Reinsurance Group of America (RGA) Global Headquarters

Technical Report I

Structural Concepts & Existing Conditions

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13 September 2013

Executive Summary

The purpose of this technical report is to explore the existing structural systems in the Reinsurance Group of America's Global Headquarters. This included preliminary investigation of the gravity and lateral systems and unique structural features of the project. It looks at the main structural components and their influence on the load paths for wind, seismic, soil and gravity, which influence the main structural systems.

This preliminary research was executed by reviewing project documents, primarily drawings, and tracking the gravity and lateral systems throughout the buildings. Findings of the systems' functionality and influence on other pieces of the project were then recorded and supporting information compiled into the body of this report.

The 40' cantilever is supported by truss system. The vertical compression members sit on a steel plate girder that spans between them and the overturning moment is resisted by the horizontal roof framing members designed to take axial tension. The roof framing distributes the large tension load along the length of the roof.

Soil load is counteracted by perimeter beams in the parking garage. These beams are isolated from the post-tensioned slab so that compressive load from the soil does not distribute into the post-tensioned slab. Surcharge load, however, is resisted by a tie-back anchor system that distributes the load into the surrounding soil and has been designed by a specialty firm.

In terms of lateral load resistance, the braced frames from the superstructure are pin connected to the concrete shear walls of the substructure. The braces are connected to an embed plate whose studs are embedded into the concrete shear wall. This allows the transfer of horizontal forces from the braces into the concrete shear wall. Vertical loads are transferred through the column base plate and anchor rods which are embedded 5' into the concrete column below.

In conclusion, critical structural features that will influence future analysis are the 40' cantilever truss system, maintaining the integrity of the soil load path, and maintaining the integrity of the lateral system and especially the connection between the substructure and superstructure.

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Purpose and Scope

The purpose of this technical report is to investigate and describe the existing structural conditions in the Reinsurance Group of America's Global Headquarters as designed by Uzun & Case Engineers. It includes a primary exploration of the main structural systems including gravity and lateral systems, unique structural features, relevant design codes, and load paths present in the design. In addition, it provides an overview of main structural components such as floor systems, typical bays, earth retaining systems, lateral systems, joint details, and building materials.

The scope of this report includes the building complex as a whole, with a few limitations. First, I will focus on the north office tower because the north and south office towers have mirrored floor plans. Secondly, for this report I will investigate the parking garage, but in future assignments I will not be focusing on the post-tensioning aspects of the parking garage in my analysis. I will include its contributions to the lateral load resisting systems and its structural considerations and relationship with the superstructure moving forward.

General Building Information

The Reinsurance Group of America's Global Headquarters serves as an office and training facility for RGA- a Fortune 500 Company. This building complex features two office towers enclosed by curtain wall façades with a lobby and amenities space linking the two towers. Inside the office towers is an open floor plan with a centrally located core that maximizes tenant circulation through the building as well as flexibility and functionality within the space. From the highway on the lower side of the site, the two parking garage levels are visible. On the opposing side, these levels are below grade, allowing for a third level of on-grade parking and fire truck access.

Figure 1: Rendering from Highway, Courtesy Gensler

Construction on this 405,000 square foot, $150 million project started in March 2013 and will continue until its expected completion in September, 2014. A Phase Two plan has been developed for the addition of a third office tower similar to the Phase One towers with additional parking to service the new tower. The site, seen below in Figure 2, features three bio-retention basins along the highway. This Design-Build project, at the request of the owner, utilized the LEED Silver Accreditation standards as a design basis.

