
4. Optimize the design of a shelter facility for community use in emergency situations.
5. Create a building information model complete with all structural systems to assist in inter-disciplinary collaboration and graphical representation of the structural design.

The Pankow Foundation states one of its goals as "to improve the quality, efficiency and value of large buildings by advancing innovations in structural components and systems that can be codified." Through striving to accomplish our five goals and applying lessons we have learned along the way (see Appendix A), we strongly believe that our design outcome improves the quality, efficiency, and value of the new Reading Elementary School, while also satisfying the criteria for a "high-performance building."

Structural Systems and Solutions

Foundation Design

As stated above, the first of our major goals was to incorporate an innovative and cost-effective foundation design into the new Reading Elementary School. The provided geotechnical report listed three options for a foundation system: shallow foundations through compaction grouting, shallow foundations through excavation and replacement, or deep foundations through driven piles. Deep foundations were the recommended solution according to the geotechnical report, but our structural team decided to investigate further. We set out to find an alternative innovative foundation system that would prove to be more efficient and cost effective. The Geopier Intermediate Foundation System(1) provided us with the solution we were searching for.

Geopiers

The Geopier foundation system utilizes rammed aggregate piers (RAP's) and prides itself on being "***a breakthrough in foundation engineering***." Treading the line between shallow foundations and deep foundations, Geopiers have a successful track record in areas of karst terrain such as that encountered on the proposed Reading Elementary School site.[1]

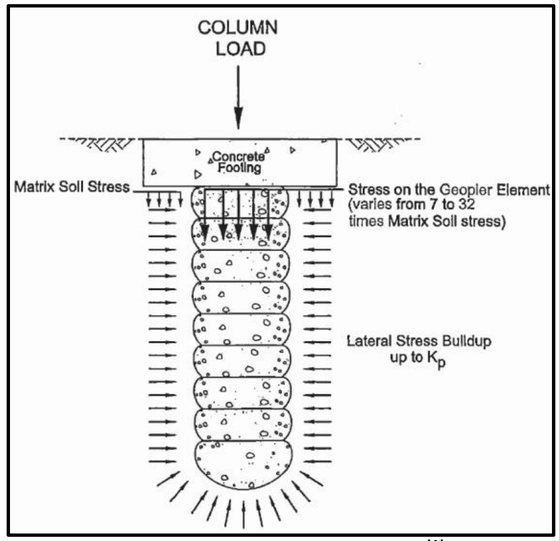
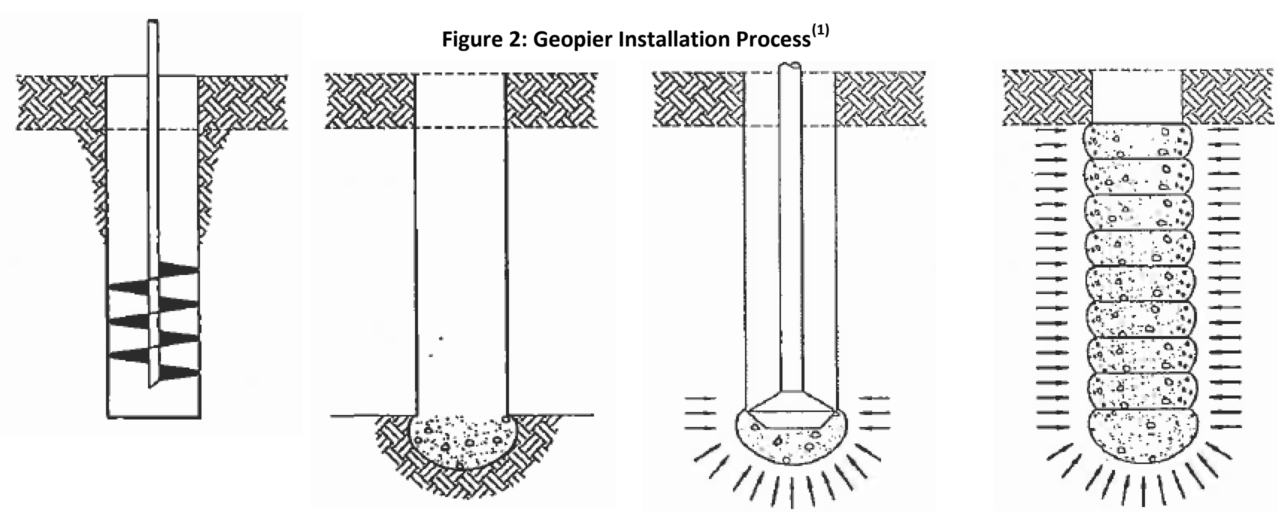


Figure 1: Geopier Load Support[1]

The basic engineering concept behind the Geopier foundation system states that shallow footings will bear on Geopier elements or soil that has been strengthened by the lateral pressures induced by Geopier installation, as seen in Figure 1. The installation process, illustrated in Figure 2, consists of drilling a cavity in the soil to a predetermined design depth. A stiff element is then created at the bottom of the cavity through placement of well-graded aggregate followed by mechanical prestressing (ramming). The Geopier shaft is then completed by filling the cavity with a series of 12-in. aggregate lifts, with impact ramming action between each lift placement. In turn, the lateral soil pressures around the shaft are increased.[1]

Figure 2: Geopier Installation Process[1]



1. Make cavity.

2. Place stone at bottom of cavity.

3. Make a bottom bulb. Densify and vertically prestress matrix soils beneath the bottom bulb.

4. Make undulated-sided Geopier shaft with 12-in. (or less) thick lifts. Build up lateral soil pressures in matrix soil during shaft construction. Use well-graded base course stone in Geopier element shaft above groundwater levels.

Based on the site soil conditions and design tables in the Geopier Soil Reinforcement Manual, we have designed our spread footings for an allowable soil bearing capacity of 5,000 psf.[1] According to project records and testing, maximum column loads which may be safely supported using Geopiers, even in poor subsoil areas, generally exceed 400 kips—approximately 130% of the anticipated maximum column loads for the Reading Elementary School.[1]

Table 1: Geopier Design Summary

Type	Avg. Load (k or k/ft)	Geopiers Req'd.	Avg. Depth (ft)	Total Lineal Feet (ft)
Isolated Column Footings	132 k	150	10.6	1590
B.5 Bearing Wall	15 k/ft	21	10	210
B.8 Bearing Wall	6 k/ft	15	10	150
Other Bearing Walls	3 k/ft	16	10	160
Multipurpose Area Walls	4 k/ft	20	10	200
Total		222	10.4	2310

Our team obtained a copy of the Geopier Soil Reinforcement Manual, which provides general guidelines for determining the number and location of aggregate piers needed in a design.(1,2) The number of Geopier elements below a particular isolated column footing is dependent upon the total unfactored column load and the capacity of each Geopier element. Column loads were determined using Bentley Engineering's RAM Structural System software package (discussed further in the Gravity Systems Design Section). After creating spreadsheets to calculate the necessary parameters to design the system, our team calculated a total of 150 Geopier elements under column footings at an average depth of approximately 11' per element. Detailed spreadsheets of the Geopier support calculations can be seen in Appendix C. Design of the Geopier elements to support the bearing walls in the basement and around the multipurpose area was much less complicated and was calculated manually (also shown in Appendix C). Overall, the design yielded a total of 222 Geopier elements at an average depth of approximately 10' as seen in Table 1. A 3-D detail of a typical isolated footing with Geopier support taken from our Revit building information model can be seen in Figure 3.

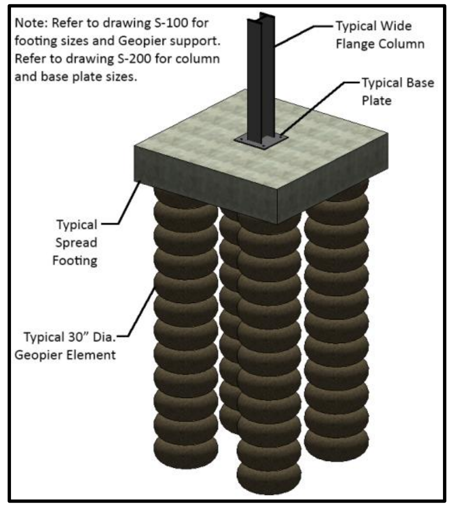


Figure 3: Typical Isolated Footing Detail

Design Rationale/Benefits

The Geopier foundation system exemplifies the Pankow Foundation's idea of improving quality, efficiency, and value of the entire building. With Geopiers, the building is provided with additional earthquake protection for shallow footings through special aggregate drain design. Special stone gradation can be provided to increase permeability, allowing the Geopier elements to relieve pore water pressures that may be induced during a seismic event. This aggregate drain design provides a practical solution to liquefaction hazards, should they arise.(1) This concept is especially applicable to the project site in Reading, a region characterized by higher seismic demands per ASCE 7-10 than its surrounding areas in the state of Pennsylvania.(3)

Also adding to the quality of the Geopier foundation system is the benefit of settlement control. Spanning the gap between deep and shallow foundations, prior projects have been successful in a variety of poor soil conditions, including uncompacted fills like those found on the proposed elementary school site. On recorded projects that have utilized Geopiers, there has not been an instance where settlements have exceeded design settlement expectations. In calculating the total settlement (seen in Appendix C), it was determined that the maximum anticipated settlement would approach 0.7" at footings B.5-14 and B.5-15 (see drawing S-100), which meets the maximum allowable value of 1" recommended by the Geopier Soil Reinforcement Manual.(1)

Not only does the Geopier system provide a high-quality foundation option, but it also doubles as a very efficient and cost-effective solution as well. Geopiers can be installed very quickly, taking approximately 20 minutes to install a 10' deep element.(1) Also, because an allowable soil bearing capacity of 5,000 psf can be used with the Geopier foundation system on our site, material costs of concrete and reinforcing for footings is decreased by approximately 40% when compared to excavation and replacement material costs. Due to the exceptional settlement control that Geopiers provide, future potential costs due to settlement problems are also minimized. All these aspects help to accomplish our first structural goal of an innovative and cost-efficient foundation system.

Comparison of Alternative Solutions

When looking at foundation options, there were a number of reasons why Geopiers were chosen over the alternatives presented in the geotechnical report. As an alternative solution, the process of compaction grouting has a multitude of question marks when it comes to cost and scheduling. Because grout is pumped until a certain pressure is reached, it is nearly impossible to determine the amount of grout needed unless borings are taken at extremely close intervals. After performing some research, our team discovered that there have been documented cases in areas of similar karst terrain where compaction grouting has not performed well because unexpected volumes of grout were needed after low pressures upon initial pumping. For example, according to Pennsylvania Centre Region Code Administration Director Walter G.M. Schneider III, construction of the Mount Nittany Medical Center in Pennsylvania experienced similar problems when trying to utilize compaction grouting in karst terrain.(4) These types of issues can wind up costing the project not only money, but time as well due to scheduling delays. On a project such as an elementary school where schedule deadlines must be met to ensure school can start on time, construction delays must be avoided at all costs. All factors considered, soil treatment by methods of compaction grouting was a risk that our design team did not want to take.

When considering excavation and replacement with proper compaction, there were a number of pros and cons that needed to be weighed against the Geopier option. Although full excavation and replacement eliminates questions about subsurface conditions to an extent, it requires a great deal of money and time. Referring back to our first goal of an innovative and cost-effective foundation system, excavation with replacement did not coincide with this goal. Based on the geotechnical report, an estimated 30,000 cubic yards of fill would need to be excavated and replaced, providing an allowable soil bearing capacity of 3,000 psf (60% of that achieved with Geopiers). Through our research, we found that at excavation depths greater than 5 feet, Geopier support is frequently less expensive and less prone to unforeseen costs or problems compared to excavation and replacement.(1) After running a cost estimate for excavation and replacement which included excavation, backfill, compaction, and reinforced concrete for footings, the total cost came to approximately $334,000, as seen in Table 2. Overall, Geopiers provide a much more cost-efficient and time-saving alternative, with a total cost of $203,000 based on historical cost data.(2) It should be noted that the aforementioned cost comparison is based on soil bearing capacities of 3000 psf and 5000 psf for excavation/replacement and Geopier support, respectively.

Table 2: Cost Summary for Foundation System Alternatives

Foundation System	Excavation/Replacement	Micropiles	Geopiers
Estimated Cost	$334,000[a]	$292,000[b]	$203,000[c]

[a]Includes excavation, backfill, compaction, reinforced concrete (footings), and labor
[b]Includes steel piles, pile driving, concrete, and labor
[c]Includes drilling, ramming, aggregate, reinforced concrete (footings), and labor

The most detailed comparison study that was made in the foundation design was that between Geopiers and micropiles. When our team first saw that micropiles were the recommended method, we were skeptical as to whether deep foundations were necessary for only a three-story elementary school, thus causing us to research for innovative alternatives. It should be noted that because micropiles were the suggested method from the geotechnical report, we carefully reviewed the use of Geopiers in our design. We found evidence that stated Geopiers are generally more economical than piles when bedrock is more than 25 feet deep, as a general rule of thumb.(1) Per the geotechnical report, the majority of piles (should they be selected) would achieve bearing on bedrock within 25 to 40 feet of the pile cap bottom.

Like compaction grouting, the installation of micropiles also comes with some uncertainty. Since soil borings cannot feasibly or cheaply be taken in dense increments on site, there is bound to be uncertainty as to depth of bedrock at most column locations. For budgeting purposes, lengths of the steel shafts as well as volume of concrete needed remain uncertain unless bedrock is cored at very close intervals.

However, for comparison purposes, our team performed a cost estimate for micropiles which included material costs for the concrete-filled steel piles as well as the labor involved in installation. Total cost proved to be more expensive than Geopiers, coming out to approximately $292,000, as seen in Table 2.

Sinkhole Mitigation

It cannot be overlooked that our site is in a region of karst topography and is prone to sinkholes. No matter which foundation system is selected, there is a potential for problems to arise both during construction, and after the building is erected. The best way to avoid a problem in the field is to be knowledgeable of the risks and choose designs that will mitigate complications.

If a sinkhole were to occur on site, there are several actions our team is prepared to take. If encountered during Geopier shaft drilling, the existing void can be filled with a low-cement-and-sand mixture or compacted soil and the shaft can be re-drilled. In the event a sinkhole occurs after construction is complete, the risk of foundation damage can be minimized through the use of cement-treated aggregate in the Geopier elements. In essence, a Geopier element is an assortment of stones supported on all sides by compressed soil due to ramming. If a sinkhole occurred and soil was removed, the Geopier element would have a tendency to collapse into the void. However, by using cement-treated aggregate in the Geopier element, the risk of collapse is substantially reduced because the element is being held together and in place by the cement, thus increasing stability.

One of the biggest contributing factors to a sinkhole void collapse is improper storm water management. Construction will be managed so as to direct as much water away from the foundation area as possible. Impermeable areas such as the building's roof and the parking lot can produce large concentrated amounts of water after a rain storm which can help fuel sinkhole activity. Through managing the runoff from parking lots and ensuring that roofwater is not drained into concentrated locations, our team believes we can reduce the potential of sinkhole development on site.

Footings

Table 3: Footing Schedule

FOOTING SCHEDULE				
MARK	SIZE	DEPTH	REINFORCING	GEOPIER DEPTH
F40	4'-0" x 4'-0"	1'-0"	(7) #5 BARS EACH WAY	9'-0"
F50	5'-0" x 5'-0"	1'-0"	(7) #5 BARS EACH WAY	10'-0"
F60	6'-0" x 6'-0"	1'-6"	(8) #5 BARS EACH WAY	11'-0"
F70	7'-0" x 7'-0"	1'-6"	(9) #5 BARS EACH WAY	12'-0"
F80	8'-0" x 8'-0"	2'-0"	(14) #5 BARS EACH WAY	13'-0"

Reinforced spread footings for columns and strip footings for the basement walls are included in the foundation in conjunction with the Geopier support. Bottom of footings will be placed at least 36" below grade in accordance with the local frost depth. Our structural team used the RAM Structural System software package (listed in Appendix B along with other utilized software and codes) to design the isolated column footings and strip footings, and these isolated footings range in size from (4'-0" x 4'-0" to 8'-0" x 8'-0"). The footing schedule for the building can be seen in Table 3 as well as drawing S-100. For constructability purposes, our structural team has standardized this footing schedule to minimize the different sizes of footings and reinforcement. During construction, it is often the case where standardization helps speed up productivity, making certain that the added cost of materials does not exceed the money saved due to repetition in construction easement. Standardization also minimizes the risk of construction errors in the field. A detailed foundation plan is provided on drawing S-100 showing footing locations along with a footing schedule, Geopier support schedule, applicable notes, and a 3-D foundation image.

Gravity Systems Design

Floor and Roof Framing Design

To aid in the design of the elementary school's gravity system, our team utilized Bentley Engineering's RAM Structural System software package. Our 3-D RAM model can be seen in Figure 4 complete with annotations and nomenclature as it applies to our team's overall school design. Of extremely important note is the fact that the school has been designed completely separate from the multipurpose area/shelter, mainly to isolate the different risk categories. Therefore, the multipurpose area is not visible in Figure 4 and is further discussed in the Specialized Areas section.

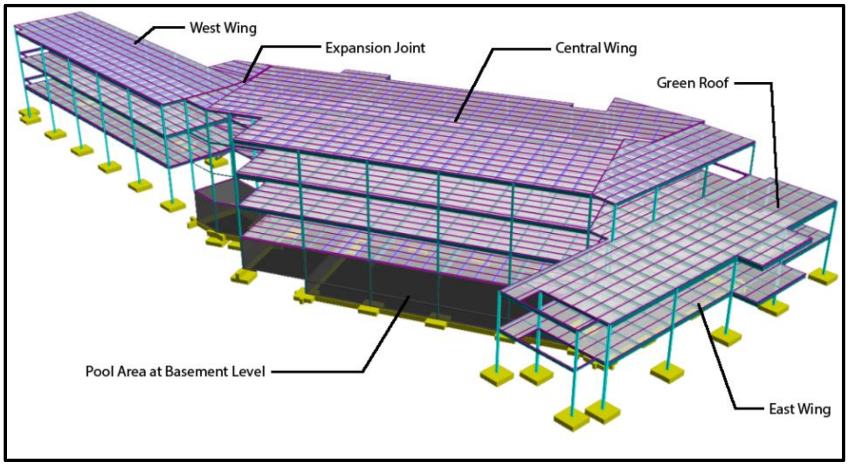


Figure 4: 3-D RAM Model Showing Gravity System

The floor framing system for our design utilizes a typical composite floor system with composite deck and steel beams. Composite framing is usually one of the most economical choices for floor systems and this system fits our design very well. The first, second, and third floors of the elementary school utilize composite framing through use of composite steel deck and steel wide-flange shapes. Due to long spans over the pool area in the basement (further discussed in the Specialized Areas section), composite steel joists were chosen to support this portion of the first floor not sitting on grade.

In order to achieve a two-hour fire rating between floors, it was determined that 3 ¼" lightweight concrete topping on 2" metal deck be used for all floor levels and the green roof at the east wing. The specific deck type chosen for these areas is Vulcraft 2VLI20 (or approved equivalent) with the aforementioned 3 ¼" LW topping, corresponding to a total slab depth of 5 ¼" inches.(5) The roof above the central and west wings is framed using 1.5" non-composite roof deck (Vulcraft or approved equivalent) in conjunction with roof joists and steel wide-flange girders.(5)

Figure 5: Typical Floor Framing Layout - RAM Structural System Model

Framing for the floor levels and roof generally runs in the north-south direction with deck spanning east-west (the east wing being the exception). A typical floor framing layout courtesy of our RAM Structural System model can be seen in Figure 5.

Through use of RAM Structural System, our structural team obtained design shapes, stud quantities, and camber values for the gravity framing system in our building along with column design sizes based on the loads described in Appendix G. After running and optimizing the design (discussed further in the design rationale section), the framing layout for a typical floor included wide-flange shapes ranging from W8X10 to W24X76. A typical large classroom bay (41'-4" x 28'-0") from our framing plans can be seen in Figure 6. Our team also verified the RAM design results by performing "spot checks" on certain members throughout the structure. Some of these calculations can be viewed in Appendix H. Complete floor and roof framing plans are provided on drawings S-101, S-102, S-103, and S-104.

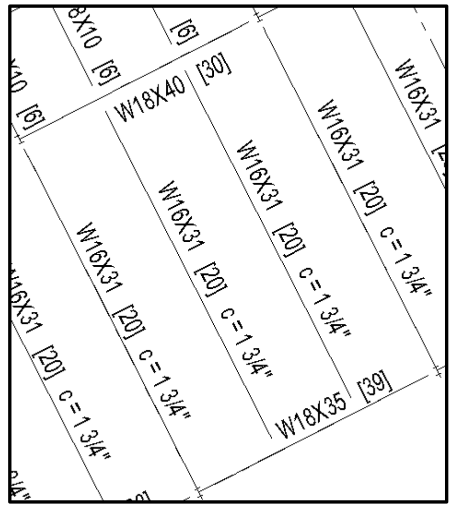


Figure 6: Typical Large Classroom Bay (Central Wing)

We were pleased with the overall results being that we wanted to keep structural depth to a minimum in particular areas where mechanical ducts and pipes would be running below the framing. As mentioned before, the design sizes obtained from RAM make sense and agree with hand calculation checks our structural design team has performed (See Appendix H). Generally, the largest composite shapes occur at the green roof area on the east wing (see drawing S-103) mainly due to high dead and live loads as well as a very high snow drift load based on a 14' difference in roof height at that location. Using a 31 psf flat roof snow load, drift loads at that location reached over 115 psf, something that obviously must be taken very seriously in the design. Snow load calculations are provided in Appendix G.

As mentioned earlier, framing for the roof above the central and west wing utilizes 1.5" roof deck (Vulcraft or approved equivalent) along with steel joists and wide-flange girders. It should be noted that typical foam insulation will also be utilized to mitigate noise penetration at the roof level. After utilizing the RAM model for design, roof joist sizes ranged from 10K1 to 30K9 (see drawing S-104). RAM also specified 32LH06 joists to frame the roof over the large classrooms in the central wing where spans exceeded 40 feet. However, for framing depth purposes, our team has decided to use 28K10 joists for these members. Calculations are provided in Appendix H.

Column Design

Column design was also executed through RAM Structural System. Typical column sizes ranged from W10X33 to W14x61. Based on the structural layout of our building, these sizes make sense and coincide with our desire to use column sizes of W10 and larger. When W8 shapes are used, it becomes difficult to connect framing members into the web of the column. The labor required to make the connection generally outweighs any savings by using the smaller column shape.

The column splicing level was taken to be just above the 2nd floor in our design. Using RAM's design output, column designs were generally controlled by the first story (1st floor to 2nd floor). Therefore, by splicing at the 2nd floor level, our team can save money by reducing steel shapes for stories above. It should be noted that the east wing of the elementary school is only two stories tall, and thus, columns in this portion of the building are not spliced. Columns were also not spliced if only one size was needed for all stories and the column ran three stories or less. Splices will be located 30" above the splice level's floor per standard design practice. A detailed column schedule for the project can be seen on drawing S-200.

Software Export Process

After RAM design was complete, our team exported our RAM structural model into Autodesk Revit 2013 to be used in our team's integrated building information model. Due to some mapping limitations, certain members did not transfer over to Revit, and thus had to be modeled manually. Not only did the Revit 2013 software assist in collaborating between other disciplines on our team, but it also aided in creating foundation and framing plans as well as schedules and details. Each framing member was given a framing tag in Revit complete with size, stud quantity, and camber as seen in Figure 6 and each framing plan. These tags were input manually after our team optimized the framing design (discussed in the upcoming Framing Optimization section). After modeling foundation walls, footings, and Geopier support, the result was a fully-functional structural building information model as seen in Figure 7.

Design Rationale

Utilizing the 3 ¼" lightweight concrete topping rather than the 4 ½" normal weight concrete for a two-hour fire-rating reduces dead load of the floor by approximately 40%. Keeping the structure as light as possible can provide benefits, especially in karst terrain. Not only does lighter floor construction reduce beam, girder, and column sizes, but it also greatly reduces seismic forces which are to be distributed to the foundation.

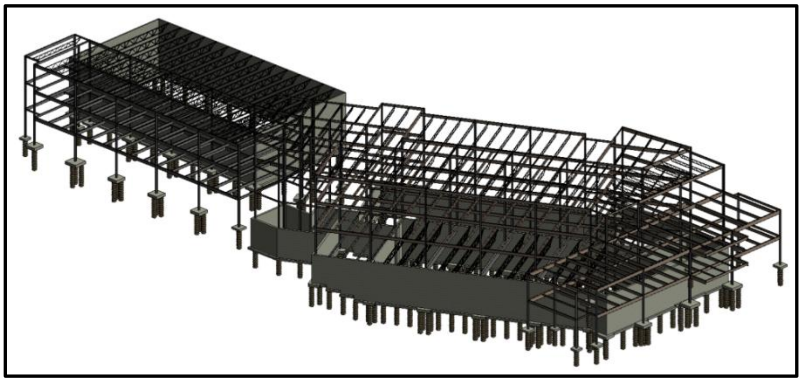


Figure 7: Revit Structural Model Showing Gravity Systems (No Deck)

Minimizing slab depth was also an area of high priority to better accommodate mechanical and electrical systems running below the structural framing, especially in tighter areas such as corridors and classrooms. This concept is discussed in greater detail in the Integrated Report.

Keeping constructability in mind, our design team opted to ensure shoring would not need to be used during construction. Thus, framing spacing on the second and third floors does not exceed the Steel Deck Institute maximum unshored clear span (3-span condition) of 10'-11" for the 2VLI20 deck type chosen.[5]

Roof joists in conjunction with select wide-flange girders were used to frame the roof of the elementary school in order to save on framing costs. Typically, utilizing joist framing is economical for a building such as an elementary school and is seen often in practice. The exception occurs at the roof of the east wing, which was framed using the same composite framing as the floor levels due to an increased green roof dead load and snow drift load.

In the early schematic design stages, our design team investigated some alternative options to the traditional steel composite beam framing system. One option was a concrete-framed elementary school. However, after running some quick calculations to determine preliminary slab depths, it was determined that slabs would be much too thick, especially in large classroom areas with larger bay sizes. For a 30' bay, we found that a flat plate slab would need to be approximately 12" thick, effectively tripling floor dead loads relative to the composite deck solution.[6] Even for a post-tensioned slab, required thickness for the same bay size proved to be approximately 9".[6] With either of these concrete solutions, the overall structure would become much heavier, which is something we wanted to avoid from the beginning given the karst terrain.

Framing Optimization

We chose steel composite beam framing over ordinary steel beams in non-composite action for the obvious benefit of reducing member sizes due to help from the concrete slab. However, our design team also wanted to emphasize constructability in our design so as to save time and money during construction. To accomplish this, we looked to achieve our second goal of an optimized gravity framing design per suggestions from Structure Magazine's April 2009 issue(7) and Modern Steel Construction's (MSC) November 2012 issue(8). One main topic presented in both articles is beam cambering and the general rules-of-thumb that are entailed. Structure Magazine suggests a minimum camber of 0.75" in conjunction with 0.25" increments. It is also recommended that no camber be specified for beams with less than a 24' span due to camber machine configurations.(7) These parameters were input directly into the design criteria for the RAM model.

In addition, MSC suggests avoiding the framing of cambered beams into cambered girders to prevent fit-up issues in the field during construction. The article also recommends eliminating camber in spandrel beams to reduce tolerance demands on the attached façade.(8) Because RAM does not have camber criteria for individual members, our team accounted for these recommendations through two separate design runs: one with camber and one with no camber. Subsequently, our team used the design with no camber for spandrel beams and various girders while utilizing the cambered design for the rest of the framing.

We have also optimized our design by (1) eliminating instances where deep beams frame into shallow girders to decrease labor costs in making a beam fit; (2) grouping beams to be the same size to speed up detailing and fabrication; and (3) simplifying construction by reducing the number of occurrences of non-orthogonal beam-to-column connections.(7,8)

These optimization techniques are evident in the provided framing plans on drawings S-101, S-102, S-103, and S-104. The successful outcome of these techniques is also evident in the estimated 4.5 lb/ft^2 average weight of steel framing for our building. Our team was very pleased with this value, especially given the high design loads in some locations.

Lateral Systems Design

In conjunction with our third structural goal of an innovative and efficient lateral force-resisting system, our team has chosen to use hybrid masonry walls to resist lateral loads in the elementary school.

Hybrid Masonry Walls

Hybrid walls, not to be confused with regular infill walls, utilize reinforced concrete masonry combined with a steel frame to share loads induced in the building. Thus, the masonry essentially serves as the bracing for the steel frame and also aids in taking gravity loads, as seen in Figure 8. This system can be especially economical when dealing with low-rise buildings where the architecture suits using steel framing and masonry walls, much like our three-story elementary school. Concrete masonry walls are likely to be used as partitions in certain areas of the school anyway, and this system

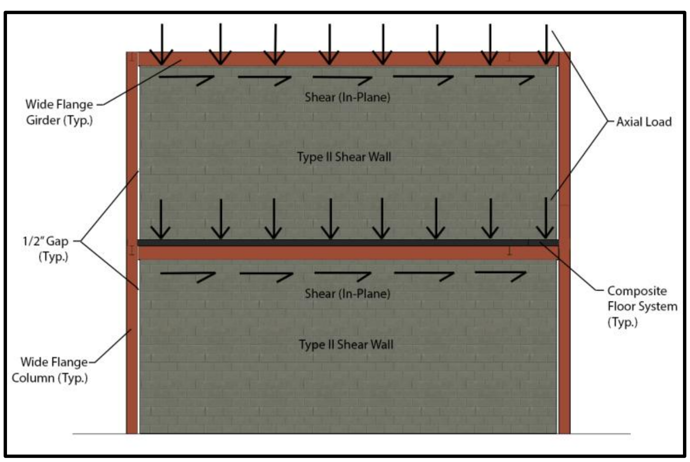


Figure 8: Type II Hybrid Wall Elevation

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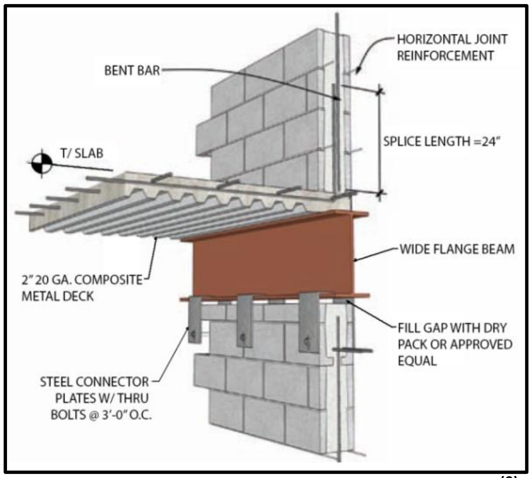


Figure 9: Isometric Section of Type II Hybrid Wall[9]

takes full advantage of these elements.

Three types of hybrid masonry walls are available (Type I, Type II, and Type III).[9] Our design team has chosen to use Type II hybrid walls for reasons explained in the upcoming Design Rationale section.

Basically, Type II hybrid walls consist of a loadbearing masonry shear wall within a steel frame. Connectors at the top of the wall transfer in-plane shear from the steel frame to the masonry wall. Because there is no gap between the top of the shear wall and the beam/girder (a gap that is present in Type I), a portion of the axial load is transferred to the wall. This helps provide redundancy to the structure and also reduces the amount of steel needed in the frame. Of important note is the fact that although moment connections may be used in the steel frame, they are not required in design of Type I or II. Thus, there are no extra costs for expensive column connections. A typical shear tab connection needs only a modest increase in strength in order to transfer shear forces induced by seismic overturning of the frame-wall assembly. A typical hybrid wall section as it applies to our school design can be seen in Figure 9. For seismic design, the Type II wall is typically assigned a response modification factor (R) of 4.0.[10] Seismic parameters and analysis can be seen in Appendix G.

As a side note, Type III walls are fully confined within the framing, with no gaps between the steel and masonry on any side. Although Type I and II walls can be designed using existing U.S. codes and standards, there are currently no standards in the U.S. which govern Type III design.[11] Therefore, our design team did not consider Type III.

We analyzed the hybrid masonry walls in RAM Structural System through use of "equivalent braces", a method personally recommended by David T. Biggs[12], a major contributor in the development of the hybrid masonry system. This method is further explained in the Design Rationale section.

When deciding on placement of the hybrid masonry wall frames in our structure, we wanted to place frames in the core of the building as well as along the perimeter to improve torsional resistance associated with lateral forces due to wind and seismic loads. An expansion joint divided the school into two sections as previously illustrated in Figure 4, thus separating the west wing from the central/east wing. The width of the expansion joint thickness was driven by the lateral drifts of each wing based on wind and seismic analysis (see Appendix G), and a width of 3 in. was adopted throughout the height of the building.

The location of hybrid masonry walls can be seen in Figure 10 as shown on a typical RAM model framing plan. It also must be noted that although CMU partitions may be used throughout the building, hybrid walls are only needed in these certain locations to satisfy lateral requirements, as illustrated in Appendix G. Please see Appendix D and drawing S-300 for hybrid masonry wall details.

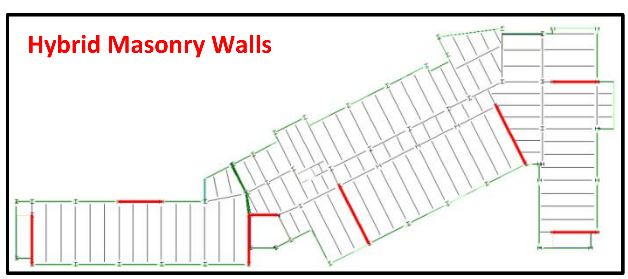


Figure 10: RAM Plan View Showing Hybrid Masonry Wall

Design Rationale

Our design team wanted to implement a unique and innovative design for our lateral system which also assists in taking gravity loads. However, we still wanted to maintain a budget. Many classrooms in schools are typically separated by masonry walls, so we decided to take advantage of that material while also using the benefits of the steel framing. The collaboration of the two materials is what brought us to hybrid masonry walls. As previously mentioned, our team consulted David T. Biggs for engineering logic and assumptions behind the system as well as modeling techniques.[12]

Hybrid masonry walls look similar to a typical steel frame with a masonry infill wall. However, the two materials are connected in such a way that they work together in taking both gravity loads and shear loads. While typical masonry infill walls are unreinforced, hybrid masonry walls are constructed with reinforced masonry to take the shear load and provide the stiffness for the lateral force-resisting system. The construction sequence allows the steel framing around the exterior to receive the dead load while the masonry takes the live load. This load sharing process takes advantage of each material's most valuable characteristics while also decreasing the amount of steel used. The end product is an extremely stiff load-bearing wall that could be used throughout the building.

While hybrid masonry walls are very efficient to resist lateral forces (as evidenced by the small lateral drifts seen in Appendix G), there are other reasons to employ them throughout our building. The first reason is the fact that it simplifies the construction. Some engineers try to detail lateral bracing up against a masonry infill wall which can create conflicts with the masonry and bracing. The second reason is that this system creates a lot of redundancy. Hybrid masonry limits progressive collapse by offering more than one load path. If the masonry is damaged, the gravity load transfers over to the steel frame, and if the steel frame is damaged, the gravity load is taken by the masonry. The system also makes sense from an efficiency standpoint. Being that CMU walls are usually used between classrooms and along stairwells (both prime locations for lateral frames in our building), it is logical to take advantage of the walls as lateral elements.

Modeling Methodology

Because the system is relatively new, there is no specific option for modeling an actual hybrid wall in current editions of structural analysis software. Instead, we modeled the system using the "equivalent brace" method.[12] Basically, one first solves for the standalone lateral stiffness (k) of the masonry wall as a function of modulus of elasticity (E_m) and shear modulus (G_m) of concrete masonry. Next, based on the stiffness equation for an inclined steel brace ($k = (AE/L)*\cos^2\theta$), the necessary area (A) of steel required to reach the wall stiffness can be solved for. The angle θ corresponds to the arctangent of the story height divided by the bay width containing the hybrid wall. Traditionally, this area is very large compared to a typical steel angle brace. The steel brace sizes used in modeling and analyzing typical hybrid masonry frames can be seen in Table 4 and supporting calculations are provided in Appendix D. Of note is that the steel beam-to-column connection needs to be designed for the shear forces induced by overturning due to wind or seismic lateral forces. Because of the small gap between the wall and the columns, moments induced on the beam and connections are small.

Table 4: Hybrid Wall Equivalent Brace Sizes

Bay Length (ft)	Brace Size
28'-0"	(2) W30x124
31'-4"	(2) W36x160
41'-4"	(2) W40x297

Specialized Areas

Pool

The structural design of our team's proposed elementary school includes several specialized areas which are not typical throughout the rest of the structure. One of these areas is the pool area which our team has located at the basement level under the central wing large classrooms as seen in Figure 11. Please refer to the Integrated Report for rationale behind this decision as well as detailed illustrations of the space. In order to accommodate the pool dimensions, the area below the south-facing classrooms and corridor needed to be spanned without intermediate supports. To accomplish this, we designed composite floor joists spaced at 7'-0" on center which bear on concrete walls to more evenly distribute the load. Per the Vulcraft Composite and Noncomposite Floor Joists design manual[13], these joists will be specified as 40CJ32 joists with (36) ¾" shear studs spaced evenly along the length.

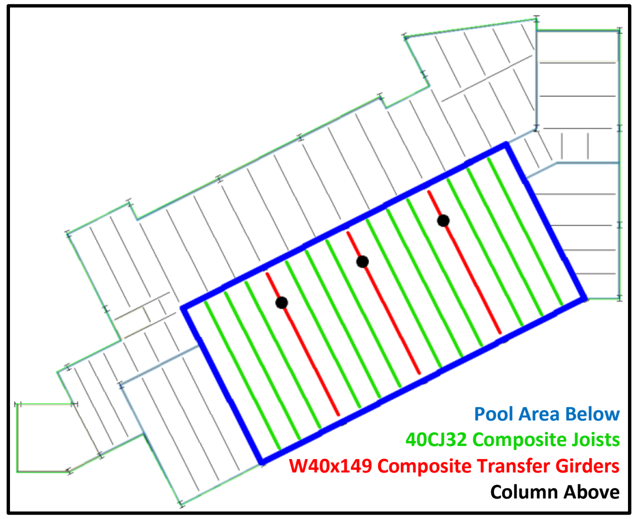


Figure 11: 1st Floor Framing Layout (Central Wing Above Pool)

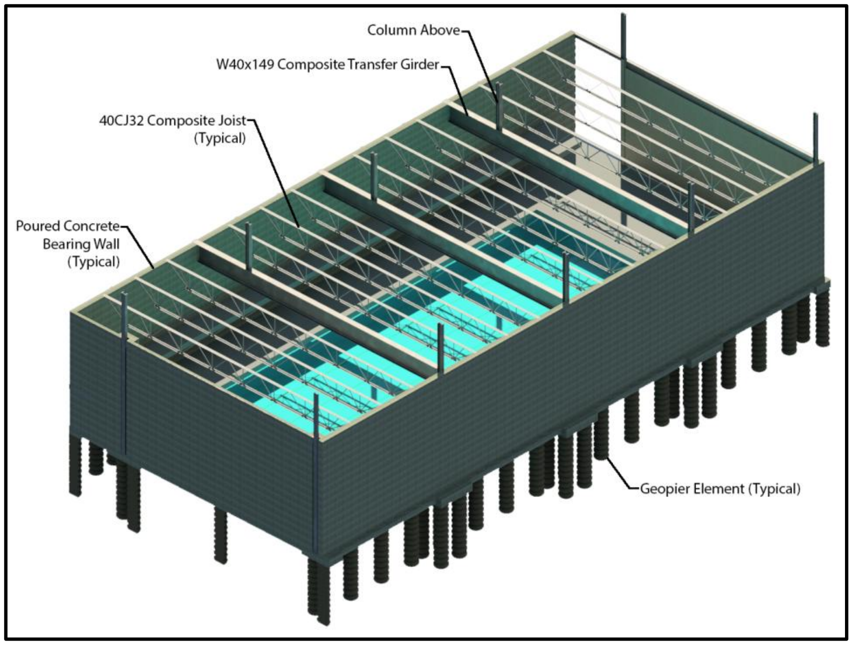


Figure 12: Revit 3-D Image of Structure Above Pool Area

However, in addition to the long span, there was another major structural challenge that needed to be solved. Column lines exist along both sides of the corridor in stories above, and the southern-most corridor column line runs through the plan area of the pool. To accommodate this issue, we designed steel transfer girders to take the column load of approximately 240 kips (dead + live) from above. In total, three composite W40x149 transfer girders spanning 53'-7" are located above the pool area spaced at 28'-0" on center. The aforementioned layout is shown in Figure 11, and a 3-D image of the structure above the pool can be seen in Figure 12. The W40x149 transfer girders coincide with the depth of the 40CJ32 joists, giving the floor framing in this area a uniform depth of 40 in.

Multipurpose Area/Shelter

Another specialized area in the elementary school is the multipurpose area which will double as a shelter facility for the community in coordination with the local Homeland Security department. When the provided competition program mentioned the use of the school as a shelter facility, our design team saw an enormous opportunity to enhance the overall value of the elementary school by optimizing a shelter facility design, leading to our fourth structural goal as stated earlier. It is our design team's expectation that this facility will be used during severe weather events such as hurricanes, tornadoes, and blizzards as well as other emergencies such as power outages. This facility may also be used as a safe haven for students and teachers in the event of a school shooting, such as that tragically seen at Sandy Hook Elementary. Because of these expectations, the multipurpose area has been designed using Risk Category IV, triggering the use of higher importance factors for seismic, wind, and snow loads.(3)

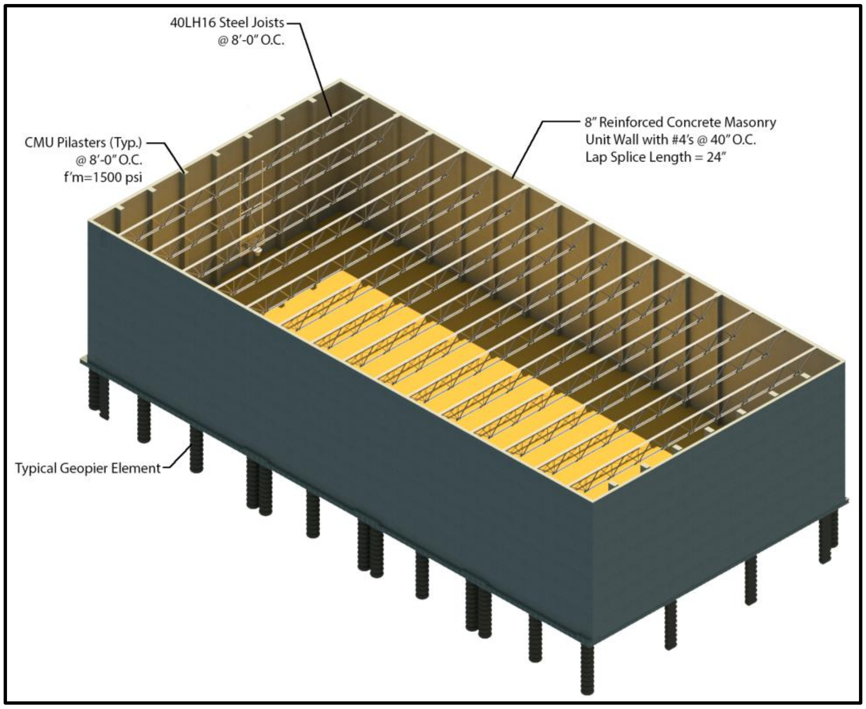


Figure 13: Revit 3-D Image Showing Structure of Multipurpose Area/Shelter

The walls will be constructed with 8" fully-grouted reinforced concrete masonry units (CMU's) with pilasters spaced at 8 feet on center to add additional out-of-plane stiffness, as seen in Figure 13. In accordance with design in the RAM Elements Masonry Module as well as the Masonry Society Standard 402-08, reinforcing provided will be #4 bars spaced at 40".(14) With the school located in an urban area, flying debris during a severe storm is also very likely, which is why each cell in the CMU wall will be fully grouted. See drawing S-300 for wall and pilaster details.

The roof system above the multipurpose area will be constructed using an engineered roof system specialized for severe weather conditions. Our structural design team recommends the Sika Sarnafil Engineered Roof System(15) or an approved equivalent mainly for uplift resistance and waterproofing. This system is specialized to function very well when subjected to high uplift forces during severe or extreme weather conditions. In fact, Sika Sarnafil offers a wind protection warranty for winds of up to 120 mph for 10-foot spans.(15)

Roof framing consists of typical roof joists (Vulcraft or approved equivalent) spanning 60' spaced at 8 feet. Per the Vulcraft Steel Joists & Joist Girders design guide(16), the joist size has been determined as 40LH16. This size was conservatively chosen based on a large snow drift load (see Appendix G) and a snow importance factor (I_s) of 1.20 based on Risk Category IV.(3) Joists are illustrated in Figure 13.

Reinforced concrete masonry bearing walls with pilasters made the most sense to our design team for the exterior of the shelter facility. Being that this facility will be used during severe weather events where high winds can be expected, fully-grouted reinforced masonry walls provide resistance to out-of-plane flexural loading, in-plane shear loading, and out-of-plane impact loading. The latter accounts for the scenario of debris impacting the exterior walls at high velocities. Without grout in each CMU cell, it is likely that debris such as a 2x6 wood stud from a nearby residential building would penetrate the wall if traveling at a high velocity in a severe storm event.

When talking about threat level certification per UFC 4-023-07 ("Design to Resist Direct Fire Weapons Effects"), a fully-grouted 8" CMU wall qualifies as having "High" threat level resistance in regards to ballistics resistant construction.(17) To put this in perspective, the "High" threat level certification is equivalent to withstanding a .30 caliber Armor Piercing projectile.(17)

Façade

The façade chosen for the elementary school was a result of an integrated team decision incorporating all disciplines. The SlenderWall precast panel system[18], seen in Figure 14(a), has benefits relating to each discipline, each of which are discussed in more detail in the Integration Report. It should also be noted that using panels from a local SlenderWall manufacturer is one way our structural team has achieved LEED points, as seen in Appendix F.

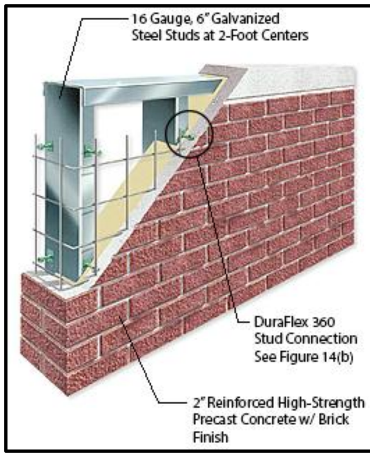


Figure 14(a): SlenderWall Precast Panel System[18]

Figure 14(b): DuraFlex 360° Stud Connection[18]

This precast façade panel system consists of lightweight panels weighing only 30 psf.[18] Due to its relatively light weight, the panel system reduces structural steel and foundation requirements in comparison to other façade choices. The system also utilizes differential movement technology through the use of DuraFlex 360° stud frame connections, as seen in Figure 14(b). The 16 gauge, 6" galvanized steel studs, a fraction of which are not embedded in the concrete, allow 360° movement to isolate the panels from structural stresses induced in the main lateral force resisting system through wind loading, seismic activity, and floor displacements.[18]

The system has options for transferring the gravity load to the main structure of the building. Our team has chosen to utilize steel edge angles at slab edges as seen in Appendix E. Typically, edge angles are used when concrete is poured on top of the deck, and thus, these angles can also be used to make the façade gravity connection. The studs will first transfer the concrete panel load to metal studs which subsequently transfer load to the edge angle. Details of this system are further discussed in Appendix E.

Conclusion

By designing and implementing a Geopier foundation system, optimized gravity framing, hybrid masonry walls, and a multihazard-resistant shelter, our structural design team accomplished the first four main goals we set out to achieve. To bring these systems together and achieve our fifth and final goal, we took to Revit and created a fully-functional structural building information model for use in the integrated project delivery. It is our firm belief that the aforementioned achievements, all of which come together in the building information model seen in Figure 15, strongly reflect the Pankow goals for structural innovations and help to solidify the Reading Elementary School as a "high performance building."

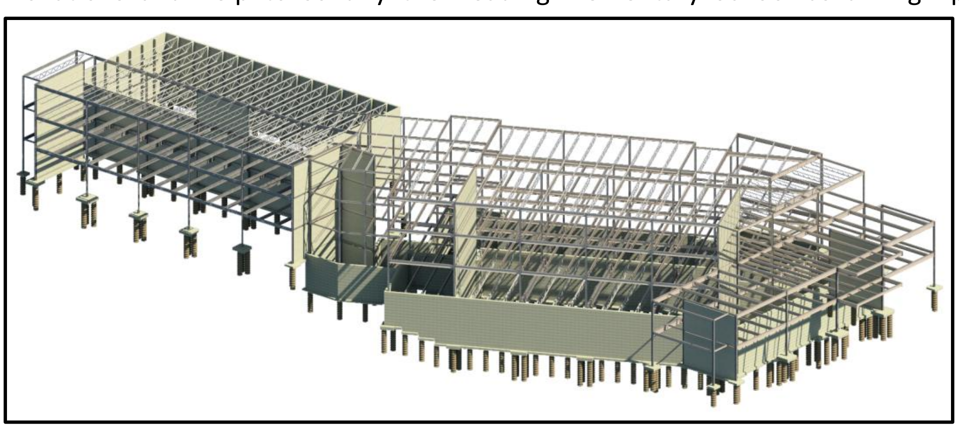


Figure 15: Rendered Building Information Model Complete with All Structural Systems

SECTION 2: SUPPORTING DOCUMENTATION

Appendix A: Lessons Learned

Our structural design team has learned a multitude of very important lessons throughout the design process. These lessons have not only helped us progress through our degree program, but we are also confident they will help us as we begin and progress through our careers. Some of the most important lessons we learned in the design process are listed below.

1. *A logical grid layout and naming convention is vital.*

   Laying out a logical grid system with an easily-understood naming convention up front can prove to be very beneficial throughout the entire design. When dealing with a non-orthogonal building, such as the proposed elementary school, a logical grid layout is important for inter-discipline communication as well as representation on drawings.

2. *File organization is crucial.*

   When progressing through the design, it is important to date and organize all files so that they may be easily accessed in the future if a design iteration is required. This applies to structural models, documents, spreadsheets, images, and presentations.

3. *RAM Structural System is an extremely powerful tool which must be used with discretion.*

   RAM Structural System is a very helpful tool and definitely makes the design process much easier. However, it is imperative that designers take extra caution when relying on RAM Structural System to design a building's structural system. Many engineers and professors often refer to the phrase "garbage in equals garbage out," meaning that wrong input will yield flawed output. Extra attention must be paid to modeling the structure in RAM as well as inputting the loads. It is very important to look at design warnings that may pop up during analysis and modify the model to ensure its accuracy. It is also very helpful and in our opinion, worthwhile, to perform manual spot checks on various members to ensure the software analysis and design is being performed as intended.

4. *BIM software is enormously helpful in graphical representation and integrated project delivery.*

   When collaborating with other disciplines who may not be familiar with structural systems or logistics, the saying, "a picture is worth a thousand words" holds true. Using software such as Autodesk Revit to model a system is very helpful in communicating the idea or concept to another team member, faculty member, etc. Not to mention, Revit is vital for integrated project delivery as it can be used for clash detection and other BIM-related uses.

5. *Communication with industry professionals is often the best resource.*

   Sending an e-mail or making a phone call to an established engineer can be very intimidating, but we have found that they are often very happy to answer a question or provide advice. Communicating with professionals not only enhances engineering knowledge, but it can also create significant personal and business connections which can help as we begin and further our careers as aspiring structural engineers.

Appendix B: Applicable Codes, Standards, and Software

Codes and Standards

- American Concrete Institute (ACI). "Building Code Requirements for Structural Concrete and Commentary." *ACI Standard 318-11.* (2011).
- American Institute of Steel Construction (AISC). *Steel Construction Manual.* 14th Edition. (2011).
- American Society of Civil Engineers (ASCE). "Minimum Design Loads for Buildings and Other Structures." *ASCE/SEI Standard 7-10.* (2010).
- International Code Council (ICC). *International Building Code*. International Code Council, Falls Church, VA. (2009).
- Masonry Standards Joint Committee (MSJC). "Building Code Requirements and Specification for Masonry Structures." *The Masonry Society (TMS) Standard 402-08.* (2008).
- Unified Facilities Criteria (UFC). "Design to Resist Direct Fire Weapons Effects." *UFC Standard 4-023-07.* United States of America Department of Defense. (2008).

BIM and Structural Analysis/Design Software

- "Autodesk Revit 2013." Autodesk. (2013).
- "RAM Structural System." Bentley Engineering (2012).
- "RAM Elements." Bentley Engineering. (2012).
- "CSC TEDDS." Computer Solution Central (CSC). (2012).

Appendix C: Geopier Foundation System Methodology/Calculations

Geopier Spread Footing Calculations

In design, several design assumptions were made based on the provided geotechnical report. These assumptions are as follows:

Geopier Element Diameter = 30"
Geopier Element Capacity (Q_{gp}) = 70 k
Element Stiffness Modulus (k_{rap}) = 175 pci
Existing Soil Bearing Capacity = 1001-2300 psf
Allowable Composite Footing Bearing Pressure (q_{max}) = 5,000 psf

Following the procedure set forth in the Geopier Soil Reinforcement Manual(1), our team created a spreadsheet with all necessary parameters for each isolated column footing and the wall footings in our building. The procedure is based on two types of settlement – upper zone and lower zone. The upper zone settlement deals with the area from grade to the bottom of the Geopier element cavity and is primarily a function of the element's stiffness modulus, the concentrated stress on the element, and the matrix soil modulus. The lower zone, on the other hand, entails the region of soils beneath the Geopier elements, or rammed aggregate piers (RAP's). This settlement is dependent on the size and shape of the footing, the footing stresses, and the compressibility and consolidation parameters of the lower zone soil. Plan dimension sizes and thicknesses were determined using RAM Structural System, and the spreadsheets used to design the Geopier foundation system can be seen on the following pages in Tables C-1 and C-2. Table C-3 denotes the minimum footing size requirements based on number of Geopier elements beneath each spread footing. As seen in Table C-1, all spread footings meet these requirements.