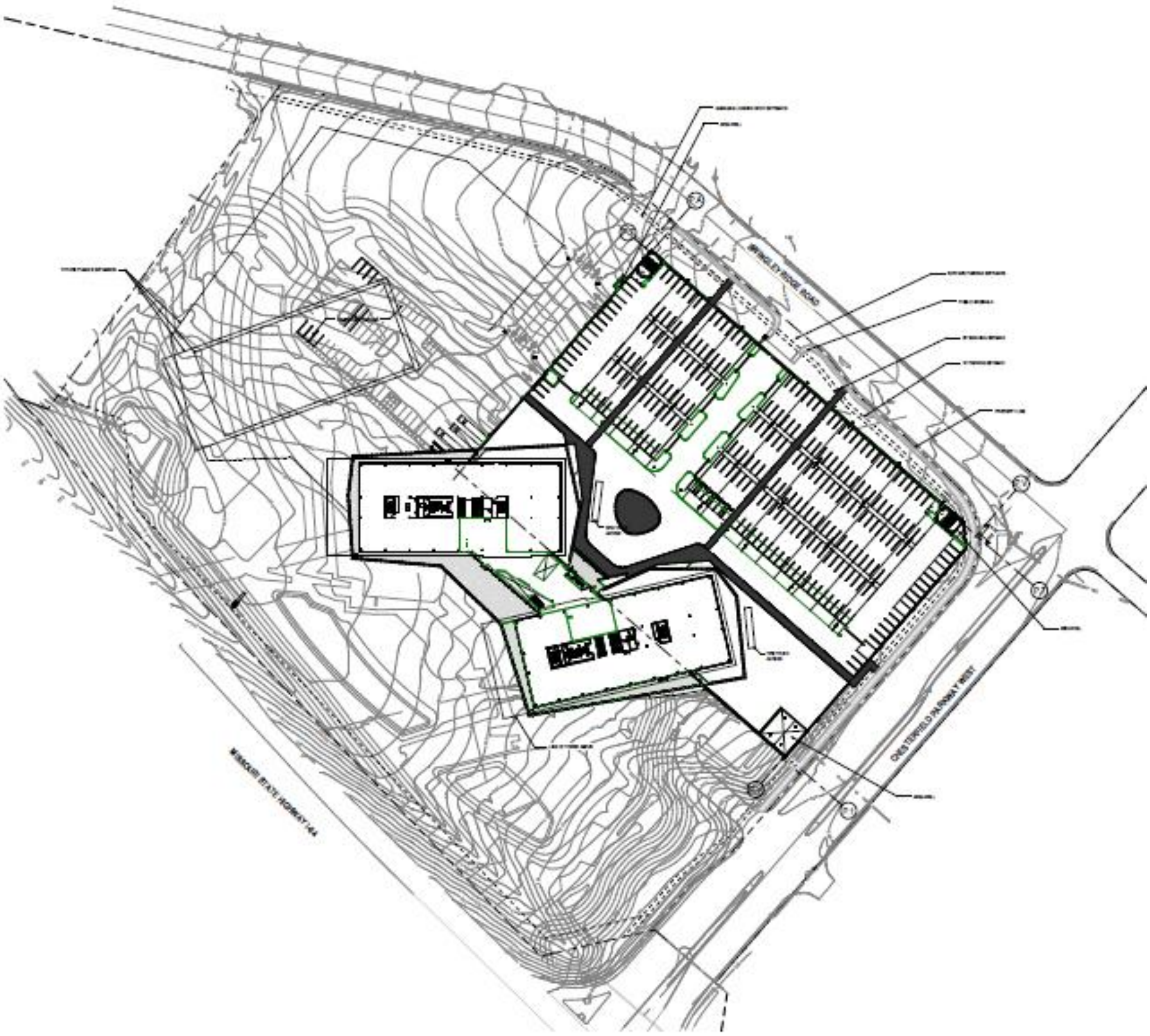


Figure 2: Site Plan Oriented to True North

Structural System Overview

RGA Global Headquarters has two five story, steel and curtain wall office buildings with mirrored, rectangular floor plans. Floors two through five are cantilevered 5' over the first floor on three sides and 40’ on the fourth side. A truss system bearing on a built up-plate girder supports the large cantilever. All exposed steel is Architecturally Exposed Structural Steel (AESS) at the owner's request. The office buildings have a braced frame lateral system that transfers load into concrete shear walls in the below grade parking garage. Post-tensioned one-way slab systems supported by post-tensioned concrete beams comprise the parking garage's structure and support the loading above at the parking levels. The foundation consists of grade beams supported by concrete drilled piers, with the exception of a portion of the site where the bedrock rises to meet the parking garage; there the foundation is a rock bearing spread footing. This section of the report will provide more detail into these systems.

Foundation

A geotechnical report was conducted by SCI Engineering, Inc. in October, 2012, as a follow-up to a previous report they had done in January, 1999. Based on their findings, SCI Engineering recommended drilled pier foundations, rock bearing shallow foundations, aggregate piers, and shallow foundations; however, SCI recommended the use of multiple systems as the most feasible. Predominant soils in the area were the topsoil, clays, areas of shale and unknown infill, and bedrock with groundwater appearing about 37' to 60' below the existing grade.

Drilled piers are the predominant foundation system selected, bearing on bedrock, with an allowable end bearing pressure of 80 ksf and a concrete compressive strength of 3,000 psi. Pier diameters range from 36" to 78" with vertical reinforcement of #8 bars through #11 bars and #4 ties. Minimum pier length is specified as 2.2 times the pier diameter, but when the depth to competent bedrock is less than this minimum, rock bearing spread footings should take their place. When tension piers are required, rock anchors with a 150 ksi minimum ultimate tensile strength embedded a minimum of 10' into the limestone bedrock and lapped 8'-4" with the vertical reinforcement in the pier, are required to achieve sufficient anchorage. An overall detail is shown below in Figure 3 and a more complete detail is available in Appendix A. Tension piers are found most commonly supporting the lateral system. Pier caps are typically 3' to 4' in depth and are 6" to 1' wider on either side of the pier with caps supporting the shear walls typically the larger. The rock bearing spread footings are designed for an 8,000 psf net allowable bearing pressure with reinforcement of #8 through #10 bars. Soil beneath these footings is replaced with 2,000 psi lean concrete. In the case of a footing bearing on soil, a net allowable bearing capacity 2,500 psf is recommended.