Table C-1: Upper Zone Settlement and Total Settlement

Floor	Footing Location	Column Load (kips)			Footing	Footing	Footing	RAP	Est'd	Actual	RAP	RAP	RAP	Soil	Stress	Area	Soil	RAP	Settlement		
		Dead DL	Live LL	Total TL	Width B	Length L	Stress q_o ksf	Capacity Q_{rap} kip	RAPs	RAPs	Dia d_{rap}	Depth Z_{rap}	Mod. K_{rap}	Mod. K_m pci	Ratio R_s	Ratio R_a	Stress q_m	Stress q_{rap}	UZ S_{UZ}	LZ S_{LZ}	Total S_{total}
					ft	ft	q_{max} 5				in	ft	pci	1001			ksf	ksf	in	in	in
Pool	B.5-13	102.5	73.4	176	6.0	6.0	4.9	70	2.5	3	30	11.0	175	6.95	25.2	0.41	0.45	11.3	0.45	0.06	0.51
	B.5-14	195.1	118.0	313	8.0	8.0	4.9	70	4.5	5	30	13.0	175	6.95	25.2	0.38	0.48	12.0	0.48	0.18	0.65
	B.5-15	195.2	118.0	313	8.0	8.0	4.9	70	4.5	5	30	13.0	175	6.95	25.2	0.38	0.48	12.0	0.48	0.18	0.65
	B.5-16	174.7	114.7	289	8.0	8.0	4.5	70	4.1	5	30	13.0	175	6.95	25.2	0.38	0.44	11.1	0.44	0.16	0.60
	B.5-17	81.0	62.7	144	6.0	6.0	4.0	70	2.1	3	30	11.0	175	6.95	25.2	0.41	0.37	9.2	0.37	0.05	0.41
	B.6-13	124.2	78.6	203	7.0	7.0	4.1	70	2.9	3	30	12.0	175	6.95	25.2	0.30	0.50	12.6	0.50	0.10	0.60
	B.6-18	115.3	76.3	192	7.0	7.0	3.9	70	2.7	3	30	12.0	175	6.95	25.2	0.30	0.47	11.9	0.47	0.09	0.57
	B.8-13	124.5	55.3	180	6.0	6.0	5.0	70	2.6	3	30	12.0	175	6.95	25.2	0.41	0.46	11.5	0.46		0.46
	B.8-14	154.7	71.2	226	7.0	7.0	4.6	70	3.2	4	30	12.0	175	6.95	25.2	0.40	0.43	10.9	0.43	0.11	0.54
	B.8-15	154.5	71.1	226	7.0	7.0	4.6	70	3.2	4	30	12.0	175	6.95	25.2	0.40	0.43	10.8	0.43	0.11	0.54
	B.8-16	154.4	71.1	226	7.0	7.0	4.6	70	3.2	4	30	12.0	175	6.95	25.2	0.40	0.43	10.8	0.43	0.11	0.54
	B.8-18	96.9	48.2	145	6.0	6.0	4.0	70	2.1	3	30	11.0	175	6.95	25.2	0.41	0.37	9.3	0.37	0.05	0.42
	C.2-21	86.6	65.2	152	6.0	6.0	4.2	70	2.2	3	30	11.0	175	6.95	25.2	0.41	0.39	9.7	0.39	0.05	0.44
	C.3-19	92.5	72.4	165	6.0	6.0	4.6	70	2.4	3	30	11.0	175	6.95	25.2	0.41	0.42	10.6	0.42	0.05	0.48
	C.3-21	134.9	116.4	251	8.0	8.0	3.9	70	3.6	4	30	13.0	175	6.95	25.2	0.31	0.47	11.7	0.47	0.14	0.61
	C.4-21	109.5	88.8	198	7.0	7.0	4.0	70	2.8	3	30	12.0	175	6.95	25.2	0.30	0.49	12.3	0.49	0.10	0.59
	C.5-21	61.2	45.1	106	6.0	6.0	3.0	70	1.5	2	30	11.0	175	6.95	25.2	0.27	0.39	9.8	0.39	0.04	0.42
Basement	A.2-9	34.0	35.4	69	4.0	4.0	4.3	70	1.0	1	30	9.0	175	6.95	25.2	0.31	0.52	13.0	0.51		0.51
	A.2-6.9/7	28.3	32.7	61	5.0	5.0	2.4	70	0.9	1	30	9.0	175	6.95	25.2	0.20	0.42	10.7	0.42	0.03	0.45
	A.3-7	10.9	2.6	13.5	4.0	4.0	0.8	70	0.2	1	30	9.0	175	6.95	25.2	0.31	0.10	2.5	0.10		0.10
	B.1-17	52.9	16.8	70	4.0	4.0	4.4	70	1.0	1	30	9.0	175	6.95	25.2	0.31	0.52	13.0	0.52		0.52
	B.2-12	55.7	35.1	91	5.0	5.0	3.6	70	1.3	2	30	10.0	175	6.95	25.2	0.39	0.35	8.7	0.35		0.35
	B.2-13	86.8	47.5	134	6.0	6.0	3.7	70	1.9	2	30	11.0	175	6.95	25.2	0.27	0.49	12.4	0.49	0.04	0.54
	B.3-14	101.5	43.1	145	6.0	6.0	4.0	70	2.1	3	30	11.0	175	6.95	25.2	0.41	0.37	9.3	0.37	0.05	0.42
	B.3-15	101.3	43.1	144	6.0	6.0	4.0	70	2.1	3	30	11.0	175	6.95	25.2	0.41	0.37	9.3	0.37	0.05	0.42
	B.3-16	70.5	31.9	102	5.0	5.0	4.1	70	1.5	2	30	10.0	175	6.95	25.2	0.39	0.39	9.8	0.39		0.39
	B.3-17	68.0	34.7	103	5.0	5.0	4.1	70	1.5	2	30	10.0	175	6.95	25.2	0.39	0.39	9.9	0.39		0.39
	B.4-12	69.7	62.9	133	6.0	6.0	3.7	70	1.9	2	30	11.0	175	6.95	25.2	0.27	0.49	12.2	0.48	0.04	0.53
	B.6-11	107.4	69.5	177	6.0	6.0	4.9	70	2.5	3	30	11.0	175	6.95	25.2	0.41	0.45	11.4	0.45	0.06	0.51
	B.7-8	17.4	3.8	21	4.0	4.0	1.3	70	0.3	1	30	9.0	175	6.95	25.2	0.31	0.16	4.0	0.16		0.16
	B.7-9	25.0	14.8	40	4.0	4.0	2.5	70	0.6	1	30	9.0	175	6.95	25.2	0.31	0.30	7.4	0.30		0.30
	B.8-11	84.6	31.3	116	6.0	6.0	3.2	70	1.7	2	30	11.0	175	6.95	25.2	0.27	0.42	10.7	0.42	0.04	0.46
	C.1-19	102.5	46.2	149	6.0	6.0	4.1	70	2.1	3	30	11.0	175	6.95	25.2	0.41	0.38	9.5	0.38	0.05	0.43
First Floor	A.1-1	28.4	9.8	38.2	4.0	4.0	2.4	70	0.5	1	30	9.0	175	6.95	25.2	0.31	0.28	7.1	0.28		0.28
	A.1-2	38.9	39.5	78.4	5.0	5.0	3.1	70	1.1	2	30	10.0	175	6.95	25.2	0.39	0.30	7.5	0.30		0.30
	A.1-3	94.1	72.2	166.3	6.0	6.0	4.6	70	2.4	3	30	11.0	175	6.95	25.2	0.41	0.42	10.7	0.42	0.06	0.48
	A.1-4	91.9	62.1	154	6.0	6.0	4.3	70	2.2	3	30	11.0	175	6.95	25.2	0.41	0.39	9.9	0.39	0.05	0.44
	A.1-5	84.6	59	143.6	6.0	6.0	4.0	70	2.1	3	30	11.0	175	6.95	25.2	0.41	0.37	9.2	0.37	0.05	0.41
	A.1-6	93	65.4	158.4	6.0	6.0	4.4	70	2.3	3	30	11.0	175	6.95	25.2	0.41	0.40	10.2	0.40	0.05	0.46
	A.2-2	18.8	21.5	40.3	4.0	4.0	2.5	70	0.6	1	30	9.0	175	6.95	25.2	0.31	0.30	7.5	0.30		0.30
	A.3-1	29.1	1.5	30.6	4.0	4.0	1.9	70	0.4	1	30	9.0	175	6.95	25.2	0.31	0.23	5.7	0.23		0.23
	A.5-2	64.2	48.4	112.6	6.0	6.0	3.1	70	1.6	2	30	11.0	175	6.95	25.2	0.27	0.41	10.4	0.41	0.04	0.45
	A.5-3	110.8	73	183.8	7.0	7.0	3.8	70	2.6	3	30	12.0	175	6.95	25.2	0.30	0.45	11.4	0.45	0.09	0.54
	A.5-4	108.9	52.4	161.3	6.0	6.0	4.5	70	2.3	3	30	11.0	175	6.95	25.2	0.41	0.41	10.4	0.41	0.05	0.46
	A.5-5	109	54.9	163.9	6.0	6.0	4.6	70	2.3	3	30	11.0	175	6.95	25.2	0.41	0.42	10.5	0.42	0.05	0.47
	A.5-6	112.4	56.7	169.1	6.0	6.0	4.7	70	2.4	3	30	11.0	175	6.95	25.2	0.41	0.43	10.9	0.43	0.06	0.49
	A.5-6.9	56.1	30.5	86.6	5.0	5.0	3.5	70	1.2	2	30	10.0	175	6.95	25.2	0.39	0.33	8.3	0.33		0.33
	B.3-7.9/8	42.9	23.2	66.1	5.0	5.0	2.6	70	0.9	1	30	10.0	175	6.95	25.2	0.20	0.46	11.6	0.46		0.46
	B.3-10	40.9	23.1	64	4.0	4.0	4.0	70	0.9	1	30	9.0	175	6.95	25.2	0.31	0.48	12.0	0.47		0.47
	B.4-7.9/8	73	61.7	134.7	6.0	6.0	3.7	70	1.9	2	30	11.0	175	6.95	25.2	0.27	0.49	12.4	0.49	0.04	0.54
	B.4-10	48.8	44.5	93.3	5.0	5.0	3.7	70	1.3	2	30	10.0	175	6.95	25.2	0.39	0.36	9.0	0.36		0.36
	C.1-22	17.4	3.7	21.1	4.0	4.0	1.3	70	0.3	1	30	9.0	175	6.95	25.2	0.31	0.16	3.9	0.16		0.16
	C.1-24	72.7	32.2	104.9	5.0	5.0	4.2	70	1.5	2	30	10.0	175	6.95	25.2	0.39	0.40	10.1	0.40		0.40
	C.3-22	23.2	21.5	44.7	4.0	4.0	2.8	70	0.6	1	30	9.0	175	6.95	25.2	0.31	0.33	8.4	0.33		0.33
	C.3-24	143.7	67.2	210.9	7.0	7.0	4.3	70	3.0	4	30	12.0	175	6.95	25.2	0.40	0.40	10.1	0.40	0.10	0.51
	C.4-24	143.1	65.4	208.5	7.0	7.0	4.3	70	3.0	3	30	12.0	175	6.95	25.2	0.30	0.51	13.0	0.51	0.10	0.62
	C.6-20	94.2	36	130.2	6.0	6.0	3.6	70	1.9	2	30	11.0	175	6.95	25.2	0.27	0.48	12.0	0.48	0.04	0.52
	C.6-24	121.9	43.9	165.8	6.0	6.0	4.6	70	2.4	3	30	11.0	175	6.95	25.2	0.41	0.42	10.6	0.42	0.06	0.48
	C.7-22	13.1	7	20.1	4.0	4.0	1.3	70	0.3	1	30	9.0	175	6.95	25.2	0.31	0.15	3.8	0.15		0.15
	C.7-24	19.5	0.2	19.7	5.0	5.0	0.8	70	0.3	1	30	10.0	175	6.95	25.2	0.20	0.14	3.5	0.14		0.14
	C.8-20	75	24.1	99.1	5.0	5.0	4.0	70	1.4	2	30	10.0	175	6.95	25.2	0.39	0.38	9.5	0.38		0.38
	C.8-23	19.5	0.2	19.7	4.0	4.0	1.2	70	0.3	1	30	9.0	175	6.95	25.2	0.31	0.15	3.7	0.15		0.15

Table C-2: Lower Zone Settlement

Floor	Footing Location	SOG FF Elev	Const Grade Elev	Ftg. Width	Ftg. Thick.	Ftg. Reference Depth	BOF Elev	Bot RAP Shaft Elev	RAP Drill Depth	Equiv. Ftg. Width B'	Max Analysis Depth	Max. Analysis Depth Elev	Ftg. Stress q_o	RAP Length H_s	Layer 1						
															Bot1	Bot1	z1	f(B)1	q/qo1	Es1	s1
		ft	ft	ft	ft	ft	ft	ft	ft	ft	ft	ft	ksf	ft	ft	elev	ft	B	%	tsf	in
Pool	B.5-13	345.0	345.0	6.0	1.50	-2.0	341.5	330.5	14.67	6.0	12.0	329.5	4.89	11.0	12.0	329.5	1	1.9	0.08	40	0.06
	B.5-14	345.0	345.0	8.0	2.00	-2.0	341.0	328.0	17.33	8.0	16.0	325.0	4.89	13.0	16.0	325.0	3	1.8	0.08	40	0.18
	B.5-15	345.0	345.0	8.0	2.00	-2.0	341.0	328.0	17.33	8.0	16.0	325.0	4.89	13.0	16.0	325.0	3	1.8	0.08	40	0.18
	B.5-16	345.0	345.0	8.0	2.00	-2.0	341.0	328.0	17.33	8.0	16.0	325.0	4.52	13.0	16.0	325.0	3	1.8	0.08	40	0.16
	B.5-17	345.0	345.0	6.0	1.50	-2.0	341.5	330.5	14.67	6.0	12.0	329.5	3.99	11.0	12.0	329.5	1	1.9	0.08	40	0.05
	B.6-13	345.0	345.0	7.0	1.50	-2.0	341.5	329.5	16.00	7.0	14.0	327.5	4.14	12.0	14.0	327.5	2	1.9	0.08	40	0.10
	B.6-18	345.0	345.0	7.0	1.50	-2.0	341.5	329.5	16.00	7.0	14.0	327.5	3.91	12.0	14.0	327.5	2	1.9	0.08	40	0.09
	B.8-13	345.0	345.0	6.0	1.50	-2.0	341.5	329.5	16.00	6.0	12.0	329.5	4.99	12.0	12.0	329.5					
	B.8-14	345.0	345.0	7.0	1.50	-2.0	341.5	329.5	16.00	7.0	14.0	327.5	4.61	12.0	14.0	327.5	2	1.9	0.08	40	0.11
	B.8-15	345.0	345.0	7.0	1.50	-2.0	341.5	329.5	16.00	7.0	14.0	327.5	4.60	12.0	14.0	327.5	2	1.9	0.08	40	0.11
	B.8-16	345.0	345.0	7.0	1.50	-2.0	341.5	329.5	16.00	7.0	14.0	327.5	4.60	12.0	14.0	327.5	2	1.9	0.08	40	0.11
	B.8-18	345.0	345.0	6.0	1.50	-2.0	341.5	330.5	14.67	6.0	12.0	329.5	4.03	11.0	12.0	329.5	1	1.9	0.08	40	0.05
	C.2-21	345.0	345.0	6.0	1.50	-2.0	341.5	330.5	14.67	6.0	12.0	329.5	4.22	11.0	12.0	329.5	1	1.9	0.08	40	0.05
	C.3-19	345.0	345.0	6.0	1.50	-2.0	341.5	330.5	14.67	6.0	12.0	329.5	4.58	11.0	12.0	329.5	1	1.9	0.08	40	0.05
	C.3-21	345.0	345.0	8.0	2.00	-2.0	341.0	328.0	17.33	8.0	16.0	325.0	3.93	13.0	16.0	325.0	3	1.8	0.08	40	0.14
	C.4-21	345.0	345.0	7.0	1.50	-2.0	341.5	329.5	16.00	7.0	14.0	327.5	4.05	12.0	14.0	327.5	2	1.9	0.08	40	0.10
	C.5-21	345.0	345.0	6.0	1.50	-2.0	341.5	330.5	14.67	6.0	12.0	329.5	2.95	11.0	12.0	329.5	1	1.9	0.08	40	0.04
Basement	A.2-9	351.0	351.0	4.0	1.00	-2.0	348.0	339.0	12.00	4.0	8.0	340.0	4.34	9.0	8.0	340.0					
	A.2-6.9/7	365.0	365.0	5.0	1.00	-2.0	362.0	353.0	12.00	5.0	10.0	352.0	2.44	9.0	10.0	352.0	1	1.9	0.08	40	0.03
	A.3-7	365.0	365.0	4.0	1.00	-2.0	362.0	353.0	12.00	4.0	8.0	354.0	0.84	9.0	8.0	354.0					
	B.1-17	351.0	351.0	4.0	1.00	-2.0	348.0	339.0	12.00	4.0	8.0	340.0	4.36	9.0	8.0	340.0					
	B.2-12	351.0	351.0	5.0	1.00	-2.0	348.0	338.0	13.33	5.0	10.0	338.0	3.63	10.0	10.0	338.0					
	B.2-13	351.0	351.0	6.0	1.50	-2.0	347.5	336.5	14.67	6.0	12.0	335.5	3.73	11.0	12.0	335.5	1	1.9	0.08	40	0.04
	B.3-14	351.0	351.0	6.0	1.50	-2.0	347.5	336.5	14.67	6.0	12.0	335.5	4.02	11.0	12.0	335.5	1	1.9	0.08	40	0.05
	B.3-15	351.0	351.0	6.0	1.50	-2.0	347.5	336.5	14.67	6.0	12.0	335.5	4.01	11.0	12.0	335.5	1	1.9	0.08	40	0.05
	B.3-16	351.0	351.0	5.0	1.00	-2.0	348.0	338.0	13.33	5.0	10.0	338.0	4.10	10.0	10.0	338.0					
	B.3-17	351.0	351.0	5.0	1.00	-2.0	348.0	338.0	13.33	5.0	10.0	338.0	4.11	10.0	10.0	338.0					
	B.4-12	351.0	351.0	6.0	1.50	-2.0	347.5	336.5	14.67	6.0	12.0	335.5	3.68	11.0	12.0	335.5	1	1.9	0.08	40	0.04
	B.6-11	351.0	351.0	6.0	1.50	-2.0	347.5	336.5	14.67	6.0	12.0	335.5	4.91	11.0	12.0	335.5	1	1.9	0.08	40	0.06
	B.7-8	351.0	351.0	4.0	1.00	-2.0	348.0	339.0	12.00	4.0	8.0	340.0	1.33	9.0	8.0	340.0					
	B.7-9	351.0	351.0	4.0	1.00	-2.0	348.0	339.0	12.00	4.0	8.0	340.0	2.49	9.0	8.0	340.0					
	B.8-11	351.0	351.0	6.0	1.50	-2.0	347.5	336.5	14.67	6.0	12.0	335.5	3.22	11.0	12.0	335.5	1	1.9	0.08	40	0.04
	C.1-19	351.0	351.0	6.0	1.50	-2.0	347.5	336.5	14.67	6.0	12.0	335.5	4.13	11.0	12.0	335.5	1	1.9	0.08	40	0.05
First Floor	A.1-1	365.0	365.0	4.0	1.00	-2.0	362.0	353.0	12.00	4.0	8.0	354.0	2.39	9.0	8.0	354.0					
	A.1-2	365.0	365.0	5.0	1.00	-2.0	362.0	352.0	13.33	5.0	10.0	352.0	3.14	10.0	10.0	352.0					
	A.1-3	365.0	365.0	6.0	1.50	-2.0	361.5	350.5	14.67	6.0	12.0	349.5	4.62	11.0	12.0	349.5	1	1.9	0.08	40	0.06
	A.1-4	365.0	365.0	6.0	1.50	-2.0	361.5	350.5	14.67	6.0	12.0	349.5	4.28	11.0	12.0	349.5	1	1.9	0.08	40	0.05
	A.1-5	365.0	365.0	6.0	1.50	-2.0	361.5	350.5	14.67	6.0	12.0	349.5	3.99	11.0	12.0	349.5	1	1.9	0.08	40	0.05
	A.1-6	365.0	365.0	6.0	1.50	-2.0	361.5	350.5	14.67	6.0	12.0	349.5	4.40	11.0	12.0	349.5	1	1.9	0.08	40	0.05
	A.2-2	365.0	365.0	4.0	1.00	-2.0	362.0	353.0	12.00	4.0	8.0	354.0	2.52	9.0	8.0	354.0					
	A.3-1	365.0	365.0	4.0	1.00	-2.0	362.0	353.0	12.00	4.0	8.0	354.0	1.91	9.0	8.0	354.0					
	A.5-2	365.0	365.0	6.0	1.50	-2.0	361.5	350.5	14.67	6.0	12.0	349.5	3.13	11.0	12.0	349.5	1	1.9	0.08	40	0.04
	A.5-3	365.0	365.0	7.0	1.50	-2.0	361.5	349.5	16.00	7.0	14.0	347.5	3.75	12.0	14.0	347.5	2	1.9	0.08	40	0.09
	A.5-4	365.0	365.0	6.0	1.50	-2.0	361.5	350.5	14.67	6.0	12.0	349.5	4.48	11.0	12.0	349.5	1	1.9	0.08	40	0.05
	A.5-5	365.0	365.0	6.0	1.50	-2.0	361.5	350.5	14.67	6.0	12.0	349.5	4.55	11.0	12.0	349.5	1	1.9	0.08	40	0.05
	A.5-6	365.0	365.0	6.0	1.50	-2.0	361.5	350.5	14.67	6.0	12.0	349.5	4.70	11.0	12.0	349.5	1	1.9	0.08	40	0.06
	A.5-6.9	365.0	365.0	5.0	1.00	-2.0	362.0	352.0	13.33	5.0	10.0	352.0	3.46	10.0	10.0	352.0					
	B.3-7.9/8	365.0	365.0	5.0	1.00	-2.0	362.0	352.0	13.33	5.0	10.0	352.0	2.64	10.0	10.0	352.0					
	B.3-10	365.0	365.0	4.0	1.00	-2.0	362.0	353.0	12.00	4.0	8.0	354.0	4.00	9.0	8.0	354.0					
	B.4-7.9/8	365.0	365.0	6.0	1.50	-2.0	361.5	350.5	14.67	6.0	12.0	349.5	3.74	11.0	12.0	349.5	1	1.9	0.08	40	0.04
	B.4-10	365.0	365.0	5.0	1.00	-2.0	362.0	352.0	13.33	5.0	10.0	352.0	3.73	10.0	10.0	352.0					
	C.1-22	365.0	365.0	4.0	1.00	-2.0	362.0	353.0	12.00	4.0	8.0	354.0	1.32	9.0	8.0	354.0					
	C.1-24	365.0	365.0	5.0	1.00	-2.0	362.0	352.0	13.33	5.0	10.0	352.0	4.20	10.0	10.0	352.0					
	C.3-22	365.0	365.0	4.0	1.00	-2.0	362.0	353.0	12.00	4.0	8.0	354.0	2.79	9.0	8.0	354.0					
	C.3-24	365.0	365.0	7.0	1.50	-2.0	361.5	349.5	16.00	7.0	14.0	347.5	4.30	12.0	14.0	347.5	2	1.9	0.08	40	0.10
	C.4-24	365.0	365.0	7.0	1.50	-2.0	361.5	349.5	16.00	7.0	14.0	347.5	4.26	12.0	14.0	347.5	2	1.9	0.08	40	0.10
	C.6-20	365.0	365.0	6.0	1.50	-2.0	361.5	350.5	14.67	6.0	12.0	349.5	3.62	11.0	12.0	349.5	1	1.9	0.08	40	0.04
	C.6-24	365.0	365.0	6.0	1.50	-2.0	361.5	350.5	14.67	6.0	12.0	349.5	4.61	11.0	12.0	349.5	1	1.9	0.08	40	0.06
	C.7-22	365.0	365.0	4.0	1.00	-2.0	362.0	353.0	12.00	4.0	8.0	354.0	1.26	9.0	8.0	354.0					
	C.7-24	365.0	365.0	5.0	1.00	-2.0	362.0	352.0	13.33	5.0	10.0	352.0	0.79	10.0	10.0	352.0					
	C.8-20	365.0	365.0	5.0	1.00	-2.0	362.0	352.0	13.33	5.0	10.0	352.0	3.96	10.0	10.0	352.0					
	C.8-23	365.0	365.0	4.0	1.00	-2.0	362.0	353.0	12.00	4.0	8.0	354.0	1.23	9.0	8.0	354.0					

Table C-3: Minimum Footing Size Requirements(1)

Number of Geopier Elements	Minimum Footing Size
1	3'-0" x 3'-0"
2	3'-0" x 6'-0"
3	6'-0" x 6'-0"
4	7'-0" x 7'-0"
5	7'-6" x 7'-6"

Essentially, the calculation process is based on the stiffness and load capacity of the rammed aggregate pier (RAP, or Geopier in this case) as well as the surrounding soil. The entire process, followed step-by-step through the Geopier Design Manual, hinges on the basic P=K*U principle. Upper zone settlements and lower zone settlements are added together and must not exceed the maximum allowable prescribed settlement of 1". As seen in Table C-1, our system has met this requirement. Also note that lower zone settlements need not be calculated where the shaft length (RAP Length) exceeds the Maximum Analysis Depth (twice the maximum footing dimension) per Geopier design requirements.(1)

The following calculations (in accordance with the Geopier Soil Reinforcement Manual)(1) pertain to the highest-loaded strip footing in the building along Grid B.5. Showing that settlement values are acceptable for this case prove that settlement values for the lesser-loaded strip footings will also pass.

Geopier Continuous Footing Calculations

Column Line B.5:

Load = 13.4 k/ft → We will use 15 k/ft to be conservative.

Allowable Composite Footing Bearing Pressure = **5000 psf**

Geopier Element and Footing Segment Capacity = **70k** (for a 30" Element)

Geopier Element Stiffness Modulus = **175 pci**

SPT = N, Blows Per Foot All Soils = **4-6**

UCS, (psf) Fine Grained Soils = **1001 psf**

Note: All values above are the most conservative for the appropriate range.

Use 30" diameter RAP's to keep base element capacity as 70k.

Continuous footing penalty: 0.9(70) = 63k

$\frac{63k}{\frac{15k}{ft}} = 4.2' \;\therefore$ We need a Geopier element every 4 feet along the bearing wall.

Matrix soil within a footing length = $3\left(\frac{30}{12}\right) = 7.5'$

$R_a = \frac{3.14}{(3x7.5)} = 0.139$

$q_{gp} = \frac{2660(25.18)}{[(0.139\,x\,25.18)-0.139+1]} = 15359\,psf$

$q = \frac{15(4)}{3(7.5)} = 2.66\,ksf$

$q_m = \frac{15359}{25.18} = 610\,psf$ (Bearing pressure on the matrix soil)

$K_m = \frac{1001}{144} = 6.95\,pci$

$R_s = \frac{175}{6.95} = 25.18$

Upper Zone Settlement: $S_{uz} = \frac{q_{GP}}{K_{GP}} = \frac{q_m}{K_m} = \frac{610}{6.95} = 87.8\,\frac{in^3}{ft^2} = \mathbf{0.61\,in}$

Influence Depth: $4B = 4(3') = 12'$

Westergaard Charts: 3.08B → 15.5% Footing Stress

Initial Element Length: $6 + \left(\frac{30}{12}\right) = 8.5'$

Static Stress Strain Modulus, E_s = 40 tsf = 80 ksf

Upper Zone Thickness = 8.5'

Stress on Lower Soil = $2.66(0.155) = 0.412$

Lower Zone Thickness = $12 - 8.5 = 3.5'$

Lower Zone Settlement = $\frac{0.412(3.5)}{80} = 0.018\,ft = \mathbf{0.22\,in}$

TOTAL SETTLEMENT = $\mathbf{0.61\,in + 0.22\,in = 0.83\,in}$

Appendix D: Hybrid Wall Calculations/Description

Equivalent Braced Frame Calculations

Note: Due to spatial limitations, only three hybrid wall calculations are shown.

Stiffness = K = $\frac{1}{\Delta}$

Deflection of Masonry Shear Wall = $\frac{V*h^3}{3*Em*I} + \frac{6*V*h}{5*Gm*A} = \frac{V}{E*t}[4(\frac{h}{L})^3 + 3\left(\frac{h}{L}\right)]$

For 28'-0" Hybrid Masonry Wall: $\Delta = \frac{1}{900*1500*7.625}\left[4\left(\frac{168}{336}\right)^3 + 3\left(\frac{168}{336}\right)\right] = 1.9428x10^{-7}$

Stiffness: $\frac{1}{1.9428x10^{-7}} = \mathbf{5146}\ \frac{\boldsymbol{k}}{\boldsymbol{in}}$

For 31'-4" Hybrid Masonry Wall: $\Delta = \frac{1}{900*1500*7.625}\left[4\left(\frac{168}{375.6}\right)^3 + 3\left(\frac{168}{375.6}\right)\right] = 1.6739x10^{-7}$

Stiffness: $\frac{1}{1.6739x10^{-7}} = \mathbf{6055}\ \frac{\boldsymbol{k}}{\boldsymbol{in}}$

For 41'-4" Hybrid Masonry Wall: $\Delta = \frac{1}{900*1500*7.625}\left[4\left(\frac{168}{496}\right)^3 + 3\left(\frac{168}{496}\right)\right] = 1.1381x10^{-7}$

Stiffness: $\frac{1}{1.13812x10^{-7}} = \mathbf{8786}\ \frac{\boldsymbol{k}}{\boldsymbol{in}}$

To calculate the equivalent area of steel for a braced frame, use the formula: $k = \frac{2AE}{L}cos^2\theta$

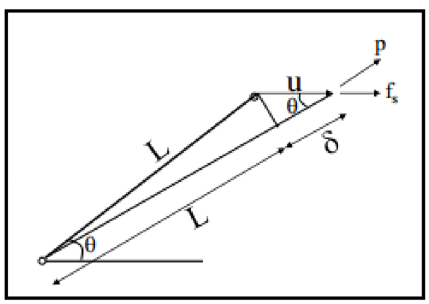


Figure D-1: Brace Geometry and Nomenclature

For 28'-0" Hybrid Masonry Wall: $5146 = 2\frac{A(29000)}{336}cos^2 26.6$ → A=37.3 in^2 → **Use W30x124 Braces**

For 31'-4" Hybrid Masonry Wall: $6055 = 2\frac{A(29000)}{376}cos^2 24.1$ → A=47.1 in^2 → **Use W36x160 Braces**

For 41'-4" Hybrid Masonry Wall: $8786 = 2\frac{A(29000)}{523}cos^2 18.72$ → A=88.3 in^2 → **Use W40x297 Braces**

Note: "Equivalent braces" can be seen in Figure D-2 as modeled in RAM for lateral analysis.

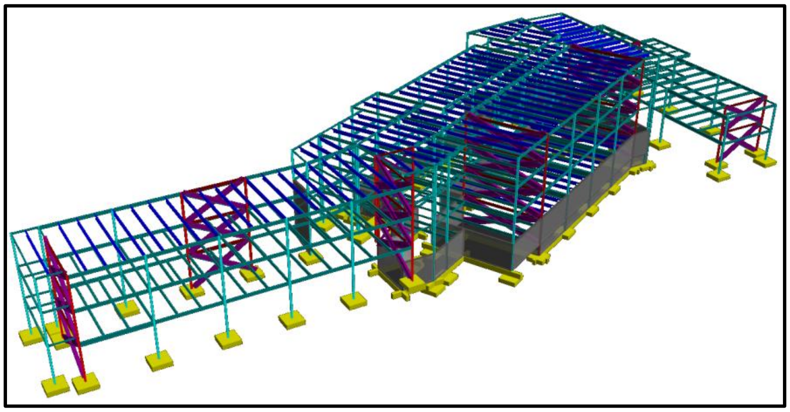


Figure D-2: 3-D RAM Model Showing Hybrid Wall "Equivalent Braces"

Hybrid masonry walls can be an economical structural choice for low-rise and medium-rise buildings, and they are very well suited to structures where the architectural design favors using both steel framing and reinforced masonry walls. The system applies well to schools, health care facilities, warehouses, retail, and offices.[19]

As mentioned earlier, our team implemented Type II hybrid walls in our design. The basic difference between Type I and Type II is the gap between the top of wall and bottom of girder. There is no gap in Type II construction allowing the loads to be shared between the steel framing and the wall, and the framing and the masonry walls are in the same plane. This hybrid type performs as a braced frame design with load-bearing shear walls. Compared to Type I, Type II offers further economic advantages to the steel framing by load-sharing both lateral loads and vertical loads. It offers efficient use of the masonry as well – as load bearing walls. For multi-story construction, Type II is preferred over Type I because the masonry weight and lateral load effects on the framing are transferred to the steel columns and the building foundation rather than requiring the steel girders to carry each floor independently.[19] Details of Type I and Type II connections are illustrated in Figures D-3 and D-4, respectively.

For Type II hybrid walls, there is an option to reduce the size of the girder framing members by load sharing with the reinforced masonry wall. If the masonry is constructed tight after the dead loads of the framing and floor/roof system are installed, the wall will attract the gravity loads that are added after the walls are built. Thus, the columns and girders could be sized to support only the dead loads.[19]

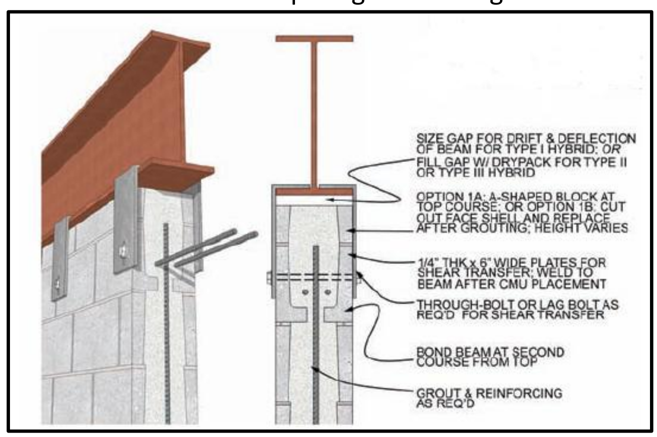


Figure D-3: Type I Connection[9]

Additional considerations are as follows: Not all masonry walls in a building must be designed as hybrid elements. The engineer has the option to use only specific bays as hybrid bracing. The remainder can be constructed as cavity wall construction. However, the more hybrid bays that are used, the overall masonry stresses in the structure are reduced. Also, since the hybrid masonry is the bracing for the building, erection bracing is required for the steel framing prior to completion of the masonry walls or they must be built in tandem with the steel (usually only possible on smaller structures).[19]

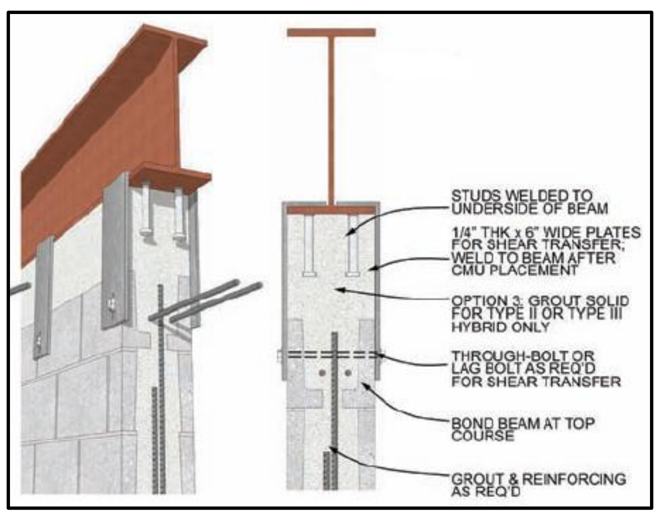


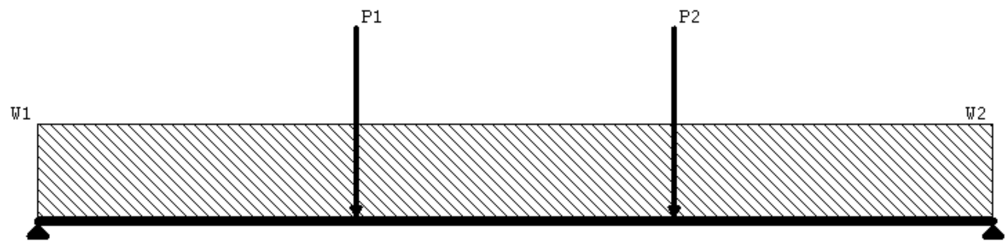
Figure D-4: Type II Connection[9]

Hybrid Wall Girder Size Reduction Calculations

A benefit of using hybrid walls throughout the building is the ability to greatly reduce the size of the girders that sit on top of the wall. As mentioned earlier in the report, the masonry in the hybrid walls carries the dead load while the steel framing carries the live load. After running RAM Structural System to design our gravity beams, we were able to view the loading diagrams that separate load types, as seen in Figure D-5. From here, we redesigned the hybrid wall girders for only their live loads. The resulting girders are significantly smaller, and since they sit on top of masonry, there is no deflection criteria which needs to be satisfied.

Floor Type: 2nd Floor **Beam Number = 17**

Span information (ft): I-End (69.92,40.00) J-End (96.92,40.00)



Load	Dist	DL	LL+	LL-	PL+	PL-	Max Tot
	ft	kips	kips	kips	kips	kips	kips
P1	9.000	10.769	9.862	0.000	1.800	0.000	22.432
P2	18.000	10.769	9.862	0.000	1.800	0.000	22.432
	ft	k/ft	k/ft	k/ft	k/ft	k/ft	k/ft
W1	0.000	0.035	0.000	0.000	0.000	0.000	0.035
W2	27.000	0.035	0.000	0.000	0.000	0.000	0.035

Figure D-5: RAM Structural System Girder Loading Diagram

Resulting hybrid wall girder calculation:

Using Table 3-22a from the AISC Steel Manual[20], we find that the maximum moment = $0.333 * P * L$

$$0.333(9.862)(27) = 88.67\ k * ft$$

$$Mu = 1.6 * 88.67 = 141.9\ k * ft$$

$\emptyset Mn \geq Mu \rightarrow$ **<u>Use a W14x26</u>** as opposed to a W21x44

Appendix E: SlenderWall Typical Details and Test Data

SlenderWall Typical Details

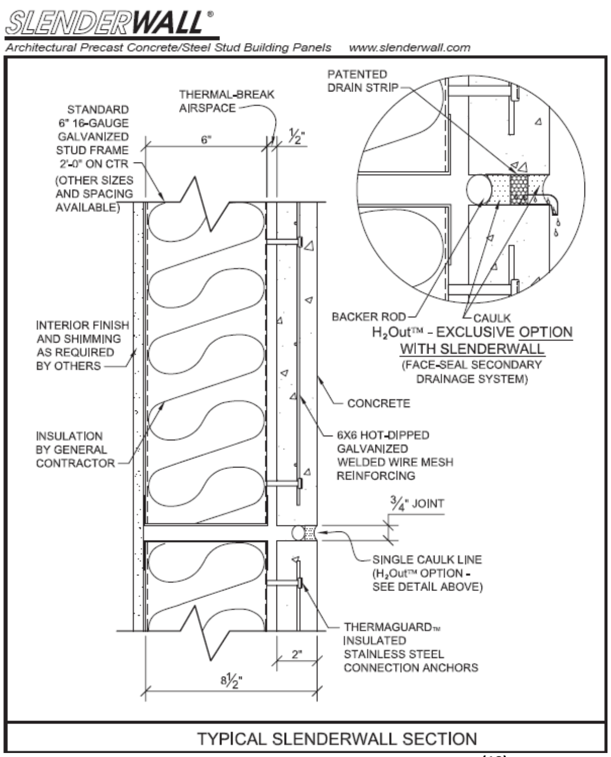


Figure E-1: Typical SlenderWall Section(18)

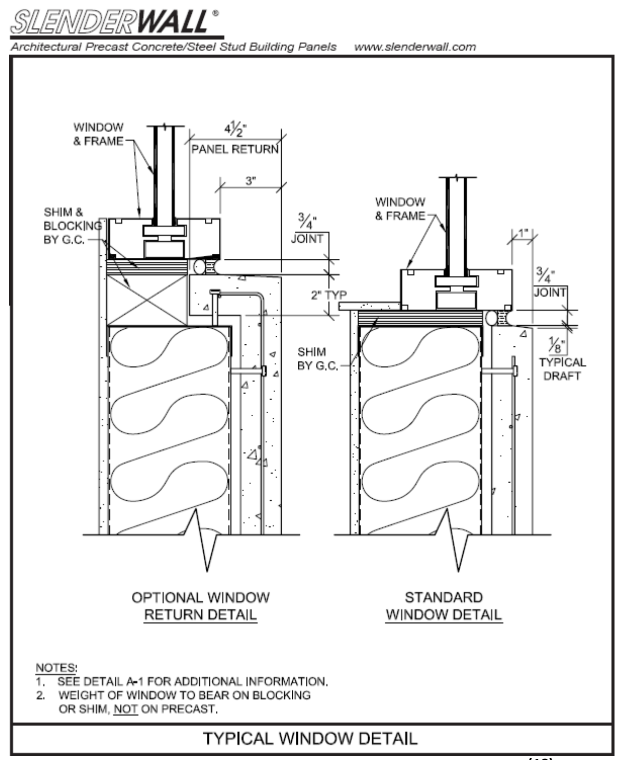


Figure E-2: Typical SlenderWall Window Detail(18)

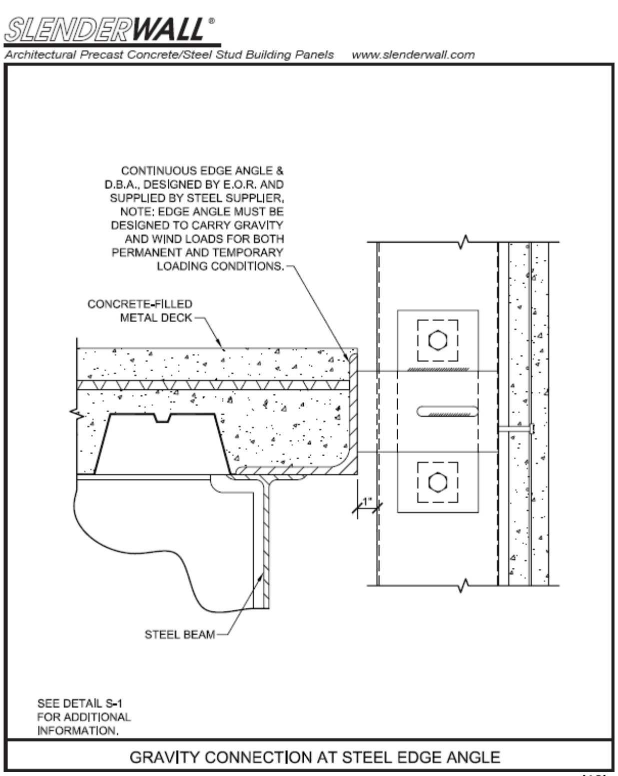


Figure E-3: Typical Gravity Connection at Steel Edge Angle(18)

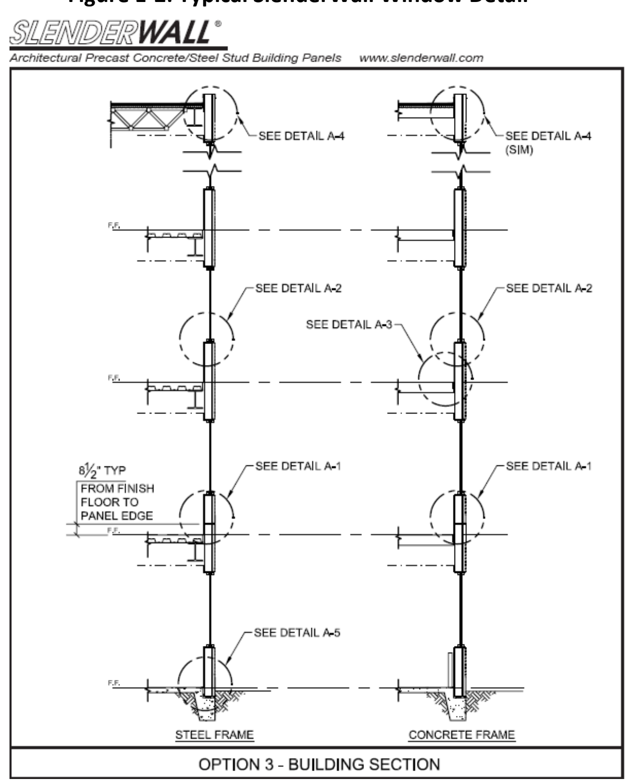


Figure E-4: Typical Panelization (Steel Frame on Left)(18)

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The SlenderWall panel system transfers the 2” reinforced precast concrete dead load to the building superstructure through a load transfer system which is illustrated in Figure E-5. A view from the interior side of the panel can be seen on the left of Figure E-5, while a view from the exterior side of the panel can be seen on the right.

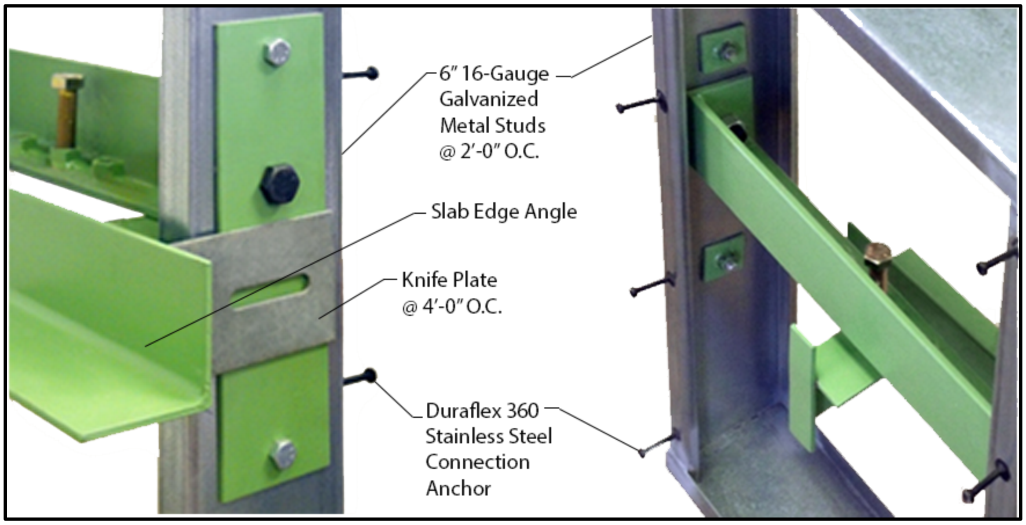


Figure E-5: SlenderWall Load Transfer System[18]

SlenderWall Panel Load Testing

The following details static load testing of a typical SlenderWall panel as performed by Smith-Midland Corp. in Midland, VA (a SlenderWall precast panel manufacturer).[18]

An 8’ x 16’ SlenderWall sample panel (such as those used on our elementary school) with 2” thick concrete, a ¾” architectural joint/chamfer along the 16’ concrete face, and WWM 6x6/2.0x2.0 reinforcing (galvanized WWM 6x6/2.9x2.9 is standard) was tested to determine deflections. Galvanized 16 gauge studs @ 24” O.C. with welded black steel nelson anchors @ 24" O.C. were embedded in the concrete and a 14 gauge top and bottom track was also provided. The test panel had been produced two years prior to testing and had been exposed to weather approximately 2 years in Smith Midland's yard. Details of the panel can be seen in Figure E-6 and test results are summarized in Table E-1.

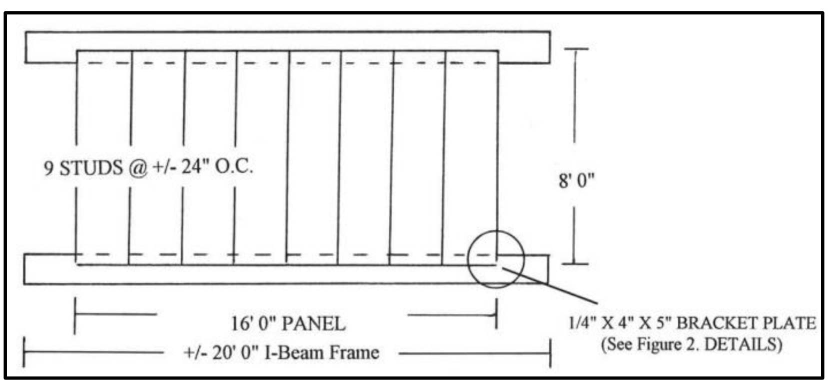


Figure E-6: SlenderWall Testing Panel[18]

Minor hairline cracks and first deformation of fixed ends of alternate studs were first noted at approximately a 130 psf total load. A deflection of L/360 (8.0’x12”/360 = 0.267”) was reached after an approximate 80 psf total load.

Inelastic failure of panel, or structural cracks in concrete, rotation of nelson anchors, and bending of steel studs occurred around 180 psf.

Concrete panel bolts and welds were still one unit as tested to ultimate failure at loadings approaching 300 psf, and gradual yielding of components was observed. The steel frame, though deformed, was still in one piece, and bolts and welds of nelson anchors and weld plates were unbroken. The concrete panel had severely broken, and the WWM was observed to be sheared in several places, but large quantities of concrete facing were still held to the steel frame by welded nelson anchors.

To summarize, the SlenderWall panel was successful in handling loads much greater than those which will be seen on the elementary school due to wind.

SLENDERWALL LOAD DEFLECTION TESTING

TOTAL LOAD DL+LL LB/SQ.FT.	STATIC LOADING LB/SQ.FT.	DEFLECTION GAUGE #1 BOLTED END	DEFLECTION GAUGE #2 CONCRETE	DEFLECTION GAUGE #3 NOT BOLTED
28	0	0.00"	0.00"	0.00"
79	51	0.15"	0.26"	0.19"
130	102	*A	*B >0.70"	0.55"

SLENDERWALL 8'X16' panel tested in horizontal position

*A - Stud movement-first signs of bolted/welded stud end deformation first hairline cracks appear in concrete

*B - L/360 deflection exceeded

Table E-1: Smith-Midland Corp. Load Test Data[18]

Appendix F: Structural LEED Considerations

Listed below are items pertaining to construction of the structural system where our team plans to achieve LEED points. These items were determined through collaboration with other team disciplines.

- Regional Materials (2 Points) – Use building materials located within 500 miles of the building site.
    - Steel fabricators
    - Concrete suppliers
    - Aggregate and CMU's from local suppliers
    - Formwork from local lumber yards
    - SlenderWall precast panel supplier (Smith-Midland Corp. in Midland, VA)

- Certified Wood (1 Point) – At least 50% of wood products used on site are Forest Stewardship Council (FSC) approved.
    - Formwork from lumber yards carrying FSC wood products

Appendix G: Building Design Loads, Parameters, and Analysis

Dead Loads

Level 1, 2, 3 Framing

2" Composite Metal Deck (Vulcraft or Approved Equivalent)	2 psf
3.25" Lightweight Concrete Fill	42 psf
Miscellaneous Concrete Overpour Weight	1 psf
MEP	10 psf
Floor/Ceiling	5 psf
	60 psf

Main Roof

1.5" Roof Deck (Vulcraft or Approved Equivalent)	2 psf
Membrane and Insulation	8 psf
MEP	10 psf
Ceiling	3 psf
	23 psf

Green Roof

2" Composite Metal Deck (Vulcraft or Approved Equivalent)	2 psf
3.25" Lightweight Concrete Fill	42 psf
Miscellaneous Concrete Overpour Weight	1 psf
Membrane and Insulation	8 psf
Green Roof System	25 psf
MEP	10 psf
Ceiling	5 psf
	93 psf

Facade

Second and Third Levels (14' floor-to-floor height)	0.42 k/ft
Roof Level (7' floor-to-floor height)	0.21 k/ft

Note: Façade line loads are based on a 30 psf façade weight.[18]

Live Loads

Lobbies/Vestibules	100 psf
Corridors at 1st Floor	100 psf
Corridors above 1st Floor	80 psf
Restrooms	100 psf
Library	60 psf
Offices	50 psf
Classrooms	40 psf
Roof	35 psf
Green Roof	100 psf

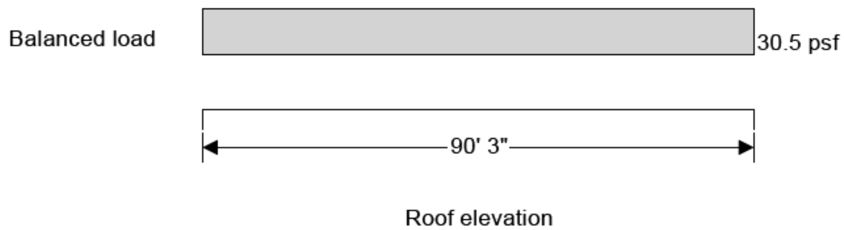
Note: A 10 psf partition load has also been included to accommodate future flexibility.

Snow Loads

SNOW LOADING (ASCE7-10)

TEDDS calculation version 1.0.02

Building details	
Roof type	Flat
Width of roof	b = **90.25** ft
Ground snow load	
Ground snow load	p_g = **40.00** lb/ft^2
Density of snow	$\gamma = \min(0.13 \times p_g / 1\text{ft} + 14\text{lb/ft}^3, 30\text{lb/ft}^3)$ = **19.20** lb/ft^3
Terrain type	B
Exposure condition (Table 7-2)	Fully exposed
Exposure factor (Table 7-2)	C_e = **0.90**
Thermal condition (Table 7-3)	All
Thermal factor (Table 7-3)	C_t = **1.10**
Importance category (Table 1-1)	III
Importance factor (Table 7-4)	I_s = **1.10**
Min snow load for low slope roofs (Sect 7.3.4)	$p_{f\_min} = I_s \times 20$ lb/ft^2 = **22.00** lb/ft^2
Flat roof snow load (Sect 7.3)	$p_f = \max(0.7 \times C_e \times C_t \times I_s \times p_g, p_{f\_min})$ = **30.49** lb/ft^2



Drift calculations	
Balanced snow load height	$h_b = p_f / \gamma$ = **1.59** ft
Length of upper roof	l_u = **140.00** ft
Length of lower roof	l_l = **36.67** ft
Height diff between uppper and lower roofs	h_{diff} = **14.00** ft
Height from balance load to top of upper roof	$h_c = h_{diff} - h_b$ = **12.41** ft
Drift height leeward drift	$h_{d\_l} = 0.43 \times (\max(20\text{ ft}, l_u) \times 1\text{ft}^2)^{1/3} \times (p_g / 1\text{lb/ft}^2 + 10)^{1/4} - 1.5\text{ft}$ = **4.44** ft
Drift height windward drift	$h_{d\_w} = 0.75 \times (0.43 \times (\max(25\text{ ft}, l_l) \times 1\text{ft}^2)^{1/3} \times (p_g / 1\text{lb/ft}^2 + 10)^{1/4} - 1.5\text{ft})$ = **1.72** ft
Maximum lw/ww drift height	$h_{d\_max} = \max(h_{d\_w}, h_{d\_l})$ = **4.44** ft
Drift height	$h_d = \min(h_{d\_max}, h_c)$ = **4.44** ft
Drift width	$W_d = \min(4 \times h_{d\_max}, 8 \times h_c)$ = **17.75** ft
Drift surcharge load	$p_d = h_d \times \gamma$ = **85.20** lb/ft^2

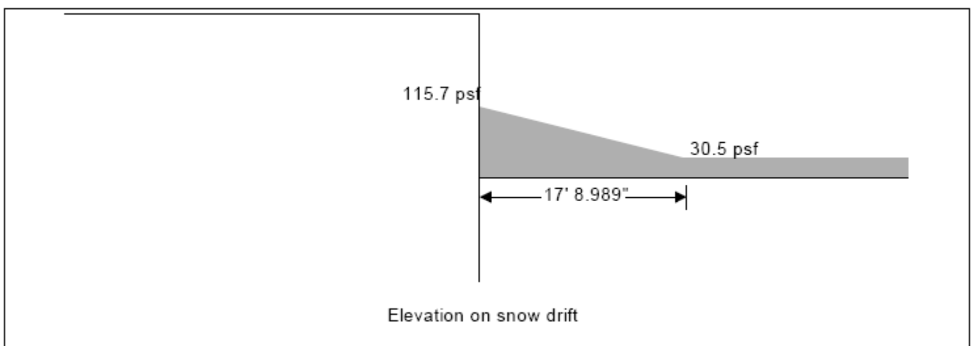
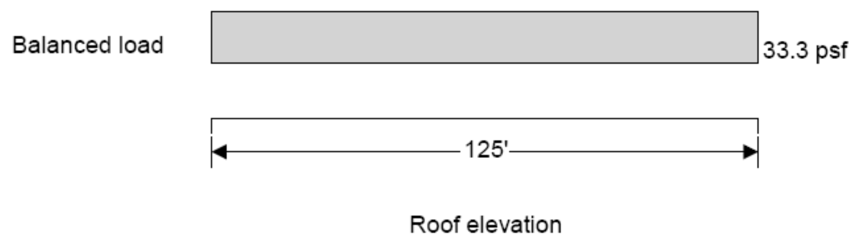


Figure G-1: Snow Drift on East Wing Green Roof – Per CSC TEDDS Software

SNOW LOADING (ASCE7-10)

TEDDS calculation version 1.0.02

Building details	
Roof type	Flat
Width of roof	b = **125.00** ft
Ground snow load	
Ground snow load	p_g = **40.00** lb/ft^2
Density of snow	$\gamma = \min(0.13 \times p_g / 1\text{ft} + 14\text{lb/ft}^3, 30\text{lb/ft}^3)$ = **19.20** lb/ft^3
Terrain type	B
Exposure condition (Table 7-2)	Fully exposed
Exposure factor (Table 7-2)	C_e = **0.90**
Thermal condition (Table 7-3)	All
Thermal factor (Table 7-3)	C_t = **1.10**
Importance category (Table 1-1)	IV
Importance factor (Table 7-4)	I_s = **1.20**
Min snow load for low slope roofs (Sect 7.3.4)	$p_{f\_min} = I_s \times 20$ lb/ft^2 = **24.00** lb/ft^2
Flat roof snow load (Sect 7.3)	$p_f = \max(0.7 \times C_e \times C_t \times I_s \times p_g, p_{f\_min})$ = **33.26** lb/ft^2