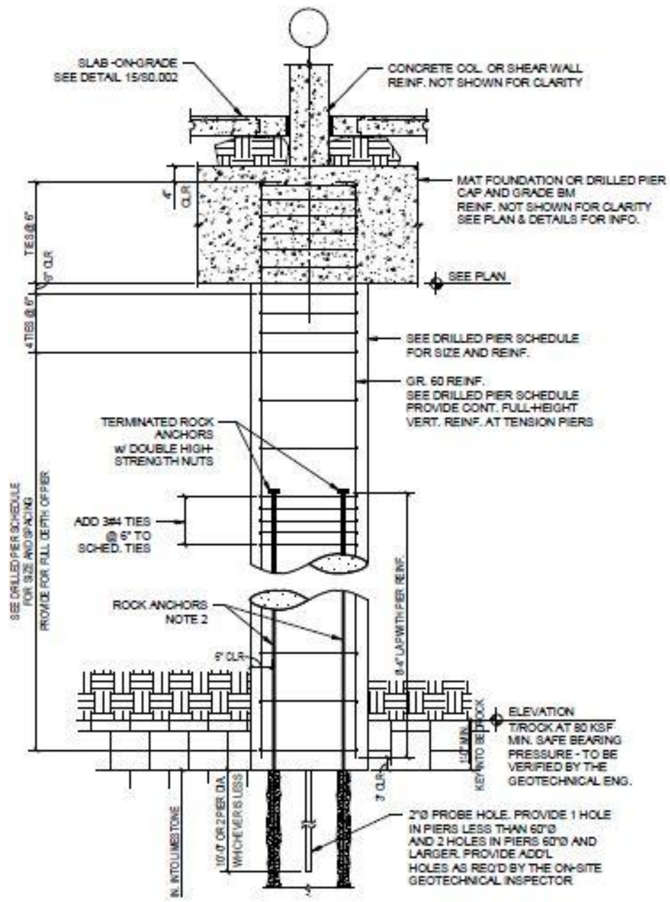


Figure 3: Typical Tension Pier Detail

The final component of the foundation system is the grade beams. They are typically 4,000 psi concrete ranging in size from 18"x18" to 42"x24" with several combinations in between. Reinforcement is Grade 60 and ranges from #8 bars through #11 bars with #4 stirrups. A typical detail is shown below in Figure 4.

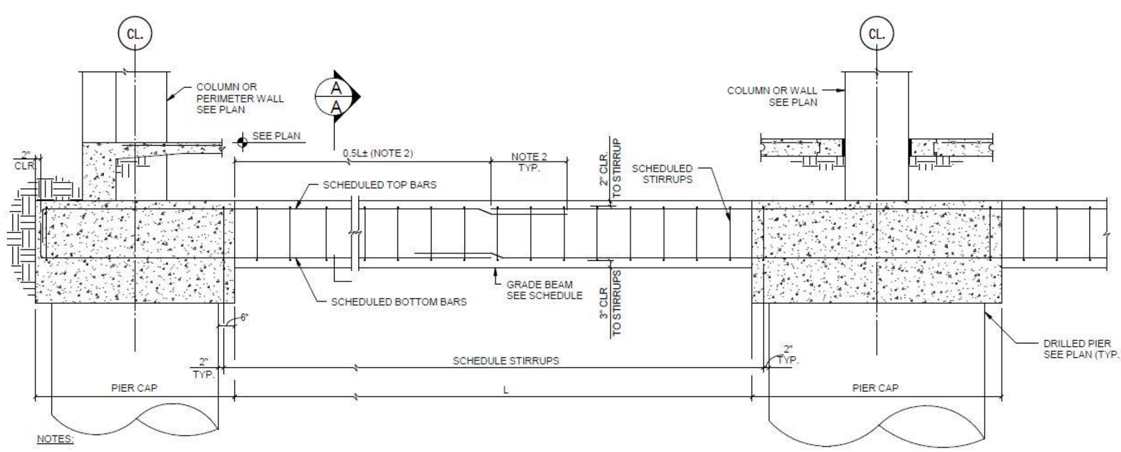


Figure 4: Typical Grade Beam Detail

Substructure

The lowest level of the parking garage is a slab on grade supported by grade beams. For the parking garage, the thickness is 5" of 3,500 psi concrete placed on compacted subgrade. Mechanical rooms, loading docks and truck service area slabs on this level are 6" thick. Concrete exterior walls on this level are typically 16" thick. At wall corners and ends, U-bars matching the local horizontal reinforcement are used with Class B lap splices. At wall intersections, 90 degree standard hooks are used with Class B laps. A representative framing plan of the substructure can be found in Appendix E.

The floor of the upper parking level increases in thickness to 7" and the floor system changes to a 5,000 psi concrete post-tensioned, one-way slab system supported by post-tensioned reinforced concrete beams. Exterior exposed concrete walls are 8" thick and increase to 12" when they are exposed to earth, below level 01 on the higher side of the site.

The slab of the parking plaza, the on-grade level of parking, is also a post-tensioned one-way slab system supported by post-tensioned beams. The difference lies in the parking plaza's slab thickness. If there is no fire truck access, the slab is 8 1/2" thick and slabs with fire truck access areas are 9 1/2" thick.

Columns in the parking garage are typically 5,000 psi concrete with the exception of four columns of 7000 psi concrete . These columns are continuations of the columns supporting the plate girder and compression members of the cantilever truss system. Square or rectangular column sizes range from 16"x16" up through 32"x32" with the most common size being 24"x24". Vertical reinforcement ranges from #8 to #11 bars. Circular columns range from a 24" diameter to a 36" diameter with the most common diameter being 28".

Superstructure

This section discusses typical bay characteristics and area-specific characteristics that cause the bay configuration in that area to differ from the typical bay. A representative structural framing plan for the superstructure can be found in Appendix C.

Typical Bay Characteristics

In a typical bay, gravity columns are A992 Grade 50 steel with typical sizes of W10x49, W12x65, W12x79, W12x87, W12x136 on lower levels and W12x65, W12x58, W12x53 on upper levels. When necessary, column splices occur 4' above Level 04. Beam sizes are discussed below. Bays are based on a 30' or 40' length and either a 25' width or a 40' width as shown in Figure 5.