Drift calculations	
Balanced snow load height	$h_b = p_f / \gamma$ = **1.73** ft
Length of upper roof	l_u = **40.00** ft
Length of lower roof	l_l = **60.00** ft
Height diff between uppper and lower roofs	h_{diff} = **14.00** ft
Height from balance load to top of upper roof	$h_c = h_{diff} - h_b$ = **12.27** ft
Drift height leeward drift	$h_{d\_l} = 0.43 \times (\max(20\text{ ft}, l_u) \times 1\text{ft}^2)^{1/3} \times (p_g / 1\text{lb/ft}^2 + 10)^{1/4} - 1.5\text{ft}$ = **2.41** ft
Drift height windward drift	$h_{d\_w} = 0.75 \times (0.43 \times (\max(25\text{ ft}, l_l) \times 1\text{ft}^2)^{1/3} \times (p_g / 1\text{lb/ft}^2 + 10)^{1/4} - 1.5\text{ft})$ = **2.23** ft
Maximum lw/ww drift height	$h_{d\_max} = \max(h_{d\_w}, h_{d\_l})$ = **2.41** ft
Drift height	$h_d = \min(h_{d\_max}, h_c)$ = **2.41** ft
Drift width	$W_d = \min(4 \times h_{d\_max}, 8 \times h_c)$ = **9.64** ft
Drift surcharge load	$p_d = h_d \times \gamma$ = **46.28** lb/ft^2

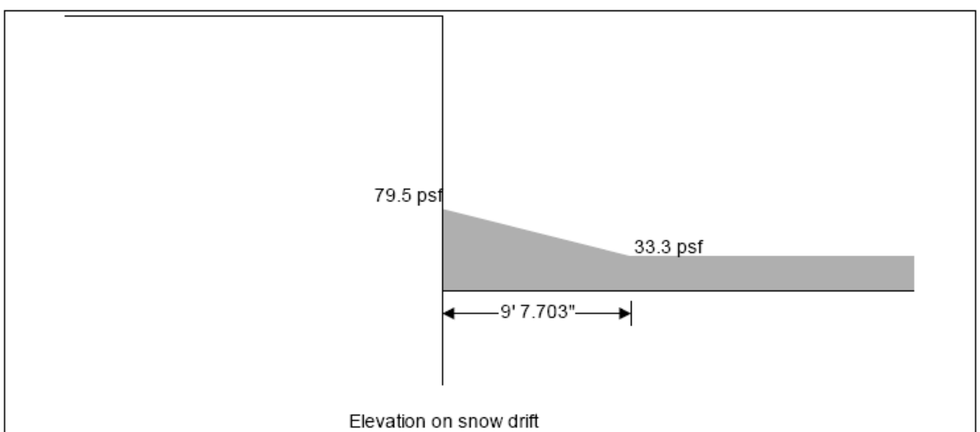


Figure G-2: Snow Drift on Multipurpose Area (Shelter) Roof – Per CSC TEDDS Software

Wind Parameters and Analysis

Table G-1: Elementary School Wind Load Calculations[3]

Basic Wind Speed, V	120 mph		
Wind Directionality Factor, K_d	0.85		
Exposure Category	B		
Velocity Pressure Exposure Coefficient, K_z	0.77		
Topographic Factor, K_{zt}	1.0		
Gust Effect Factor	0.85		
Enclosure Classification	Partially Enclosed		
Windward/Leeward/Side – Controlling Internal Pressure Coefficient, GC_{pi}	-0.55	0.55	0.55
Velocity Pressure, q_z	24.1 psf		
Windward/Leeward/Side – Wall Pressure Coefficients, C_p	0.8	-0.22	-0.7
Windward* Design Wind Pressure, p	29.7 psf		
Leeward Design Wind Pressure, p	-17.8 psf		
Side Wall Design Wind Pressure, p	-27.6 psf		

*Windward design wind pressure value at roof (highest value)

Table G-2: Shelter Wind Load Calculations[3]

Basic Wind Speed, V	135 mph		
Wind Directionality Factor, K_d	0.85		
Exposure Category	B		
Velocity Pressure Exposure Coefficient, K_z	0.68		
Topographic Factor, K_{zt}	1.0		
Gust Effect Factor	0.85		
Enclosure Classification	Partially Enclosed		
Windward/Leeward/Side – Controlling Internal Pressure Coefficient, GC_{pi}	-0.55	0.55	0.55
Velocity Pressure, q_z	27.0 psf		
Windward/Leeward/Side – Wall Pressure Coefficients, C_p	0.8	-0.3	-0.7
Windward* Design Wind Pressure, p	33.2 psf		
Leeward Design Wind Pressure, p	-21.7 psf		
Side Wall Design Wind Pressure, p	-30.9 psf		

*Windward design wind pressure value at roof (highest value)

Table G-3: Elementary School Design Wind Forces[3]

Wing	Direction	Wind Force-Resisting System	Design Force (k)	Drift (in)	Allowable Drift (in)
West	X	Intermediate Reinforced Hybrid Masonry Walls	36.0	0.07	1.26
	Y	Intermediate Reinforced Hybrid Masonry Walls	80.8	0.04	1.26
Central/ East	X	Intermediate Reinforced Hybrid Masonry Walls	44.4	0.10	1.26
	Y	Intermediate Reinforced Hybrid Masonry Walls	82.6	0.14	1.26

Seismic Parameters and Analysis

Table G-4: Seismic Parameters[3]

Seismic Site Class	C
Risk Category	III*
Seismic Importance Factor (I_e)	1.25*
Short-Period Spectral Response Acceleration (S_S)	0.195
One-Second Spectral Response Acceleration (S_1)	0.061
Short-Period Site Coefficient (F_a)	1.2
Long-Period Site Coefficient (F_v)	1.7
Adjusted Short-Period Spectral Response Acceleration (S_{MS})	0.234
Adjusted One-Second Spectral Response Acceleration (S_{M1})	0.104
Design Short-Period Spectral Response Acceleration (S_{DS})	0.156
Design One-Second Spectral Response Acceleration (S_{D1})	0.069
Seismic Design Category (SDC)	B

*Multipurpose Area/Shelter is Risk Category IV corresponding to a Seismic Importance Factor of 1.50.

Table G-5: Elementary School Effective Seismic Weight Calculation[3]

Wing	Level	Area (SF)	Façade Perimeter (ft)	Dead Load (psf)	Partitions (psf)	20% Flat Roof Snow Load (psf)	Façade Dead Load (plf)	Total Weight (k)
West	Main Roof	6150	395	23	0	6.6	210	265
	3	6150	270	60	10	0	420	544
	2	6150	270	60	10	0	420	544
							Effective Seismic Weight (k) =	1353
Central/ East	Main Roof	17720	427	23	0	6.6	210	614
	Green Roof	5400	381	93	0	6.6	210	618
	3	17720	427	60	10	0	420	1420
	2	23120	808	60	10	0	420	1958
							Effective Seismic Weight (k) =	4610

Table G-6: Elementary School Seismic Drift Analysis[3,10]

Wing	Direction	Seismic Force-Resisting System	Response Modification Coefficient (R)	Deflection Amplification Factor (C_d)	Seismic Importance Factor (I_e)	Seismic Response Coefficient (C_S)	Design Force (k)	Elastic Analysis Drift (δ_{xe}) (in)	Adjusted Maximum Drift (δ_x) (in)
West	X	Intermediate Reinforced Hybrid Masonry Walls	4.0	2.5	1.25	0.049	65.9	0.12	0.24
	Y	Intermediate Reinforced Hybrid Masonry Walls	4.0	2.5	1.25	0.049	65.9	0.04	0.08
Central/ East	X	Intermediate Reinforced Hybrid Masonry Walls	4.0	2.5	1.25	0.049	224.7	0.15	0.30
	Y	Intermediate Reinforced Hybrid Masonry Walls	4.0	2.5	1.25	0.049	224.7	0.16	0.32

Appendix H: Framing Design Spot Checks

Typical Composite Beam Design ($1^{st}/2^{nd}/3^{rd}$ Floor Small Classroom)

Span = 30'-8"
Spacing = 9'-4"
Slab/Deck = 44 psf
Misc. DL = 16 psf

$LL = 40psf + conservative\ 10\ psf\ partition\ load = 50\ psf$

$D = (44\ psf + 16psf) * (9.33') = 560\frac{lb}{ft}$

$L = (40\ psf + 10\ psf) * (9.33') = 467\frac{lb}{ft}$ (Live load reduction conservatively not applied)

$w_u = 1.2D + 1.6L = 1.2\left(560\frac{lb}{ft}\right) + 1.6\left(467\frac{lb}{ft}\right) = 1.42\frac{k}{ft}$

$$M_u = \frac{w_{u*l^2}}{8} = \frac{\left(1.42\frac{k}{ft}\right) * (30.67ft)^2}{8} = \mathbf{167\ k \cdot ft}$$

Using AISC Steel Construction Manual:(20)
In this case, it is likely that wet concrete deflection will control for a 30'-8" span:
∴ First, Satisfy Wet Concrete Deflection

$\frac{L}{360} = \frac{(30.67')(12")}{360} = 1.02"$ maximum allowable deflection

Wet concrete load =$(44\ psf) * (9.33') + 30\ lb/ft(conservative\ estimate\ of\ beam\ self\ wt.) = 441\ lb/ft$

$$\Delta_{wc} = \frac{5wL^4}{384EI} = \frac{5\left(0.441\frac{k}{ft}\right) * (30.67')^4 * (\frac{1728in^3}{ft^3})}{384 * (29000\ ksi) * (I_x)} = 1.02"$$

Solving for I_x → I_x = 297 in^4

Try a W16x26 (I_x = 301 in^4)

Composite Flexural Strength

Total Slab Thickness = 5.25"

Assume a = 1" → Y2 = 5.25" $-\frac{1"}{2}$ = 4.75" → Use Y2 = 5" (assumes a = 0.5")

Using AISC Steel Construction Manual: (20)

@Y2 = 5" for a W16x26 → ΣQ_n = 96.0 k and ϕM_n = 244 k-ft

$\emptyset Mn = 244\ k \cdot ft > Mu = 167\ k \cdot ft$ $\boxed{\text{OK}}$

(96.0 k) / (17.2 k/stud) = 5.6 → 6 studs (assuming 1 stud/rib)

6x2 = $\boxed{\text{12 studs/beam}}$

Verify $a_{actual} < a_{assumed}$

f'_c = 3 ksi and $b_{eff} = 92"\ (2 * \frac{Span}{8})$

$$a = \frac{\Sigma Q_n}{0.85*(f'_c)*(b_{eff})} = \frac{96\ k}{0.85*(3\ ksi)*(92")} = 0.41 < 0.5"\ \boxed{\text{OK}}$$

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Unshored Strength

$$w_u = 1.2\left[(44\,psf) * (9.33') + \left(26\frac{lb}{ft}\right)\right] + 1.6[(20\,psf\ Const.LL) * (9.33')] = 0.822\frac{k}{ft}$$

$$M_u = \frac{w_u L^2}{8} = \frac{\left(0.822\frac{k}{ft}\right)*(30.67')^2}{8} = 96.7\ k \cdot ft < \emptyset M_p = 166\ k \cdot ft\ \boxed{\textbf{OK}}$$

Live Load Deflection

$$w_{LL} = 0.467\frac{k}{ft}$$

$I_{LB} = 555\ in^4$ (slightly conservative to use Lower Bound Moment of Inertia)

$$\Delta_{LL} = \frac{5w_{LL}L^4}{384EI_{LB}} = \frac{5*\left(0.467\frac{k}{ft}\right)*(30.67')^4*\left(1728\frac{in^3}{ft^3}\right)}{384*(29000\ ksi)*(555\ in^4)} = 0.58 < \frac{L}{360} = 1.02\ \boxed{\textbf{OK}}$$

∴ **Use a W16x26 with 12 studs spaced evenly along length**

RAM design output = W16x26 with 12 studs **OK**

Typical Roof Joist (Small Classroom)

Span = 30'-8"
Spacing = 7'-0"
Roof DL = 23 psf
Roof LL = 35 psf → CONTROLS
Snow Load = 31 psf

$$D = (23\,psf) * (7' - 0") = 161\ lb/ft$$
$$L = (35\,psf) * (7' - 0") = 245\ lb/ft$$
$$w_u = 1.2D + 1.6L = 1.2(161\ lb/ft) + 1.6(245\ lb/ft) = 585\ lb/ft$$

Using Vulcraft Steel Joists & Joists Girders Design Guide:[16]
Try a 22K7 (9.7 lb/ft)
Total safe factored uniformly distributed load = 593 $lb/ft * >$ 585 lb/ft **OK**
Unfactored uniformly distributed live load causing a deflection of $L/240 = 415\ lb/ft ** > 245\ lb/ft$ **OK**

No other joist with a smaller weight meets the strength and service conditions.

∴ **Use a 22K7**

RAM design output = 22K7 **OK**

*Values interpolated at 30'-8"
**Values interpolated at 30'-8" and multiplied by 1.5 to obtain load causing a deflection of L/240

Typical Roof Joist (Large Classroom)

Span = 41’-3”
Spacing = 7’-0”
Roof DL = 23 psf
Roof LL = 35 psf → CONTROLS
Snow Load = 31 psf

D = (23 psf)*(7’-0”) = 161 lb/ft
L = (35 psf)*(7’-0”) = 245 lb/ft
w_u = 1.2D + 1.6L = 1.2(161 lb/ft) + 1.6(245 lb/ft) = 585 lb/ft

Using Vulcraft Joists & Joists Girders Design Guide(16)
<u>Try a 24K12</u> (16.0 lb/ft)
Total safe factored uniformly distributed load $= 635\ lb/ft * > 585\ lb/ft$ **OK**
Unfactored distributed live load causing a deflection of $L/240 = 347\ lb/ft ** > 245\ lb/ft$ **OK**

<u>Try a 28K10</u> (14.3 lb/ft)
Total safe factored uniformly distributed load $= 596\ lb/ft * > 585\ lb/ft$ **OK**
Unfactored distributed live load causing a deflection of $L/240 = 257\ lb/ft ** > 245\ lb/ft$ **OK**

<u>Try a 32LH06</u> (14.0 lb/ft) → Span falls in Safe Load Column (38’-46’) (see notes below)
Total safe factored uniformly distributed load $= 596\ lb/ft^a > 585\ lb/ft$ **OK**
Unfactored distributed live load causing a deflection of $L/240 = 295\ lb/ft^b > 245\ lb/ft$ **OK**

<u>Note:</u> RAM design output yielded a 32LH06 joist based on economy. However, recognizing that a 28K10 joist saves 4” of ceiling space (for ductwork, piping, etc.,) while only adding an additional 0.3 lb/ft of steel, our structural team has decided to use 28K10 joists for these members. Being that 18 of these particular joists are needed in the design, the total added weight of steel comes out to only 223 pounds, a tradeoff we are willing to make for the extra 4” of ceiling room.

∴ **Use a 28K10**

*Values interpolated at 41’-3”
**Values interpolated at 41’-3” and multiplied by 1.5 to obtain load causing a deflection of L/240

[a]The safe factored uniform load for the clear spans shown in the Safe Load Column is equal to (Safe Load) / (Clear Span + 0.67).(16)

$$\text{Safe factored uniform load} = \frac{25050\ lb}{41.33' + 0.67'} = 596\frac{lb}{ft}$$

[b]To solve for live loads for clear spans shown in the Safe Load Column (or lesser clear spans), multiply the live load of the shortest clear span shown in the Load Table by (the shortest clear span shown in the Load Table + 0.67 feet)^2 and divide by (the actual clear span + 0.67 feet)^2.(16)

$$\text{Safe unfactored live load} = \frac{\left(211\frac{lb}{ft}\right)*(49' + 0.67')^2}{(41.33' + 0.67')^2} = 295\ lb/ft$$

Appendix I: References

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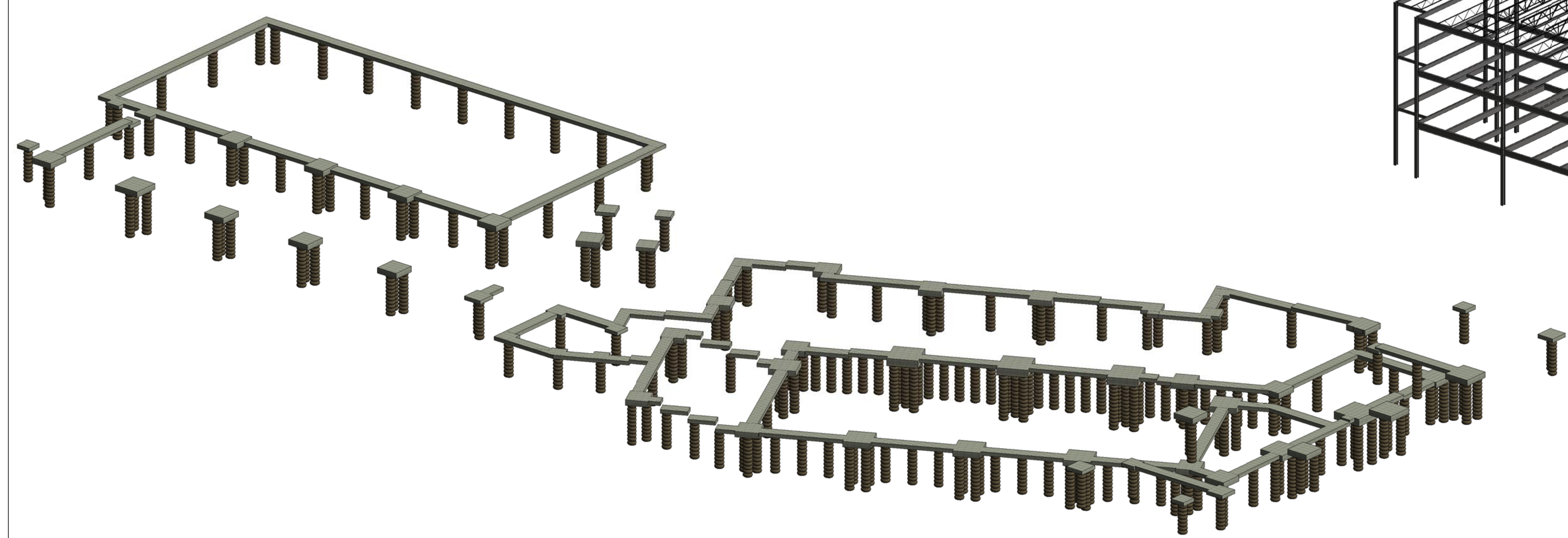
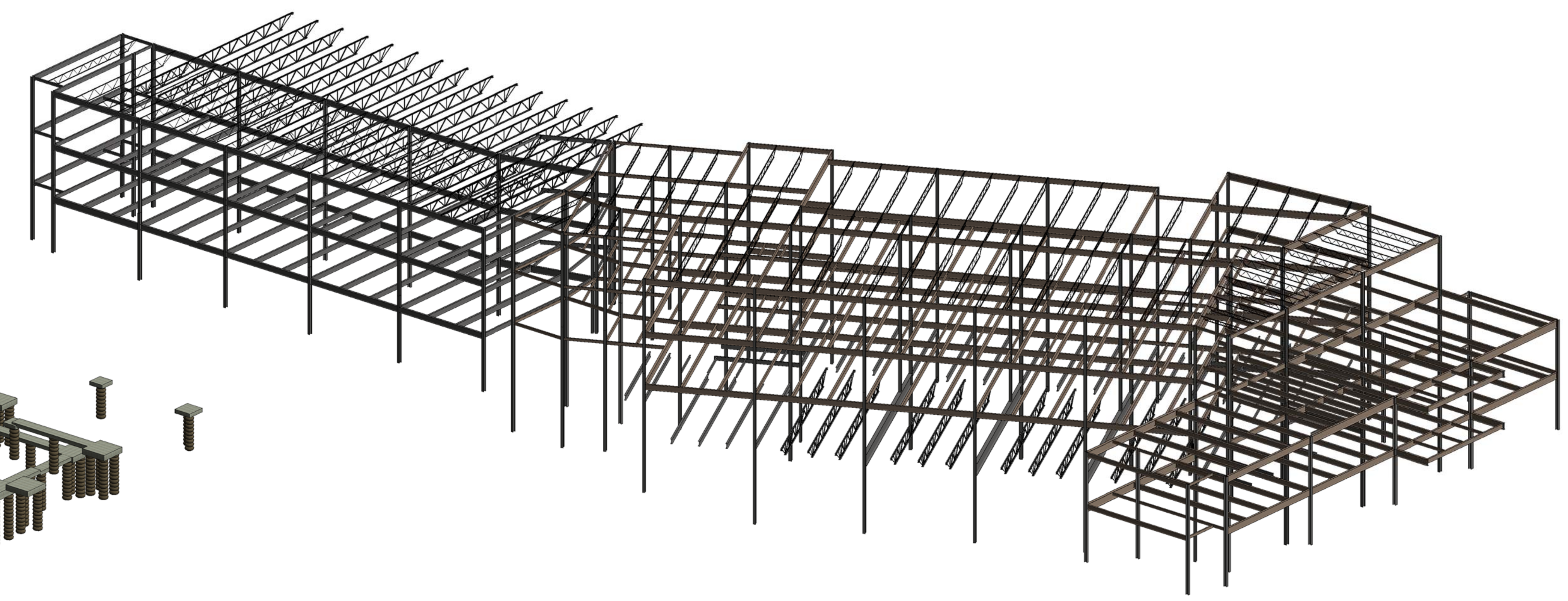
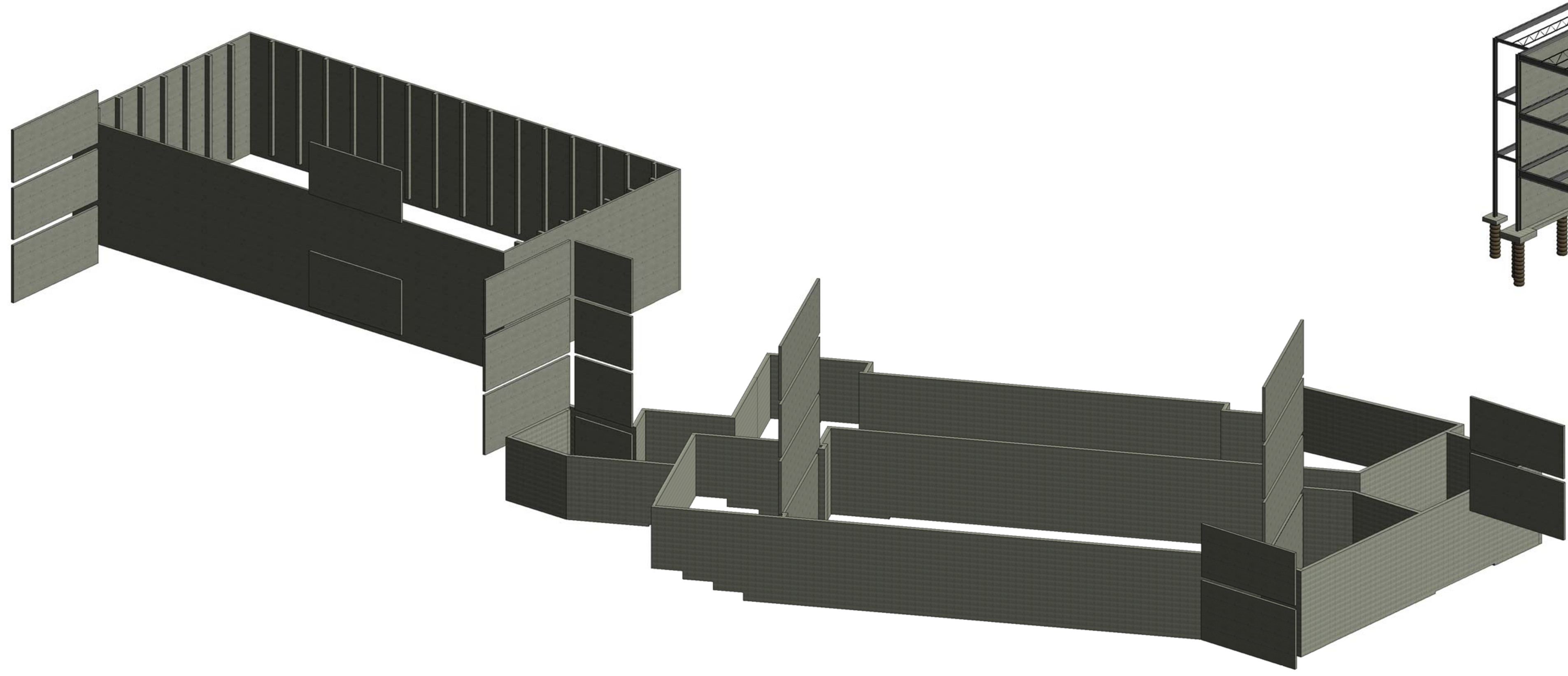


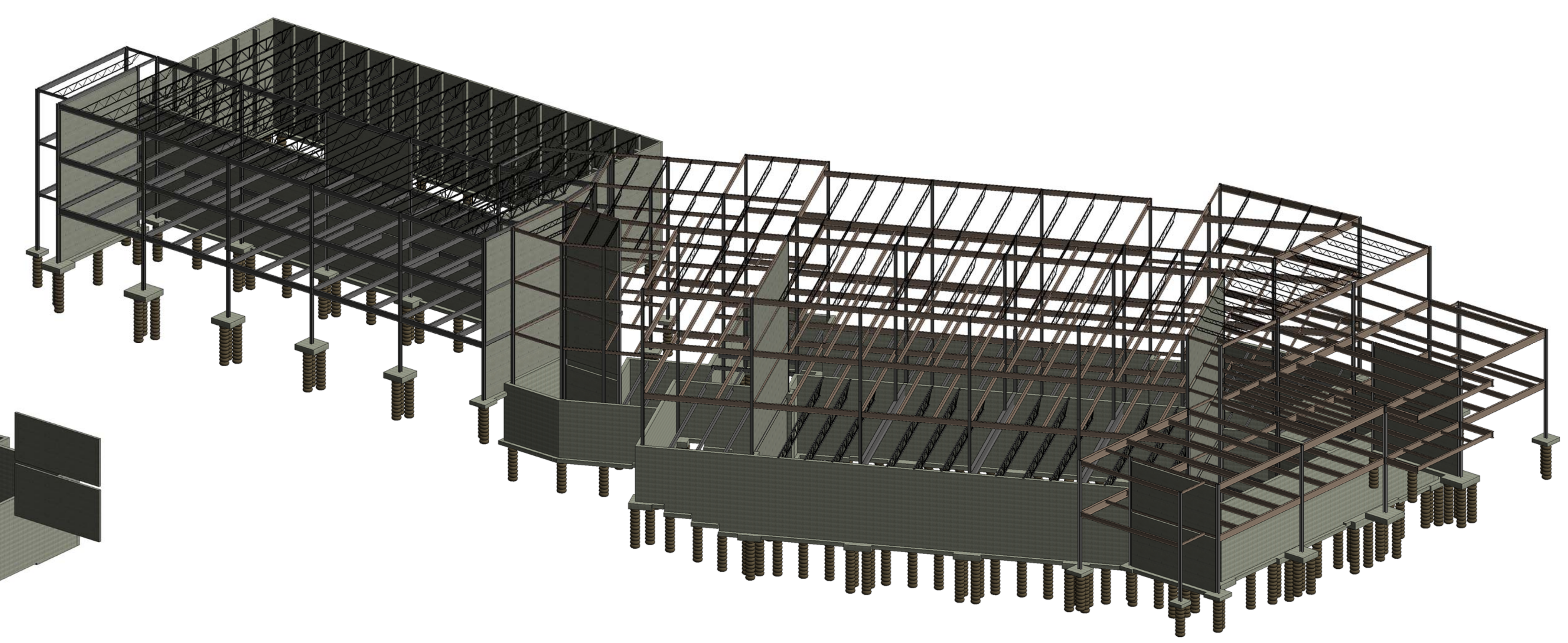
1 3-D STRUCTURE PERSPECTIVE - FOUNDATION ONLY



2 3-D STRUCTURE PERSPECTIVE - STEEL ONLY



3 3-D STRUCTURE PERSPECTIVE - WALLS AND HYBRID MASONRY ONLY



4 3-D STRUCTURE PERSPECTIVE - ALL SYSTEMS

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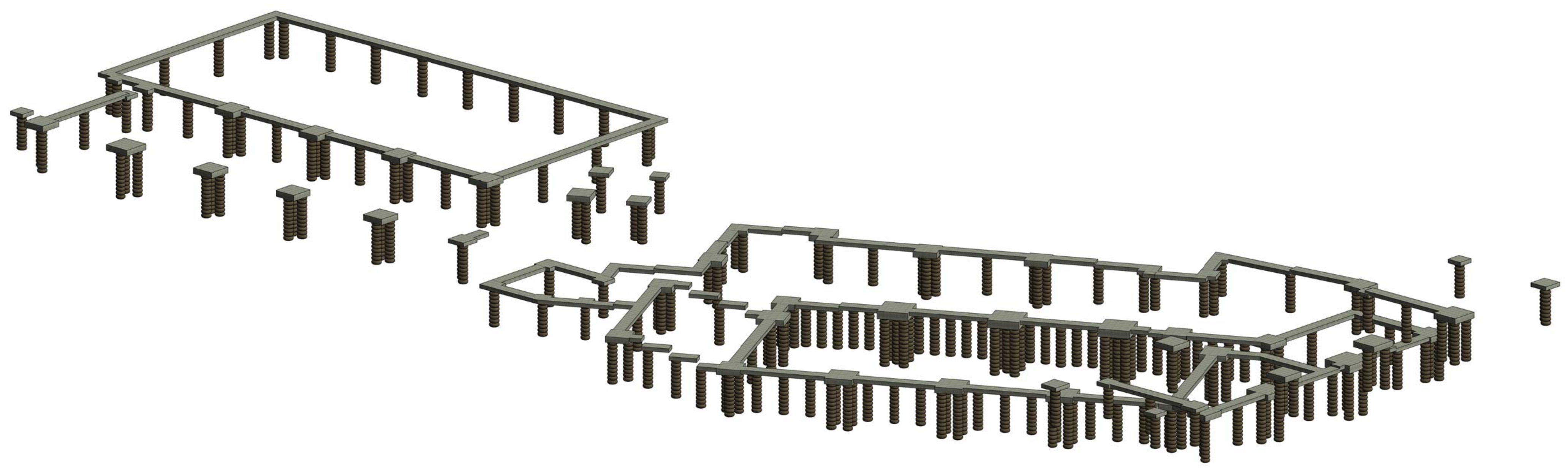
3-D STRUCTURE PERSPECTIVES

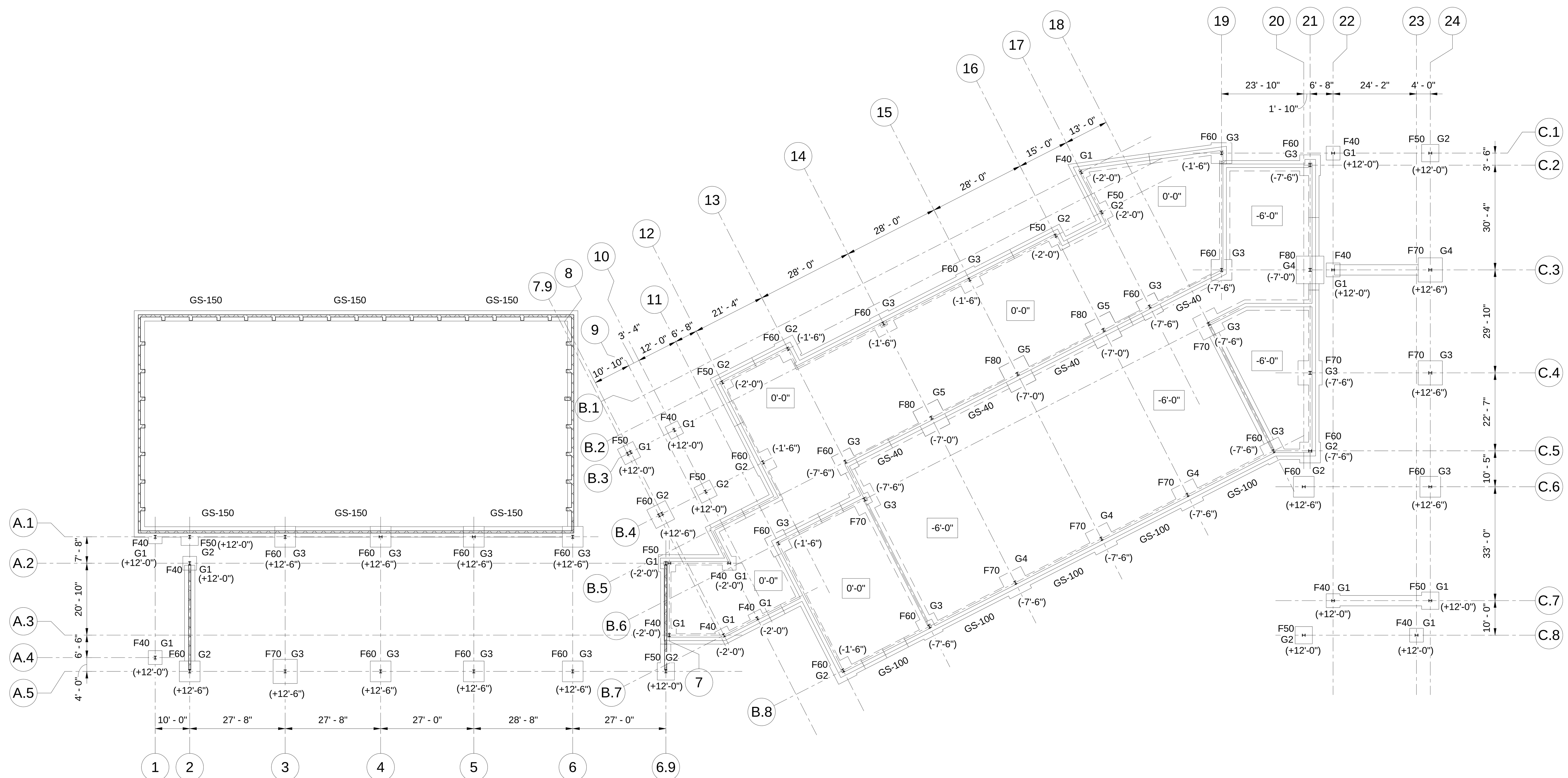
Date 02.22.2013

S-001

Scale



1 3-D FOUNDATION IMAGE



0 FOUNDATION PLAN
1/16" = 1'-0"

FOUNDATION PLAN NOTES

1.) T/SLAB ELEVATION = +351'-0" (DATUM) UNLESS OTHERWISE NOTED.

2.) +/- X'-X" = T/SLAB ELEVATION ABOVE/BELOW DATUM.

3.) (+/- X'-X") = T/FTG. ELEVATION ABOVE/BELOW DATUM.

4.) F# - COLUMN FOOTING MARK (SEE FOOTING SCHEDULE).

5.) G# - GEOPIER SUPPORT MARK (SEE GEOPIER SUPPORT SCHEDULE).

6.) PROVIDE GS-200 (20'-0" SPACING) GEOPIER SUPPORT UNDER ALL STRIP FOOTINGS UNLESS OTHERWISE NOTED ON DRAWING.

7.) PROVIDE APPROPRIATE NUMBER OF GEOPIERS SYMMETRICALLY UNDER ISOLATED FOOTINGS.

8.) FOOTING CONSTRUCTION = (f'c = 4000 psi, GR 60 REINFORCING).

9.) SOIL BEARING CAPACITY WITH GEOPIER SUPPORT = 5000 psf

10.) TYPICAL SLAB-ON-GRADE CONSTRUCTION IS 6" NW (f'c = 4000 psi) w/ 6x6-W2.1x2.1 W.W.F.

11.) ALL COLUMNS CENTERED ON COLUMN CENTERLINES.

CENTRAL WING GRID DIMENSIONS

B.1 - B.2 = 7'-0"
B.2 - B.3 = 6'-0"
B.3 - B.4 = 20'-0"
B.4 - B.5 = 10'-8"
B.5 - B.6 = 12'-3"
B.6 - B.7 = 16'-7"
B.7 - B.8 = 24'-9"

ANGLE = 27 DEGREES FROM HORIZONTAL

FOOTING SCHEDULE

MARK	SIZE	DEPTH	REINFORCING	GEOPIER DEPTH
F40	4'-0" x 4'-0"	1'-0"	(7) #5 BARS EACH WAY	9'-0"
F50	5'-0" x 5'-0"	1'-0"	(7) #5 BARS EACH WAY	10'-0"
F60	6'-0" x 6'-0"	1'-6"	(8) #5 BARS EACH WAY	11'-0"
F70	7'-0" x 7'-0"	1'-6"	(9) #5 BARS EACH WAY	12'-0"
F80	8'-0" x 8'-0"	2'-0"	(14) #5 BARS EACH WAY	13'-0"

GEOPIER SUPPORT SCHEDULE

MARK	# OF GEOPIERS (ISOLATED FOOTINGS)	MAXIMUM GEOPIER SPACING (STRIP FOOTINGS)
G1	1	-
G2	2	-
G3	3	-
G4	4	-
G5	5	-
GS-40	-	4'-0"
GS-100	-	10'-0"
GS-150	-	15'-0"
GS-200	-	20'-0"

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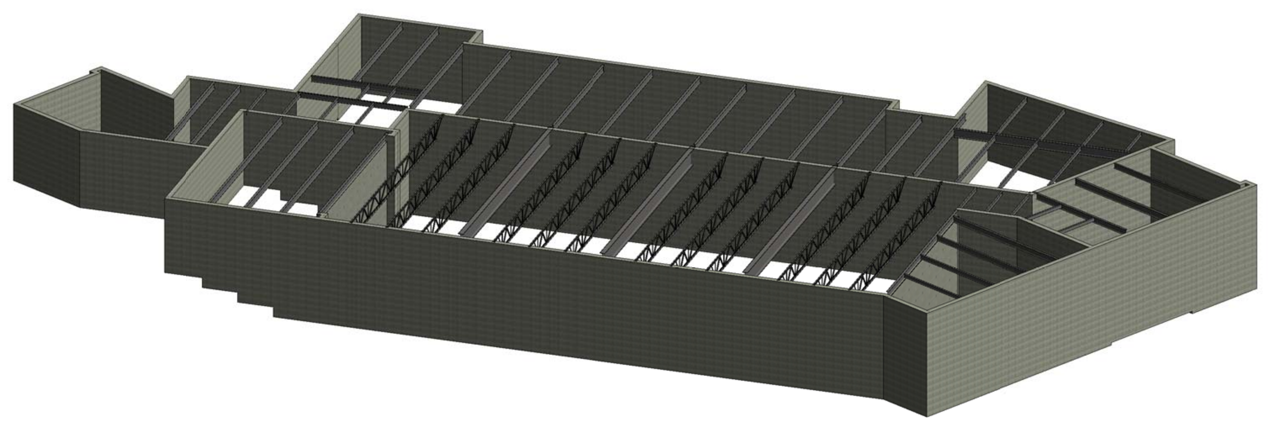
FOUNDATION PLAN
AND DETAILS

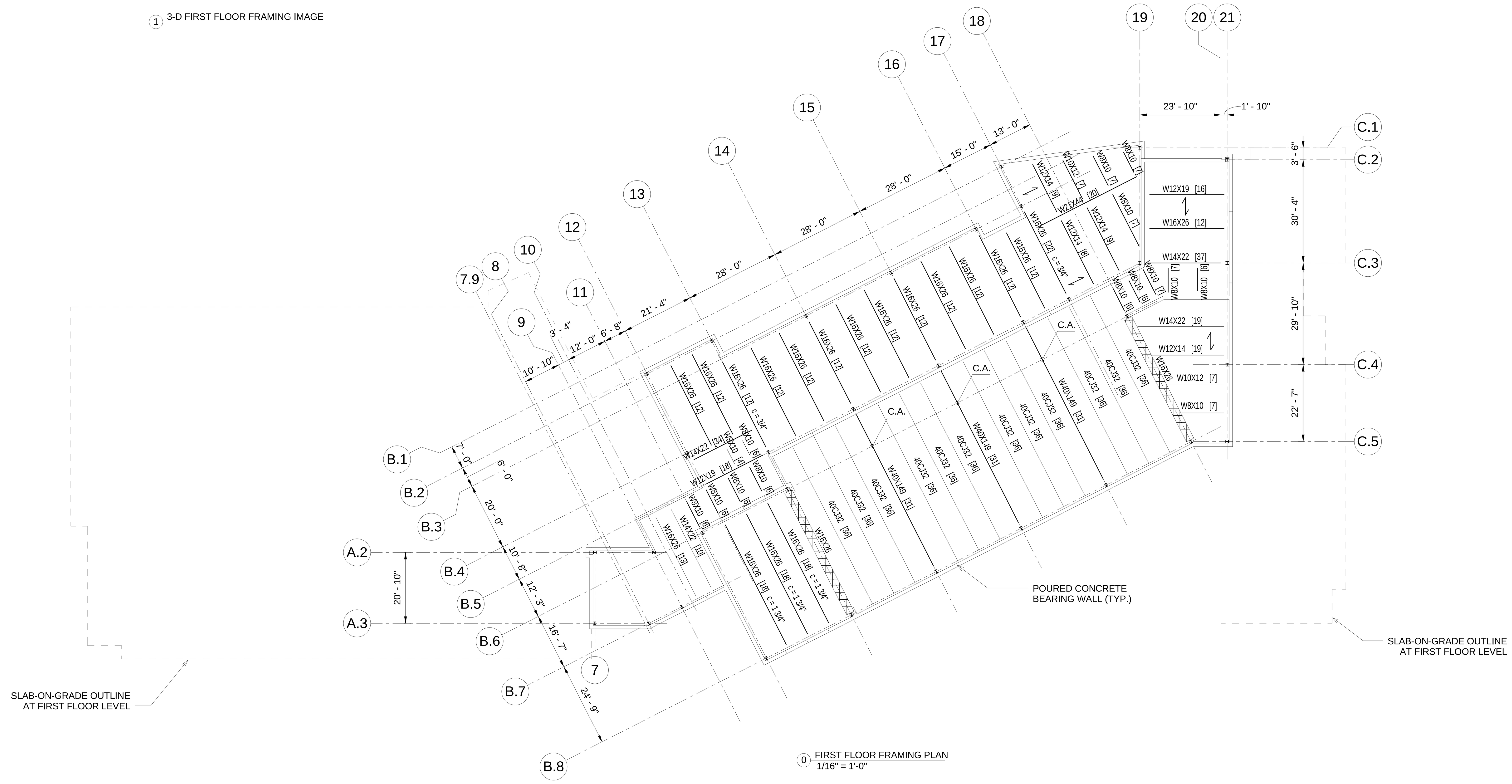
Date 02.22.2013

S-100

Scale 1/16" = 1'-0"



1 3-D FIRST FLOOR FRAMING IMAGE



0 FIRST FLOOR FRAMING PLAN
1/16" = 1'-0"

FIRST FLOOR FRAMING PLAN NOTES

1.) T/SLAB ELEVATION = +365'-0" (DATUM) UNLESS OTHERWISE NOTED.

2.) T/STL ELEVATION (-0'-5 1/4") FROM DATUM UNLESS OTHERWISE NOTED.

3.) TYP. FLOOR CONSTRUCTION = LW CONC. (f'c = 4000 psi @ 28 DAYS) ON DECK (2" 20 GAGE GALVANIZED COMPOSITE FLOOR DECK VULCRAFT OR APPROVED EQUIVALENT).

TOTAL SLAB THICKNESS = 5 1/4"

MAX UNSHORED SINGLE SPAN = 8'-5"
MAX UNSHORED 2-SPAN = 10'-7"
MAX UNSHORED 3-SPAN = 10'-11"

4.) = HYBRID MASONRY FRAME - SEE S-300

5.) TYPICAL HYBRID WALL CONSTRUCTION = 8" FULLY-GROUTED CMU w/ #4 BARS @ 40" O.C.

6.) [#] = # OF 3/4" DIAM. 4" LONG (AFTER WELDING) SHEAR STUDS.

7.) 6" SLAB OVERHANG.

8.) STEEL = ASTM-A992 GRADE 50 HIGH-STRENGTH.

CENTRAL WING GRID DIMENSIONS

ANGLE = 27 DEGREES FROM HORIZONTAL

AEI TEAM
10-2013

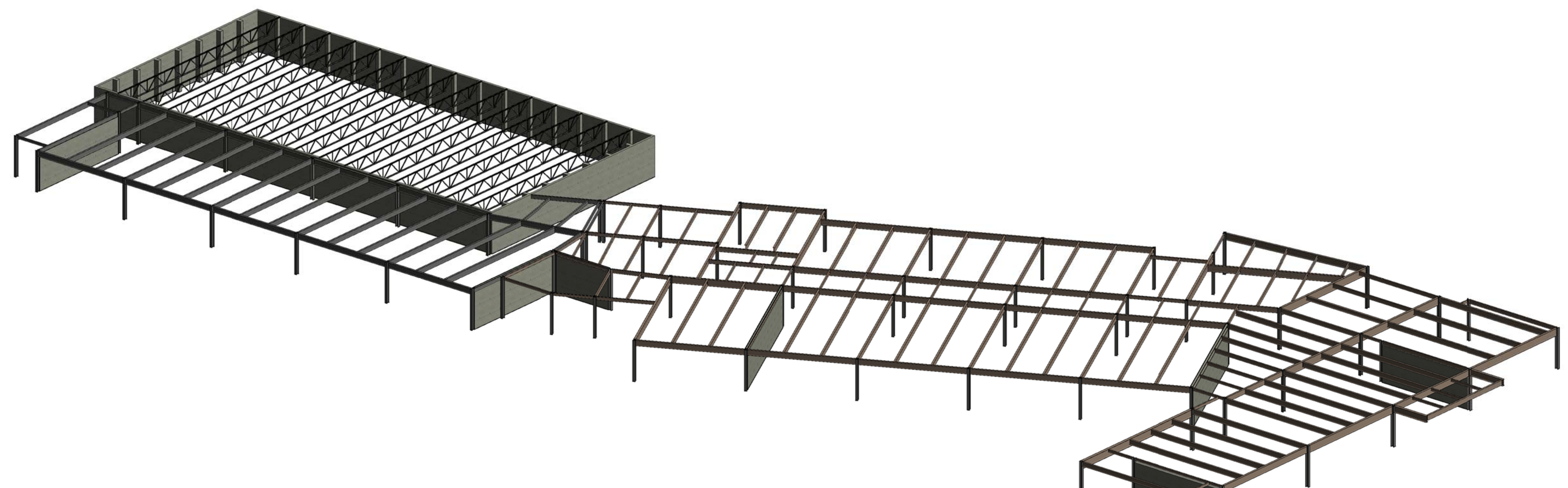
READING
ELEMENTARY SCHOOL

FIRST FLOOR
FRAMING PLAN

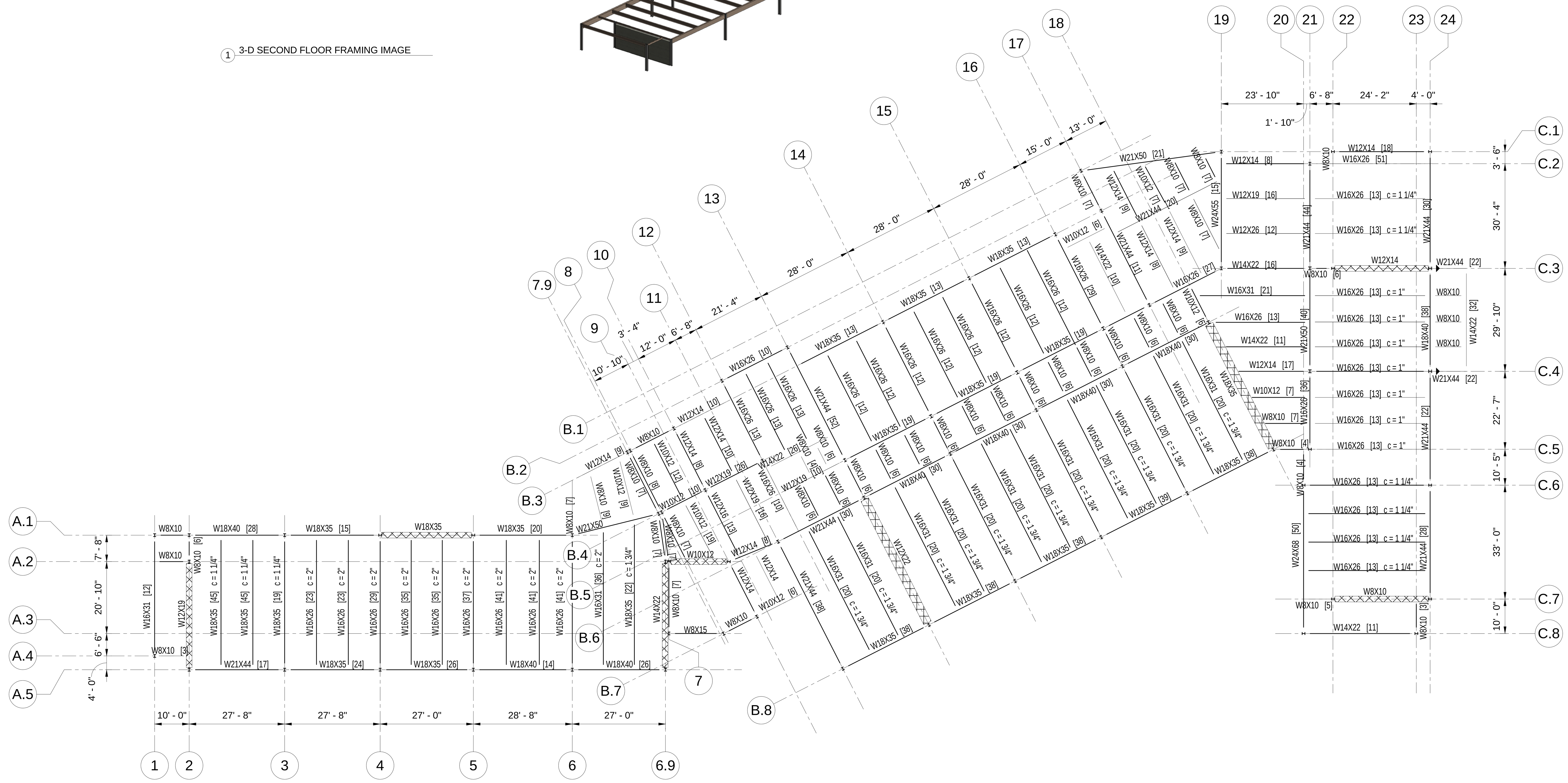
Date 02.22.2013

S-101

Scale 1/16" = 1'-0"



1 3-D SECOND FLOOR FRAMING IMAGE



0 SECOND FLOOR FRAMING PLAN
1/16" = 1'-0"

SECOND FLOOR FRAMING PLAN NOTES

1.) T/SLAB ELEVATION = +379'-0" (DATUM) UNLESS OTHERWISE NOTED.

2.) T/STL ELEVATION (-0'-5 1/4") FROM DATUM UNLESS OTHERWISE NOTED.

3.) TYP. FLOOR CONSTRUCTION = LW CONC. (f'c = 4000 psi @ 28 DAYS) ON DECK (2" 20 GAGE GALVANIZED COMPOSITE FLOOR DECK VULCRAFT OR APPROVED EQUIVALENT).

TOTAL SLAB THICKNESS = 5 1/4"

MAX UNSHORED SINGLE SPAN = 8'-5"
MAX UNSHORED 2-SPAN = 10'-7"
MAX UNSHORED 3-SPAN = 10'-11"

4.) = HYBRID MASONRY FRAME - SEE S-300

5.) TYPICAL HYBRID WALL CONSTRUCTION = 8" FULLY-GROUTED CMU w/ #4 BARS @ 40" O.C.

6.) [#] = # OF 3/4" DIAM. 4" LONG (AFTER WELDING) SHEAR STUDS.

7.) 6" SLAB OVERHANG.

8.) STEEL = ASTM-A992 GRADE 50 HIGH-STRENGTH.

CENTRAL WING GRID DIMENSIONS

B.1 - B.2 = 7'-0"
B.2 - B.3 = 6'-0"
B.3 - B.4 = 20'-0"
B.4 - B.5 = 10'-8"
B.5 - B.6 = 12'-3"
B.6 - B.7 = 16'-7"
B.7 - B.8 = 24'-9"

ANGLE = 27 DEGREES FROM HORIZONTAL

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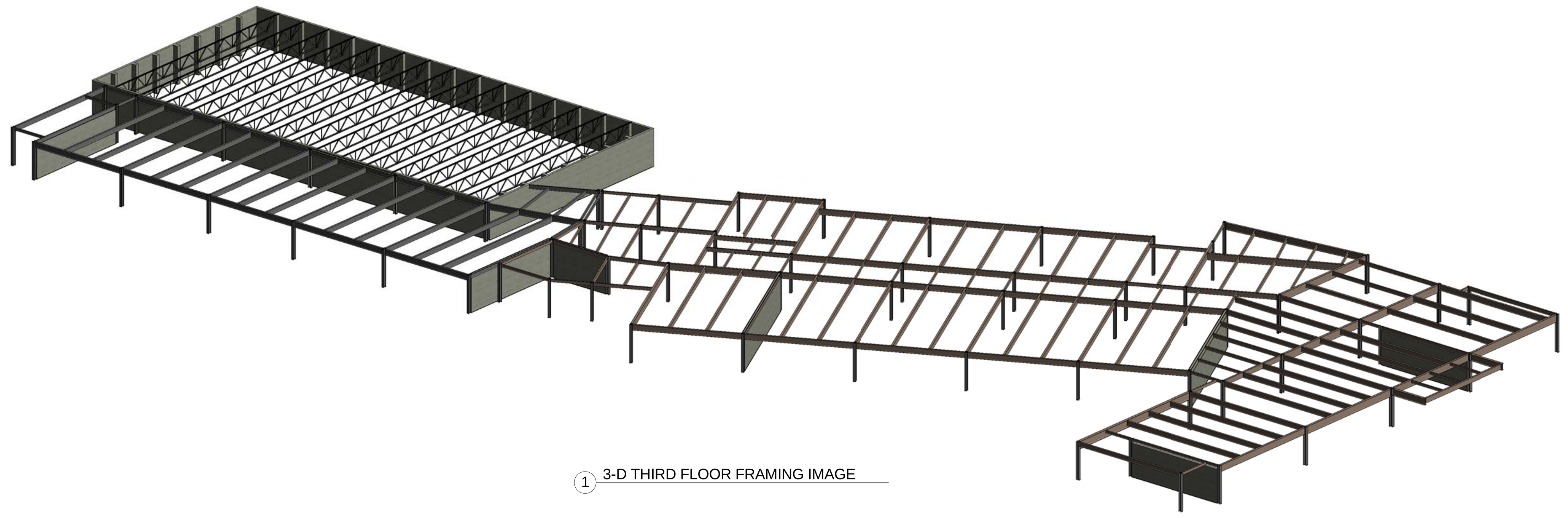
SECOND FLOOR FRAMING PLAN

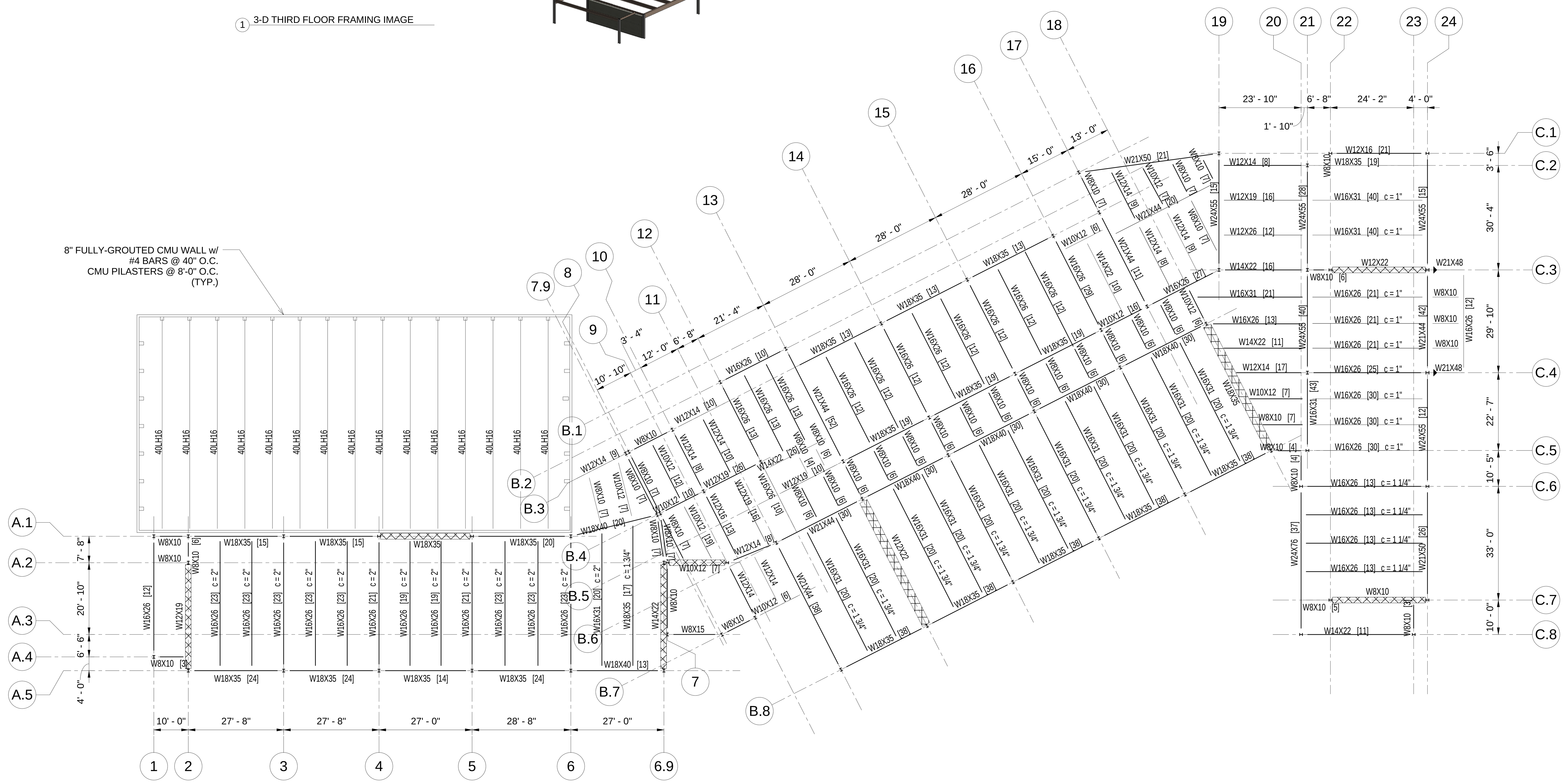
Date 02.22.2013

S-102

Scale 1/16" = 1'-0"



1 3-D THIRD FLOOR FRAMING IMAGE



0 THIRD FLOOR FRAMING PLAN
1/16" = 1'-0"

THIRD FLOOR FRAMING PLAN NOTES

1.) T/SLAB ELEVATION = +393'-0" (DATUM) UNLESS OTHERWISE NOTED.