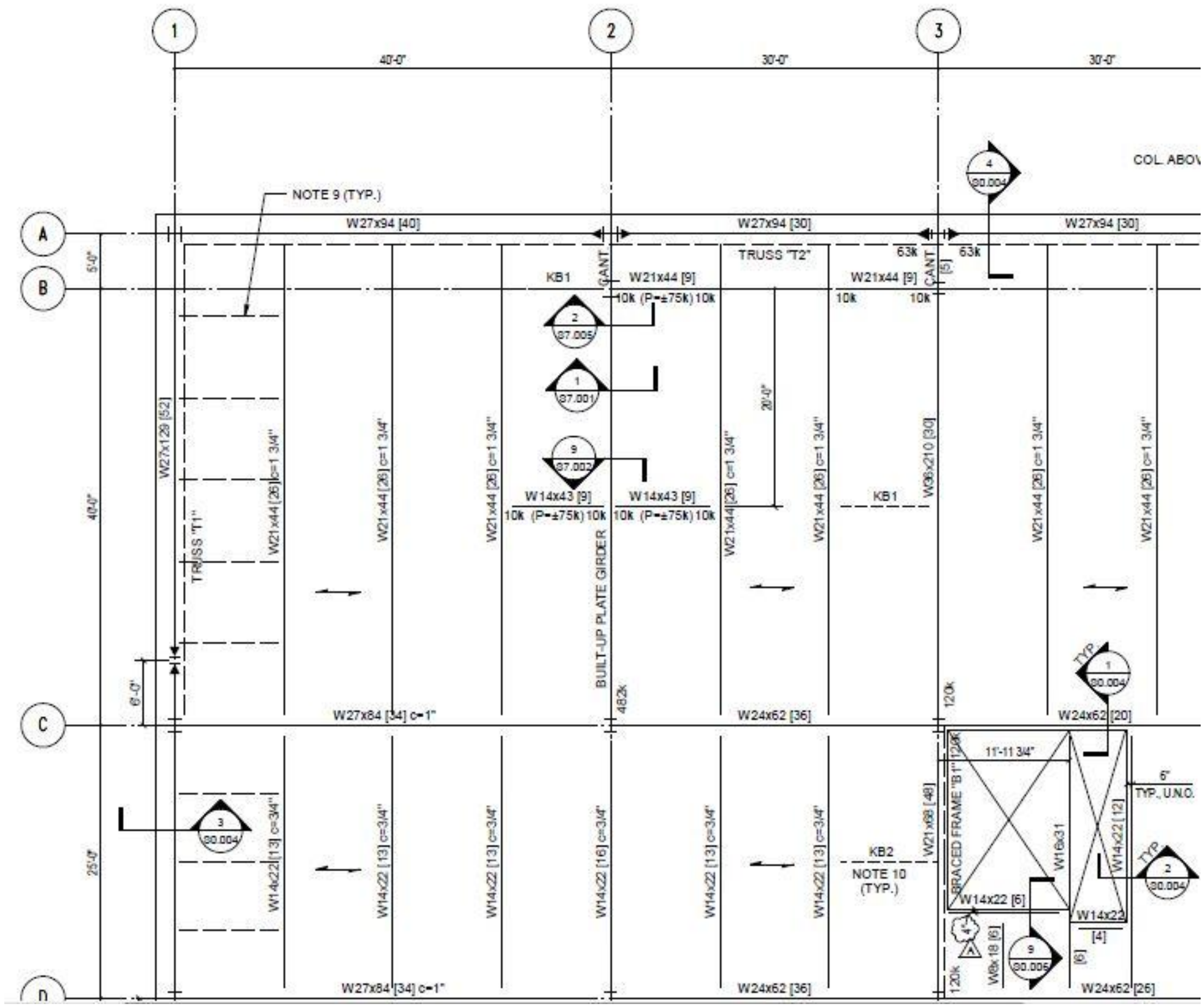


Figure 5: Partial Plan of Typical Bay Dimensions

Base plates are A36 steel and range in thickness from 1" to 2 3/4". Gravity column bases anchor into the foundation with four Grade 55 anchor rods with diameters of 3/4" to 1" and embedded a minimum of 1'. This connection type still allows the base plate to deform, so the connection is a pinned base.

Typical moment connections consist of a 3/8" minimum shear tab with 5/16" fillet weld to the beam flange and 3/4" diameter A325 slip critical bolts the full length of the shear tab. The flanges are field welded with a full penetration bevel weld with backing.

Area-Specific Characteristics

The floor system on Level 01 of the office structures has multiple sections. Where the office superstructure overlaps the parking structure, the floor is an overbuilt 4" thick, 3,000 psi semi-lightweight concrete slab reinforced with welded wire fabric 1" from the top of the slab. Where the superstructure does not overlap, the floor is a 25" deep pan joist system consisting of a 5" slab and 20" deep pans spaced a maximum of 6' center to center. Typical pan joists are 6" wide at the bottom and have bottom reinforcing ranging from #5 to #9 bars usually in a combination of sizes and top reinforcement sizes are #4 through #6 bars. Pan joists are supported on 25" deep post-tensioned or reinforced concrete beams. In the terrace area, the system changes to a one-way slab supported on concrete beams to support the extra dead load associated with the landscaping materials. A framing plan of the terrace area can be found in Appendix F.

Levels 02 through 05 have a composite floor system consisting of 3" 20 gage galvanized type 3.0SB composite steel deck with 3 1/2" 3,000 psi semi-lightweight concrete topping for a 6 ½" total thickness. Shear studs in all composite floors are specified to be installed in the strong position. The slab is reinforced with welded wire fabric and is unshored during construction. The deck has a maximum span of 11'-9" for a three span condition. Typical beam sizes for these levels include typical interior girders of W24x62, typical perimeter girders of W21x50, and typical infill beams of W21x44 and W14x22 with cambers of 3/4" to 1 3/4". Beams are spaced evenly between columns where possible.

On Level 06, the roof deck is 3" 20 gage Type N composite deck. Typical framing sizes include typical interior girders of W21x50, typical exterior girders of W21x57, and typical infill beams of W21x44 and W12x19 cambered 3/4" were needed. Penthouse framing sizes are typicallyW16x26 girders and infill beams of W16x31 and W12x19 with the addition of C12x20.7 members that support roof davits.

Lateral System

In the steel superstructure, the lateral system is composed of ordinary concentric steel braced frames shown in Figure 6. A floor plan showing the locations of the braced frames is in Appendix D. Typical column sizes for the brace frames are W12x152, W12x136 and W12x120 for the first three stories and decreases to W12x87 for stories four and five with the column splices occurring 4'-0" above Level 04. Beams sizes in the braced frames are W24x84, W24x76, W24x68, W24x55, W21x68, W18x46, W18x35, W14x22 and W16x26. Larger beam sizes are in the lower levels of the braced frames and decrease in size moving upward. Bracing members range from HSS 6x6 to HSS 10x10 with thicknesses of ½" or 5/8" where, again, the larger braces are in the lower levels and decrease moving upward.

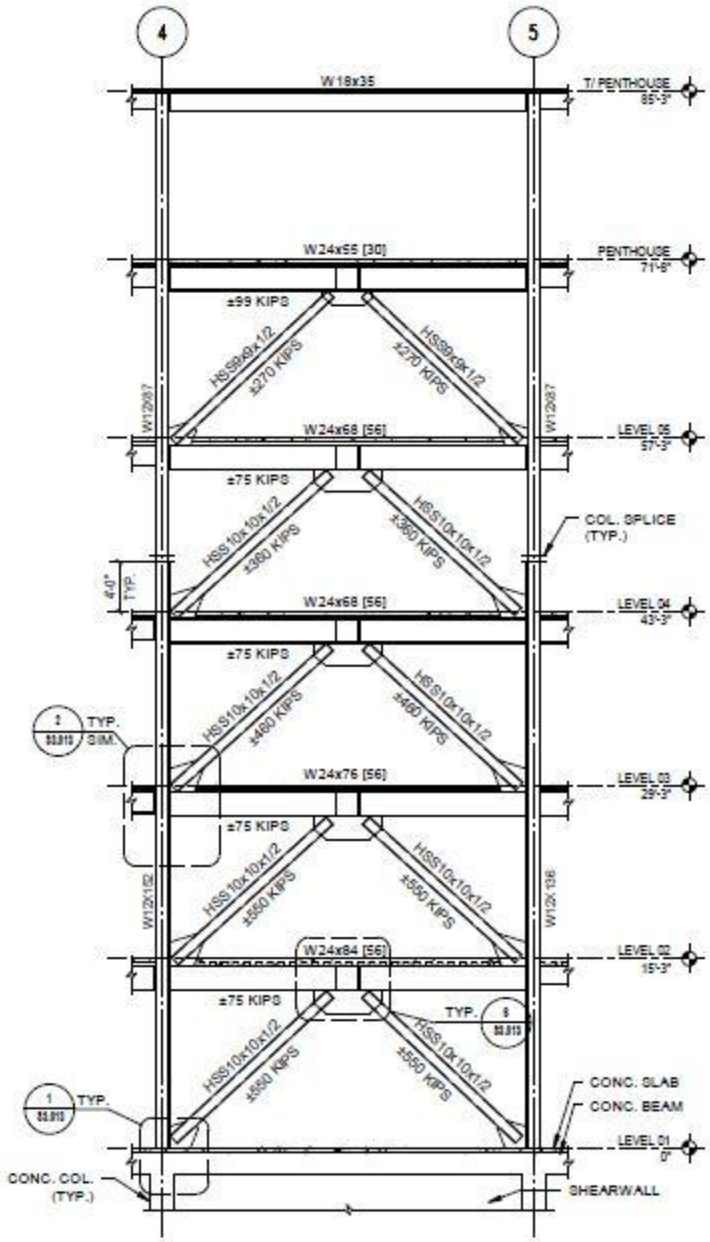


Figure 6: Typical Braced Frame Elevation with Penthouse Support Included

Additional floor diaphragm reinforcement is shown in Figure 7 below. The purpose for this additional reinforcement is to resist flexure the diaphragm, in plan, acts as a beam spanning between the supports of the braced frames. Reinforcement sizes for supplemental diaphragm reinforcement include #4, #5, and #6 bars.