2.) T/STL ELEVATION (-0'-5 1/4") FROM DATUM UNLESS OTHERWISE NOTED.

3.) TYP. FLOOR CONSTRUCTION = LW CONC. (f'c = 4000 psi @ 28 DAYS) ON DECK (2" 20 GAGE GALVANIZED COMPOSITE FLOOR DECK VULCRAFT OR APPROVED EQUIVALENT).

TOTAL SLAB THICKNESS = 5 1/4"

MAX UNSHORED SINGLE SPAN = 8'-5"
MAX UNSHORED 2-SPAN = 10'-7"
MAX UNSHORED 3-SPAN = 10'-11"

4.) = HYBRID MASONRY FRAME - SEE S-300

5.) TYPICAL HYBRID WALL CONSTRUCTION = 8" FULLY-GROUTED CMU w/ #4 BARS @ 40" O.C.

6.) [#] = # OF 3/4" DIAM. 4" LONG (AFTER WELDING) SHEAR STUDS.

7.) 6" SLAB OVERHANG.

8.) STEEL = ASTM-A992 GRADE 50 HIGH-STRENGTH.

CENTRAL WING GRID DIMENSIONS

B.1 - B.2 = 7'-0"
B.2 - B.3 = 6'-0"
B.3 - B.4 = 20'-0"
B.4 - B.5 = 10'-8"
B.5 - B.6 = 12'-3"
B.6 - B.7 = 16'-7"
B.7 - B.8 = 24'-9"

ANGLE = 27 DEGREES FROM HORIZONTAL

AEI TEAM
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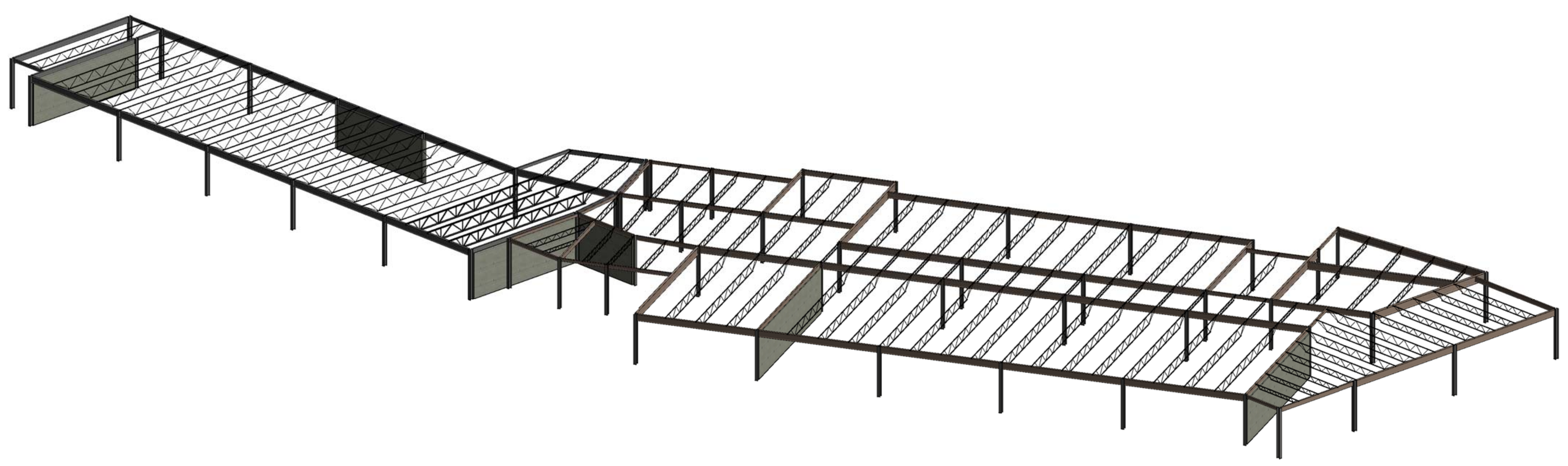
READING
ELEMENTARY SCHOOL

THIRD FLOOR
FRAMING PLAN

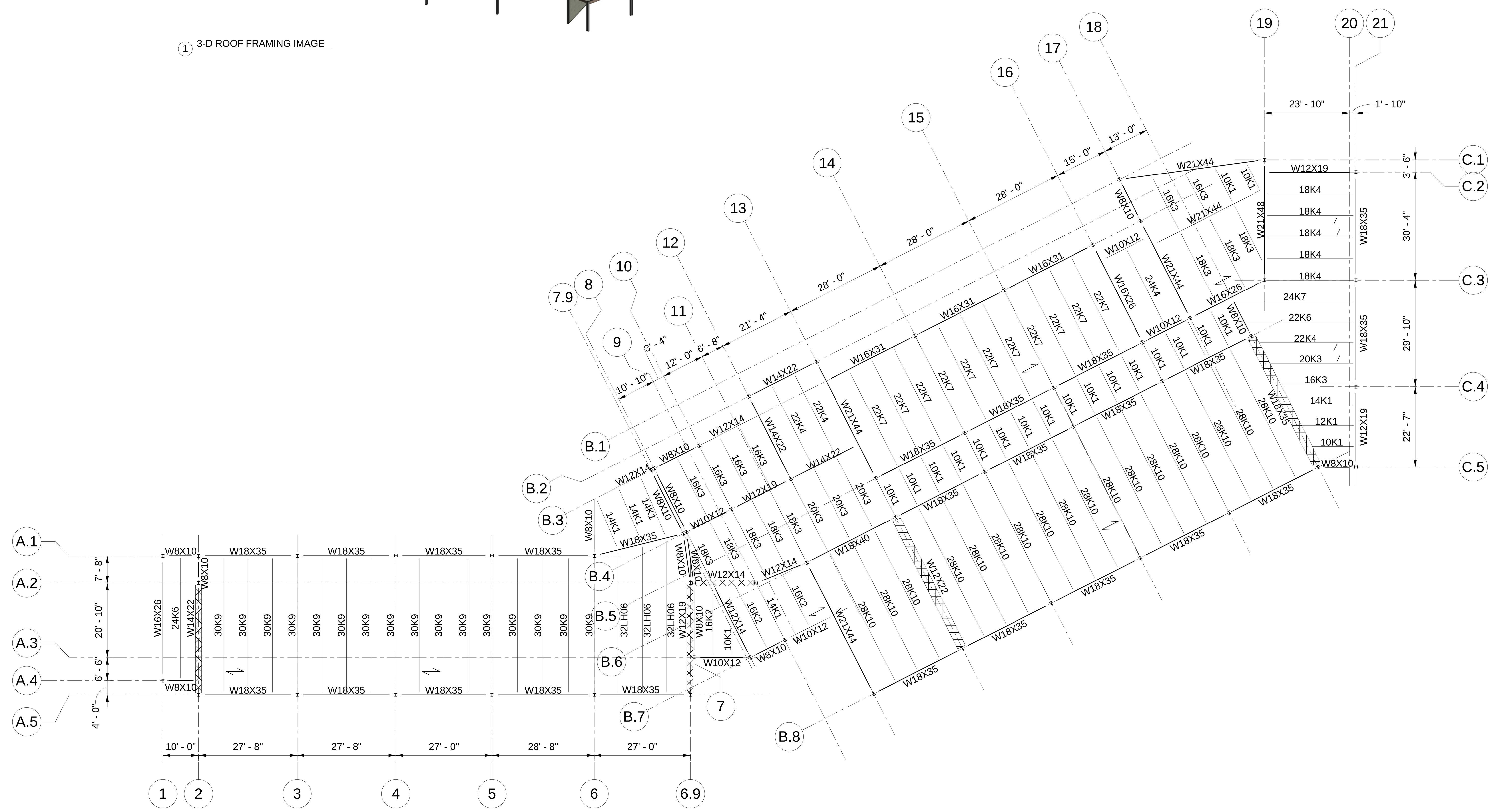
Date 02.22.2013

S-103

Scale 1/16" = 1'-0"



3-D ROOF FRAMING IMAGE



ROOF FRAMING PLAN
1/16" = 1'-0"

ROOF FRAMING PLAN NOTES

1.) T/DECK ELEVATION = +407'-0" (DATUM) UNLESS OTHERWISE NOTED.

2.) T/STL ELEVATION (-0'-1 1/2") FROM DATUM UNLESS OTHERWISE NOTED.

3.) TYP. ROOF CONSTRUCTION = 1.5" 19 GAGE WIDE RIB GALVANIZED ROOF DECK (VULCRAFT OR APPROVED EQUIVALENT).

MAX UNSHORED SINGLE SPAN = 7'-1"
MAX UNSHORED 2-SPAN = 8'-5"
MAX UNSHORED 3-SPAN = 8'-5"

4.) = HYBRID MASONRY FRAME - SEE S-300

5.) TYPICAL HYBRID WALL CONSTRUCTION = 8" FULLY-GROUTED CMU w/ #4 BARS @ 40" O.C.

6.) STEEL = ASTM-A992 GRADE 50 HIGH-STRENGTH.

CENTRAL WING GRID DIMENSIONS

B.1 - B.2 = 7'-0"
B.2 - B.3 = 6'-0"
B.3 - B.4 = 20'-0"
B.4 - B.5 = 10'-8"
B.5 - B.6 = 12'-3"
B.6 - B.7 = 16'-7"
B.7 - B.8 = 24'-9"

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ROOF FRAMING PLAN

Date 02.22.2013

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Scale 1/16" = 1'-0"

COLUMN LOCATIONS	A.1-1	A.1-2	A.1-3	A.1-4	A.1-5	A.1-6	A.2-2	A.2-6.9	A.2-7	A.2-9	A.3-7	A.4-1	A.5-2	A.5-3	A.5-4	A.5-5	A.5-6	A.5-6.9	B.1-17	B.2-12	B.2-13	B.3-7.9	B.3-8	COLUMN LOCATIONS
ROOF T.O.S.																								ROOF T.O.S.
406' - 10 1/2" THIRD FLOOR T.O.S.			W10X33	W10X33		W10X33		W10X33	W10X33	W10X33	W10X33		W10X33	W10X33	W10X33	W10X33	W10X33		W10X33	W10X33	W10X33	W10X33		406' - 10 1/2" THIRD FLOOR T.O.S.
392' - 6 3/4" SECOND FLOOR T.O.S.	W10X33	W10X33			W10X33		W10X33					W10X33						W10X33					W10X33	392' - 6 3/4" SECOND FLOOR T.O.S.
378' - 6 3/4" FIRST FLOOR T.O.S.			W10X45	W10X39		W10X39		W10X33	W10X33	W10X33	W10X33		W10X39	W10X45	W10X39	W10X39	W10X39		W10X33	W10X33	W10X33	W10X33		378' - 6 3/4" FIRST FLOOR T.O.S.
364' - 6 3/4" BASEMENT																								364' - 6 3/4" BASEMENT
351' - 0" POOL																								351' - 0" POOL
345' - 0"																								345' - 0"
BASE PLATES	12" x 10" x 3/8"	12" x 10" x 1/2"	12" x 10" x 5/8"	12" x 10" x 3/8"	12" x 10" x 3/8"	12" x 10" x 5/8"	12" x 12" x 1/4"	12" x 12" x 1/4"	12" x 12" x 1/4"	12" x 12" x 1/4"	12" x 10" x 3/8"	12" x 10" x 3/8"	12" x 12" x 1/4"	12 1/4" x 10 1/4" x 1/4"	12" x 10" x 5/8"	12" x 10" x 5/8"	12" x 10" x 5/8"	12" x 12" x 1/4"	12" x 10" x 1/2"	12" x 10" x 1/2"	12" x 10" x 5/8"	12" x 10" x 3/8"	12" x 10" x 3/8"	BASE PLATES
COLUMN LOCATIONS	A.1-1	A.1-2	A.1-3	A.1-4	A.1-5	A.1-6	A.2-2	A.2-6.9	A.2-7	A.2-9	A.3-7	A.4-1	A.5-2	A.5-3	A.5-4	A.5-5	A.5-6	A.5-6.9	B.1-17	B.2-12	B.2-13	B.3-7.9	B.3-8	COLUMN LOCATIONS

COLUMN LOCATIONS	B.3-10	B.3-14	B.3-15	B.3-16	B.3-17	B.4-7.9	B.4-8	B.4-10	B.4-12	B.5-13	B.5-14	B.5-15	B.5-16	B.5-17	B.6-11	B.6-13	B.6-14	B.6-15	B.6-16	B.6-18	B.7-8	B.7-9	B.8-11	COLUMN LOCATIONS
ROOF T.O.S.																								ROOF T.O.S.
406' - 10 1/2" THIRD FLOOR T.O.S.		W10X33	W10X33	W10X33	W10X33				W10X33	W10X33	W10X33	W10X33	W10X33	W10X33	W10X33	W10X33				W10X33	W10X33	W10X33	W10X33	406' - 10 1/2" THIRD FLOOR T.O.S.
392' - 6 3/4" SECOND FLOOR T.O.S.	W10X33					W10X33	W10X33	W10X33									W10X33	W10X33	W10X33					392' - 6 3/4" SECOND FLOOR T.O.S.
378' - 6 3/4" FIRST FLOOR T.O.S.		W10X33	W10X33	W10X33	W10X33				W10X33						W10X39						W10X33	W10X33	W10X33	378' - 6 3/4" FIRST FLOOR T.O.S.
364' - 6 3/4" BASEMENT										W10X39	W10X54	W10X54	W10X49	W10X33		W10X49				W10X49				364' - 6 3/4" BASEMENT
351' - 0" POOL																								351' - 0" POOL
345' - 0"																								345' - 0"
BASE PLATES	12" x 10" x 1/2"	12" x 10" x 5/8"	12" x 10" x 5/8"	12" x 10" x 1/2"	12" x 10" x 1/2"	12" x 10" x 1/2"	12" x 10" x 3/8"	12" x 10" x 1/2"	12" x 10" x 5/8"	12" x 10" x 3/4"	12" x 12" x 1/4"	12" x 12" x 1/4"	12" x 12" x 1"	12" x 10" x 5/8"	12" x 10" x 3/4"	12" x 12" x 3/4"	12" x 12" x 3/4"	12" x 12" x 3/4"	12" x 12" x 3/4"	12" x 12" x 3/4"	12" x 12" x 3/4"	12" x 10" x 3/8"	12" x 10" x 1/2"	BASE PLATES
COLUMN LOCATIONS	B.3-10	B.3-14	B.3-15	B.3-16	B.3-17	B.4-7.9	B.4-8	B.4-10	B.4-12	B.5-13	B.5-14	B.5-15	B.5-16	B.5-17	B.6-11	B.6-13	B.6-14	B.6-15	B.6-16	B.6-18	B.7-8	B.7-9	B.8-11	COLUMN LOCATIONS

COLUMN LOCATIONS	B.8-13	B.8-14	B.8-15	B.8-16	B.8-18	C.1-19	C.1-22	C.1-24	C.2-21	C.3-19	C.3-21	C.3-22	C.3-24	C.4-21	C.4-24	C.5-21	C.6-20	C.6-24	C.7-22	C.7-24	C.8-20	C.8-23	COLUMN LOCATIONS
ROOF T.O.S.																							ROOF T.O.S.
406' - 10 1/2" THIRD FLOOR T.O.S.	W10X33	W10X33	W10X33	W10X33	W10X33	W10X33			W10X33	W10X33	W10X33			W10X39		W10X39							406' - 10 1/2" THIRD FLOOR T.O.S.
392' - 6 3/4" SECOND FLOOR T.O.S.																							392' - 6 3/4" SECOND FLOOR T.O.S.
378' - 6 3/4" FIRST FLOOR T.O.S.						W10X33	W10X33	W10X39				W10X33	W10X68		W10X68		W10X39	W10X39	W10X33	W10X33	W10X33	W10X33	378' - 6 3/4" FIRST FLOOR T.O.S.
364' - 6 3/4" BASEMENT	W10X33	W10X45	W10X45	W10X45	W10X33				W10X33	W10X33	W10X49			W10X49		W10X39							364' - 6 3/4" BASEMENT
351' - 0" POOL																							351' - 0" POOL
345' - 0"																							345' - 0"
BASE PLATES	12 1/4" x 10 1/4" x 5/8"	12 1/4" x 10 1/4" x 5/8"	12 1/4" x 10 1/4" x 5/8"	12 1/4" x 10 1/4" x 5/8"	12" x 12" x 1/4"	12" x 10" x 5/8"	12" x 10" x 1/4"	12" x 10" x 1/2"	12" x 10" x 5/8"	12" x 10" x 3/4"	12" x 12" x 7/8"	12" x 12" x 3/4"	12" x 12" x 3/4"	12" x 12" x 7/8"	12" x 12" x 3/4"	12" x 10" x 1/2"	12" x 10" x 5/8"	12" x 10" x 5/8"	12" x 12" x 3/4"	12" x 12" x 3/4"	12" x 10" x 1/2"	12" x 10" x 1/4"	BASE PLATES
COLUMN LOCATIONS	B.8-13	B.8-14	B.8-15	B.8-16	B.8-18	C.1-19	C.1-22	C.1-24	C.2-21	C.3-19	C.3-21	C.3-22	C.3-24	C.4-21	C.4-24	C.5-21	C.6-20	C.6-24	C.7-22	C.7-24	C.8-20	C.8-23	COLUMN LOCATIONS

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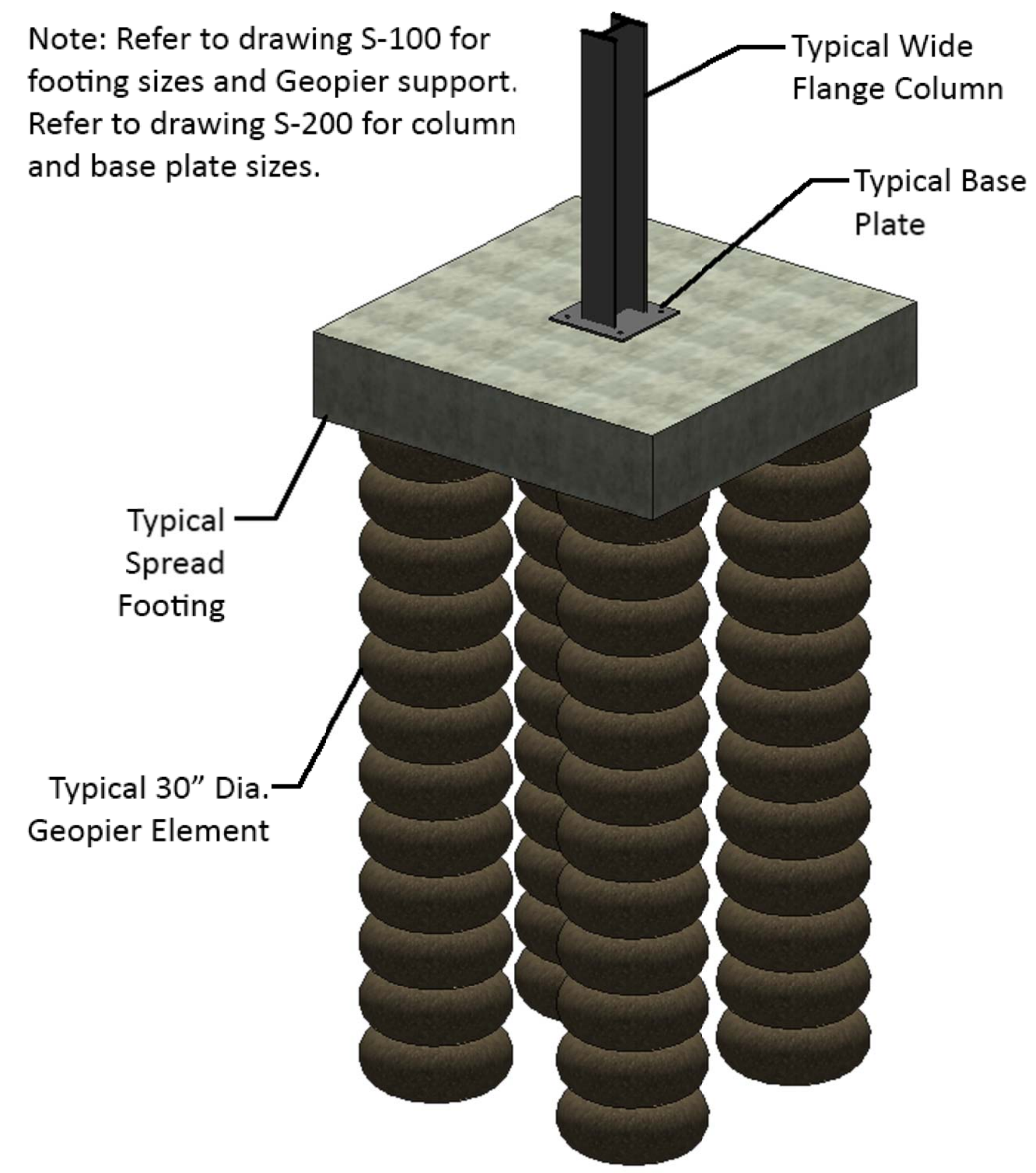
READING
ELEMENTARY SCHOOL

COLUMN SCHEDULE

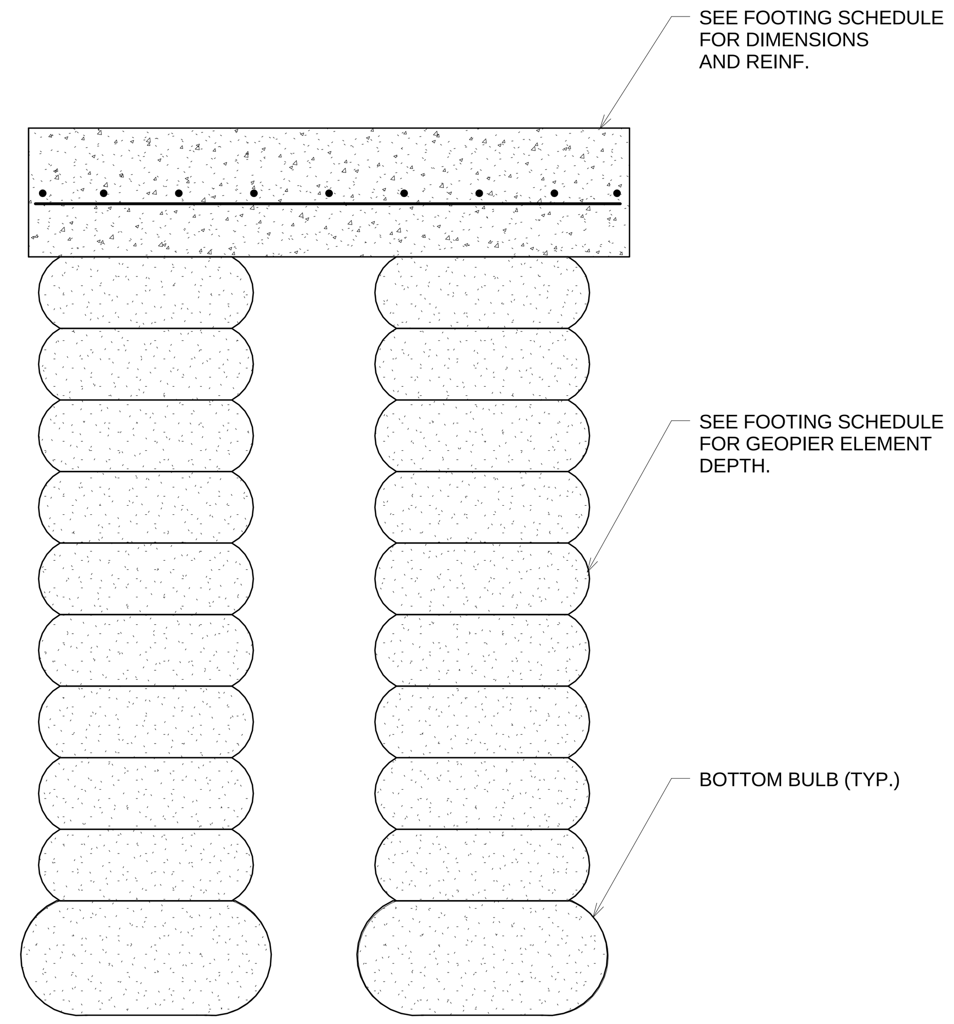
Date 02.22.2013

S-200

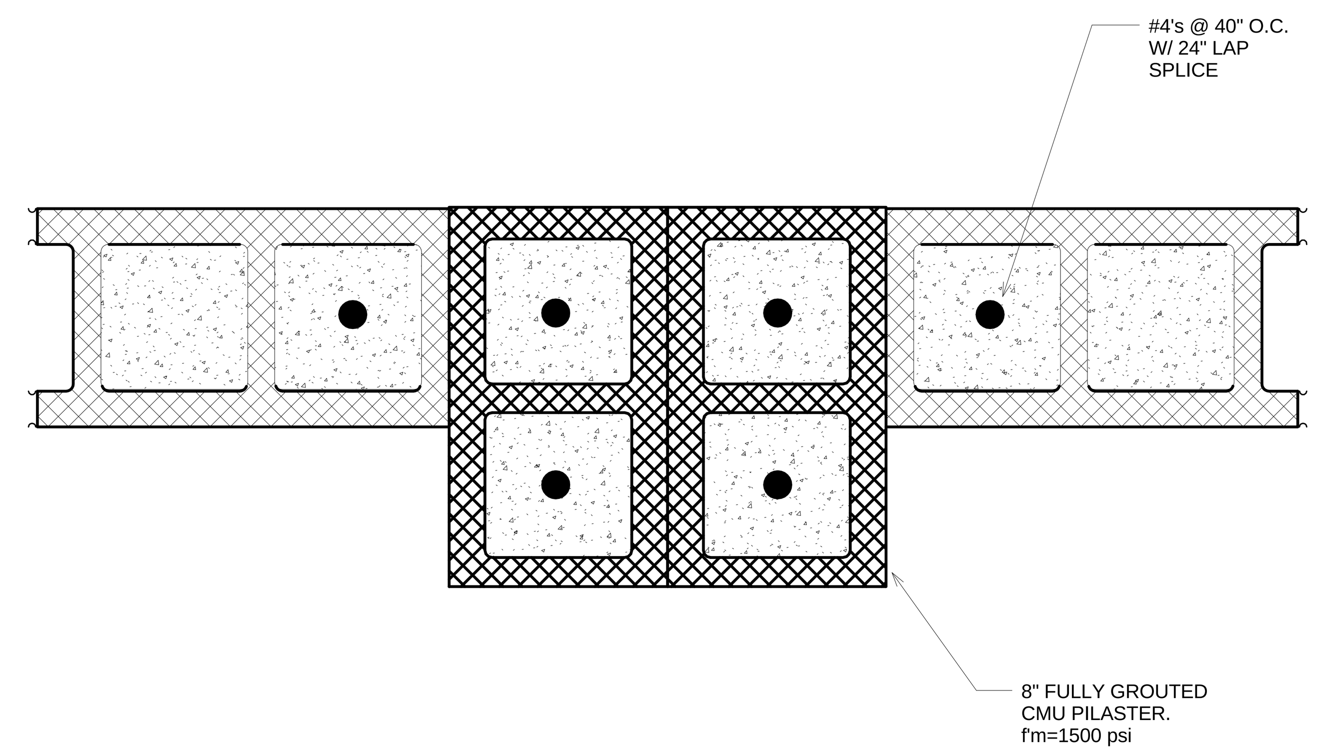
Scale



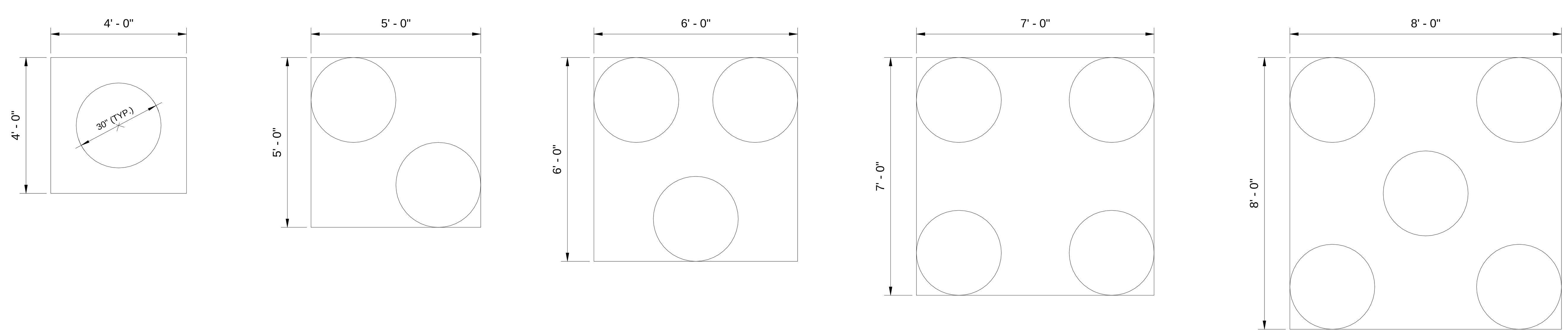
(1) TYPICAL SPREAD FOOTING WITH GEOPIER SUPPORT