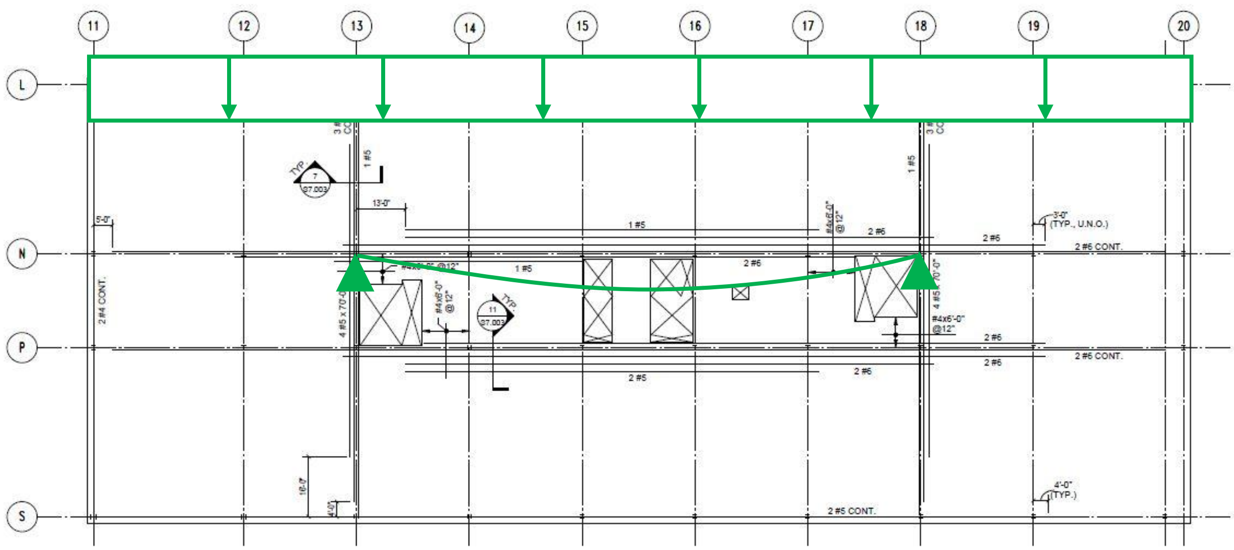


Figure 7: Floor Diaphragm Acting as a Beam Spanning Between Braced Frames

Moving down the building, the braced frames have a pinned base connection to the top of the shear walls. Brace members are welded to a gusset plate, which is welded to an embed plate. This plate, 3/4" thick, uses 3/4" diameter studs embedded into the concrete shear wall to transfer the horizontal forces from the braces into the shear wall. Column base plates are typically 3" thick made of A572 Grade 50 steel with 1 1/4" diameter, grade 105 anchor rods embedded 5' into the concrete column of the shear wall. The tensile and compressive loads are transferred into the shear wall through the base plate and anchor rods. Below in Figure 8 is a detail of this connection.

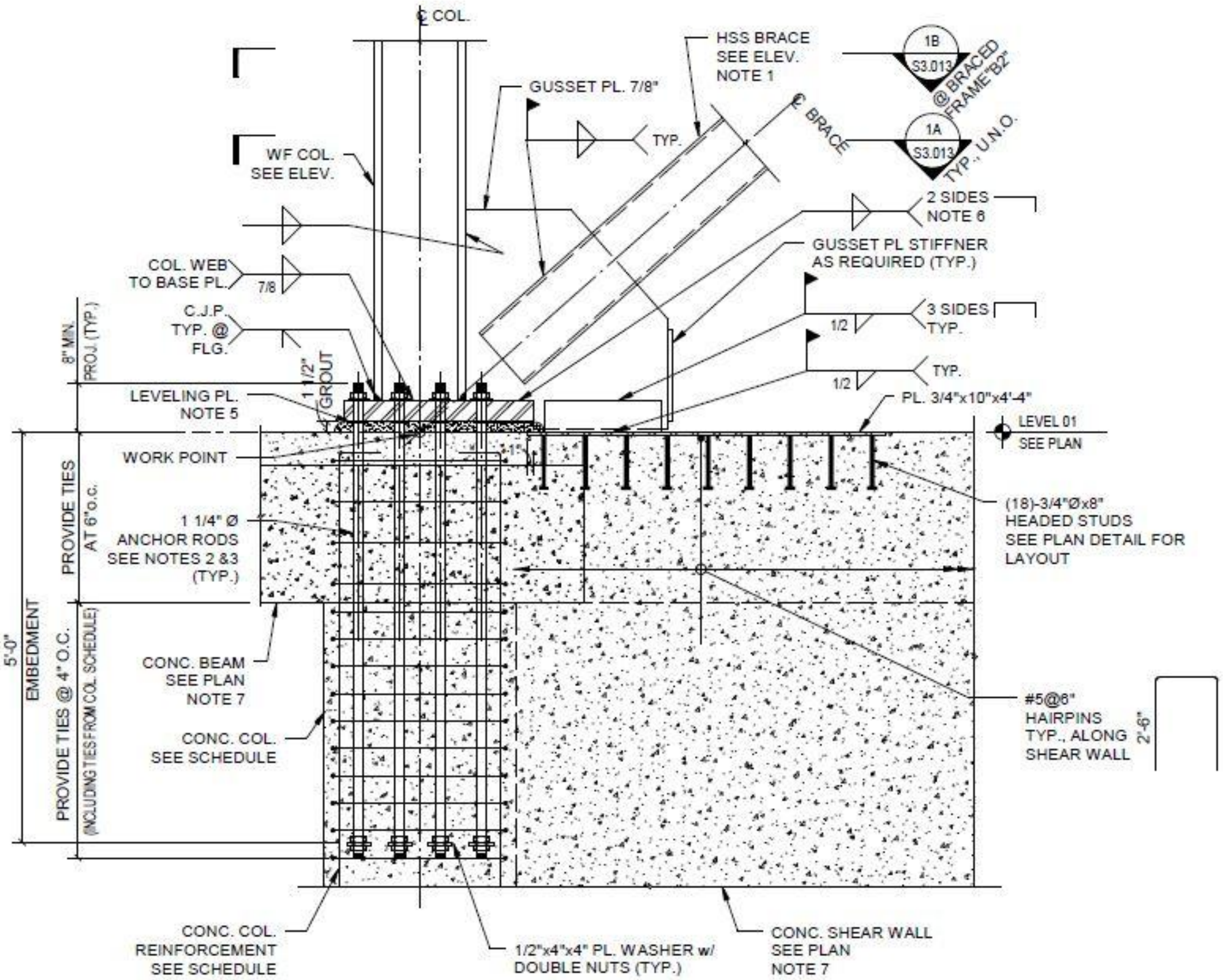


Figure 8: Typical Braced Frame to Shear Wall Connection

In the parking garage substructure, the braced frames are supported on 5,000 psi concrete shear walls. These shear walls are 16" thick with vertical reinforcement ranging from #6@12" o.c. to #10@9", 10", 12", or 13 o.c. bars and horizontal reinforcement of #5 bars at various spacing. Spacing varies based on floor levels and different walls. A sample plan of a shear wall is provided in Figure 9 below. These walls bear on grade beams which transfer the load to the foundation.

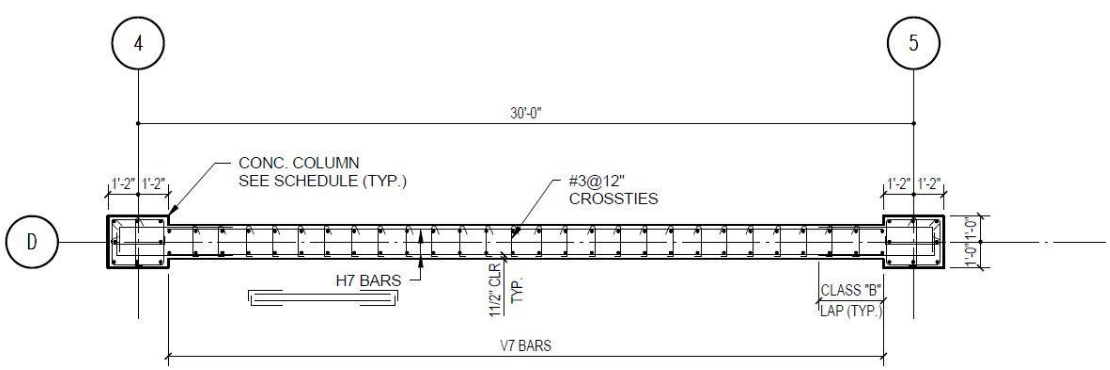


Figure 9: Shear Wall Sample Plan

Cantilever Truss System

Truss T2 is oriented along the longitudinal axis of the building. Two tension members in an inverted "V" and a vertical compression member are the main members of the system. T2 is supporting a 40' cantilever spanning from grid 1 to grid 2 in Figure 10 below. The most exterior tension member, running between grids 1 and 2, is designed for a tension load of 1544 kips and the back span diagonal, running from grids 2 to 4, is designed for a tension load of 1155 kips. Both tension diagonals are W14x176. The vertical compression member on grid 2, a W14x193, is designed for 2380 kips of compression load. These compression members on either side of the building bear on a built-up plate girder to be discussed later.