(2) TYPICAL GEOPIER ELEVATION
3/4" = 1'-0"



(3) TYPICAL MULTIPURPOSE AREA MASONRY PILASTER
3" = 1'-0"



(4) TYPICAL GEOPIER ELEMENT LAYOUTS PER GEOPIER SOIL REINFORCEMENT MANUAL
1/2" = 1'-0"

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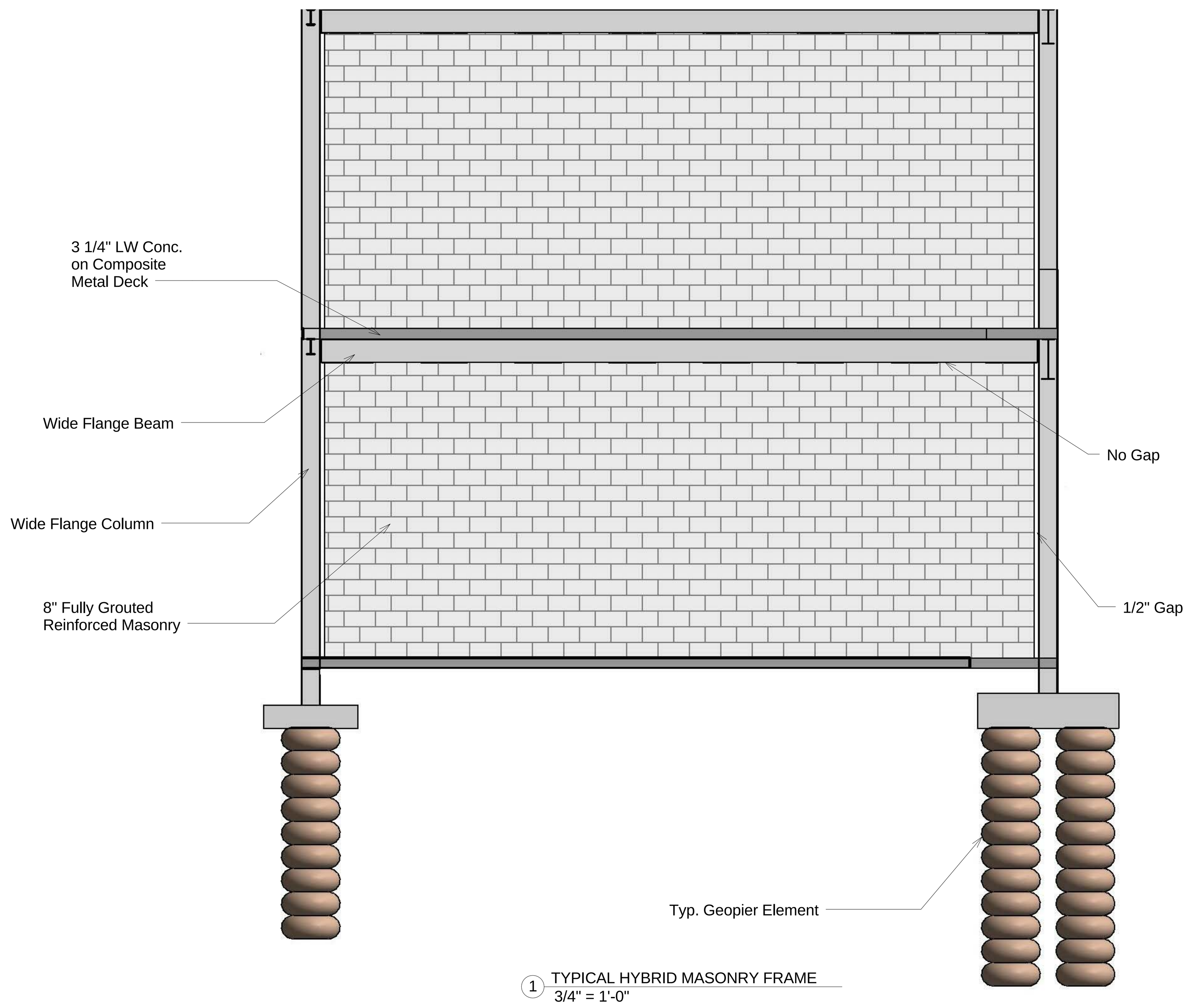
READING ELEMENTARY SCHOOL

FOUNDATION AND WALL DETAILS

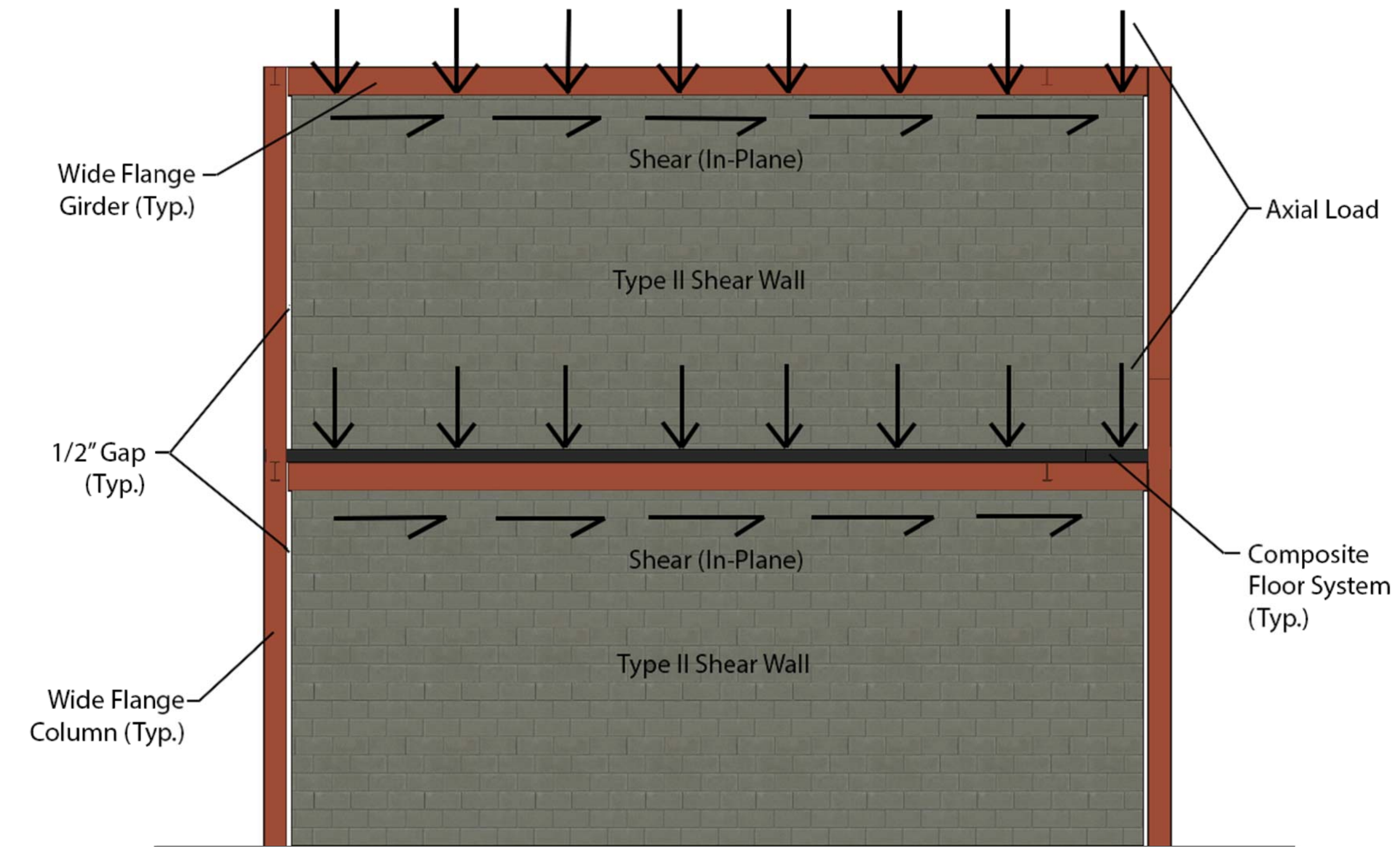
Date 02.22.2013

S-300

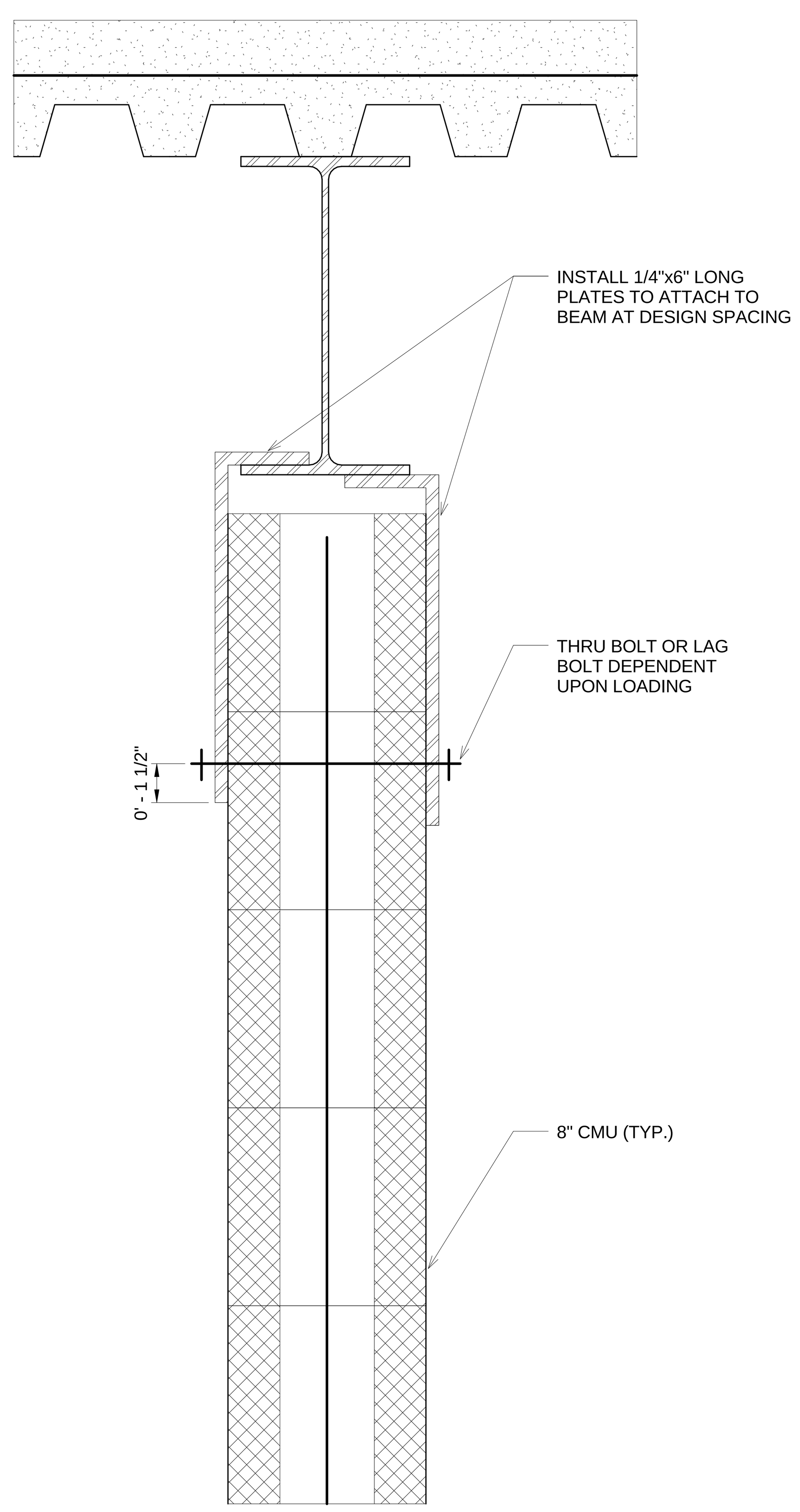
Scale As indicated



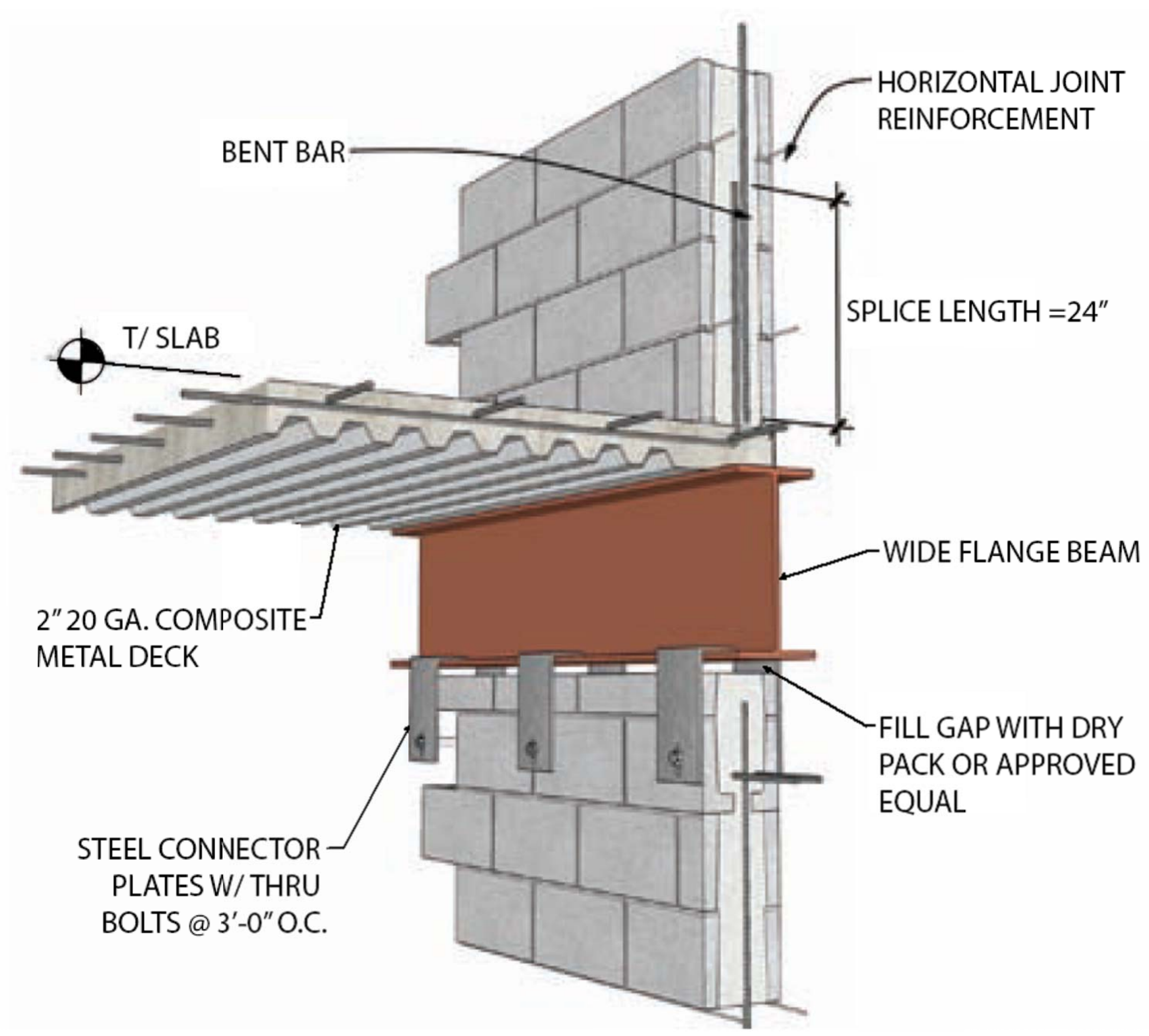
1 TYPICAL HYBRID MASONRY FRAME
3/4" = 1'-0"



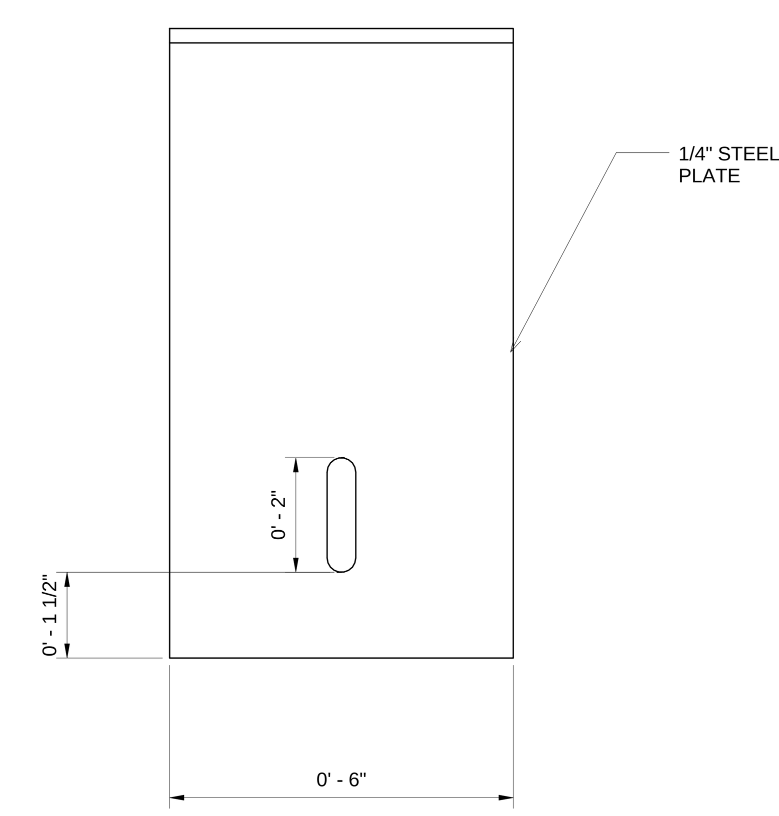
2 HYBRID WALL ELEVATION WITH LOAD SCHEMATIC



3 HYBRID MASONRY WALL SECTION
3" = 1'-0"



4 ISOMETRIC HYBRID FRAME SECTION



5 TYPICAL HYBRID CONNECTOR PLATE
6" = 1'-0"

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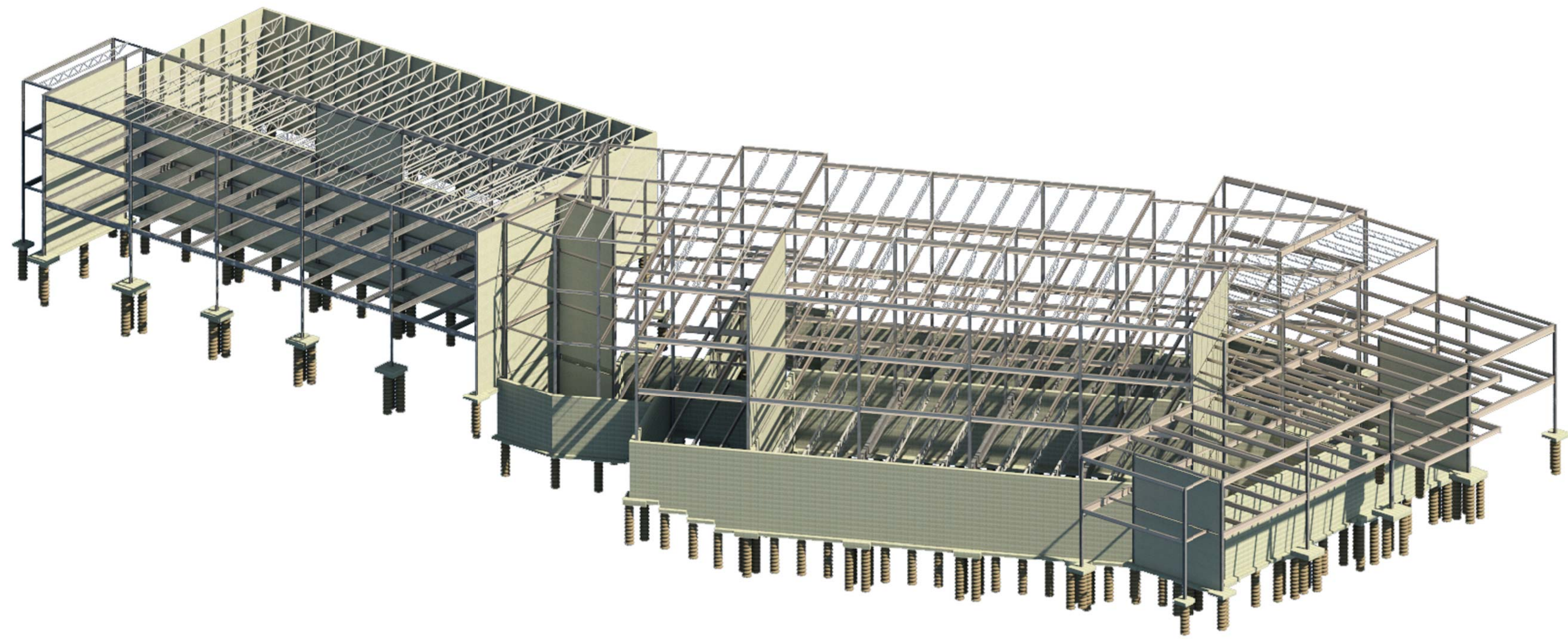
READING ELEMENTARY SCHOOL

HYBRID MASONRY DETAILS

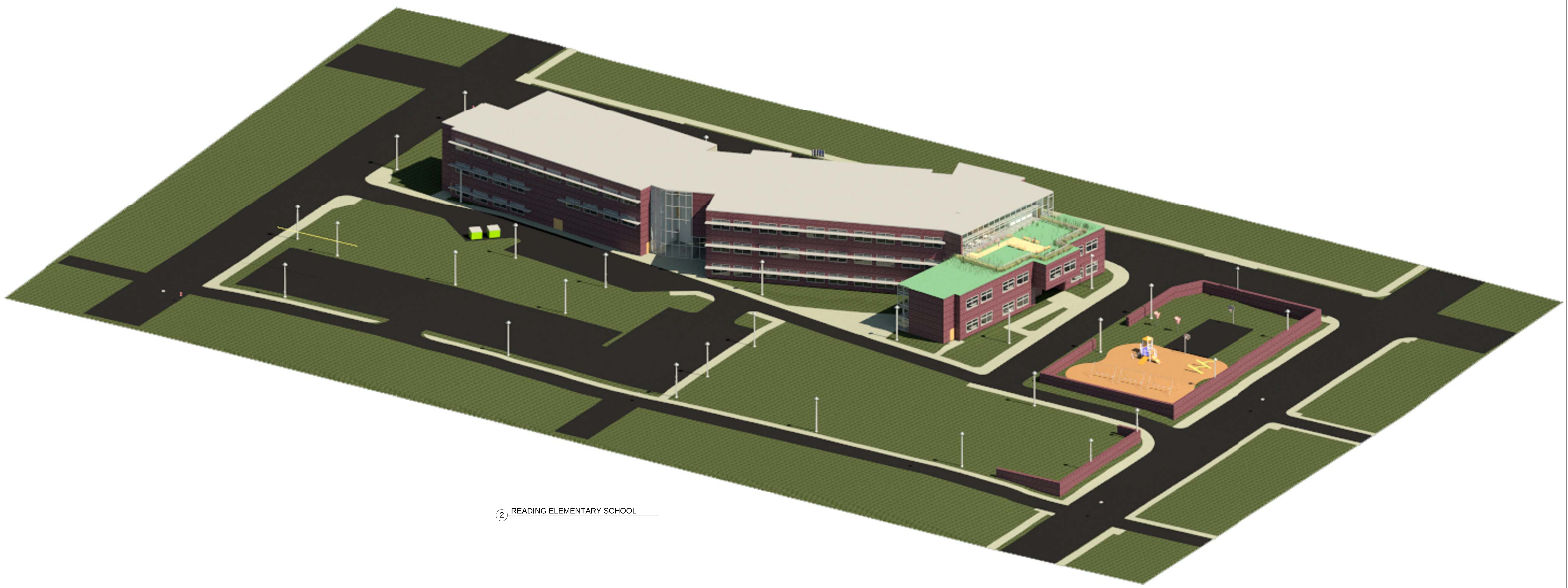
Date	02.22.2013

S-301

Scale	As indicated



1 READING ELEMENTARY SCHOOL
STRUCTURAL SYSTEM



2 READING ELEMENTARY SCHOOL

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READING ELEMENTARY SCHOOL

PERSPECTIVE RENDERINGS

Date 02.22.2013

S-400

Scale