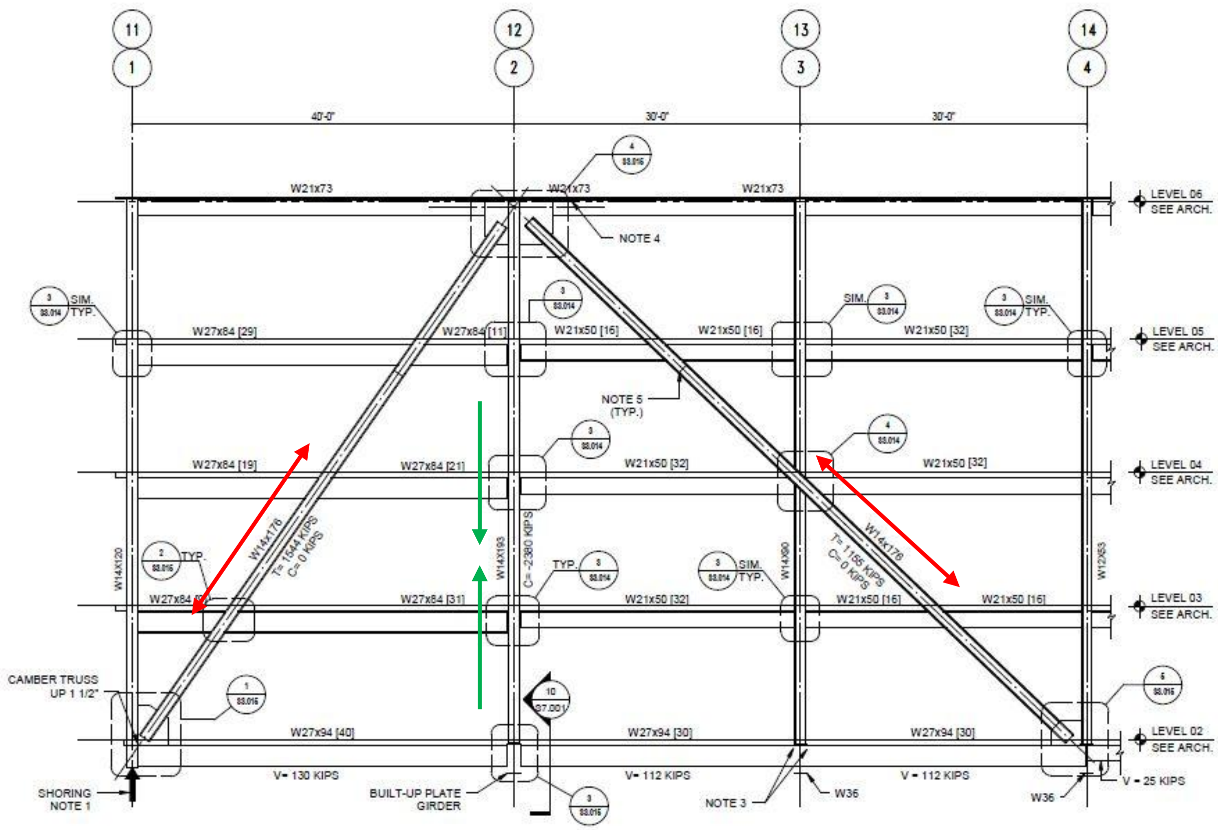


Figure 10: Truss T2 Elevation Highlighting Tensile and Compressive Forces

Truss T1, shown in elevation in Figure 11, is aligned in the transverse direction of the building consisting of W14x159 tension diagonals designed for a factored tension load of 891 kips. At the lower side of the tension members, the truss is cambered up 3/4" at Level 02 and grids N, P, C, and D. In terms of connections, the full moment splice has been offset from grid lines C and D to alleviate congestion at the column line and aid in constructability.

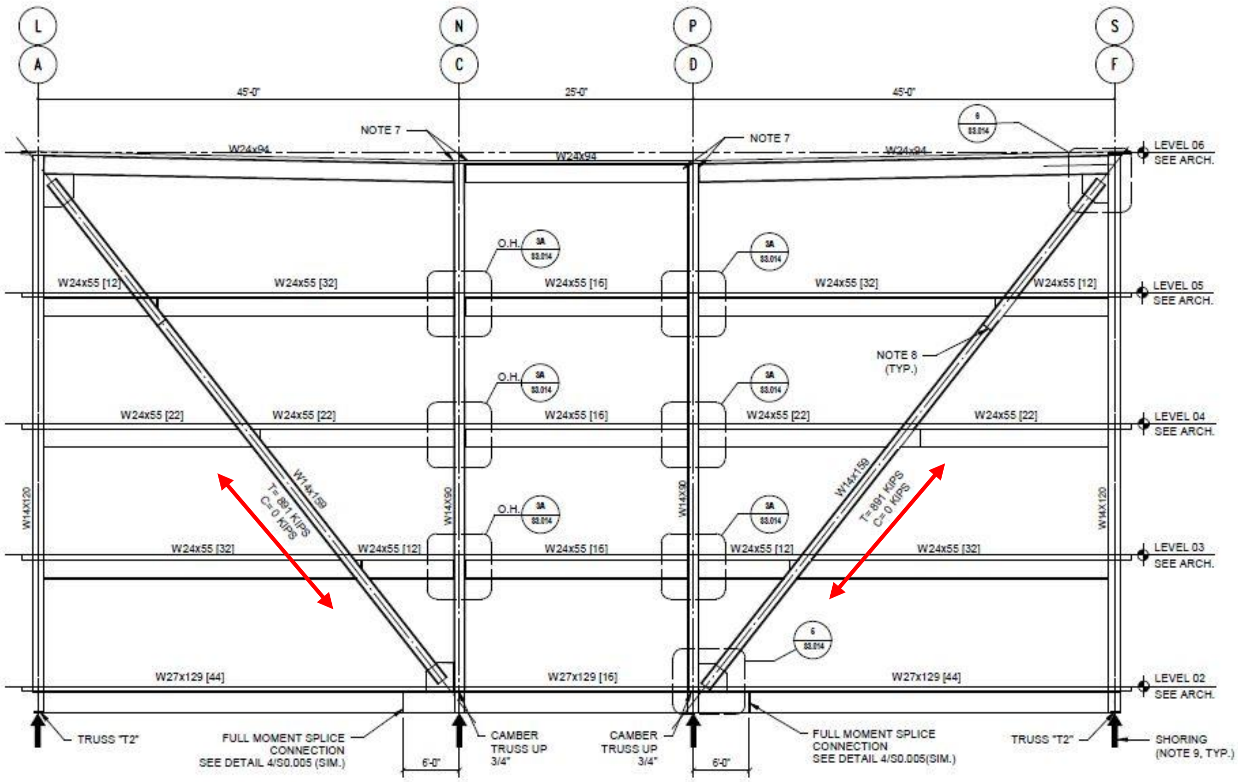


Figure 11: Truss T1 Elevation Highlighting Tensile Forces

To counteract the overturning of the cantilever, the beams on Level 06 are designed for axial tension starting where the exterior tension member of T2 meets the roof, circled in red in Figure 12 below. The truss overturning imposes axial tension loads on all beams going through the back span direction of the building, noted in red arrows in the diagram. The force decreases, or dissipates, as it moves away from the trusses. Under floor horizontal bracing, also designed for axial tension, starts where the exterior diagonal of truss T2 meets the roof which pulls the load toward the core and then follows the same horizontal path in plan through the building.

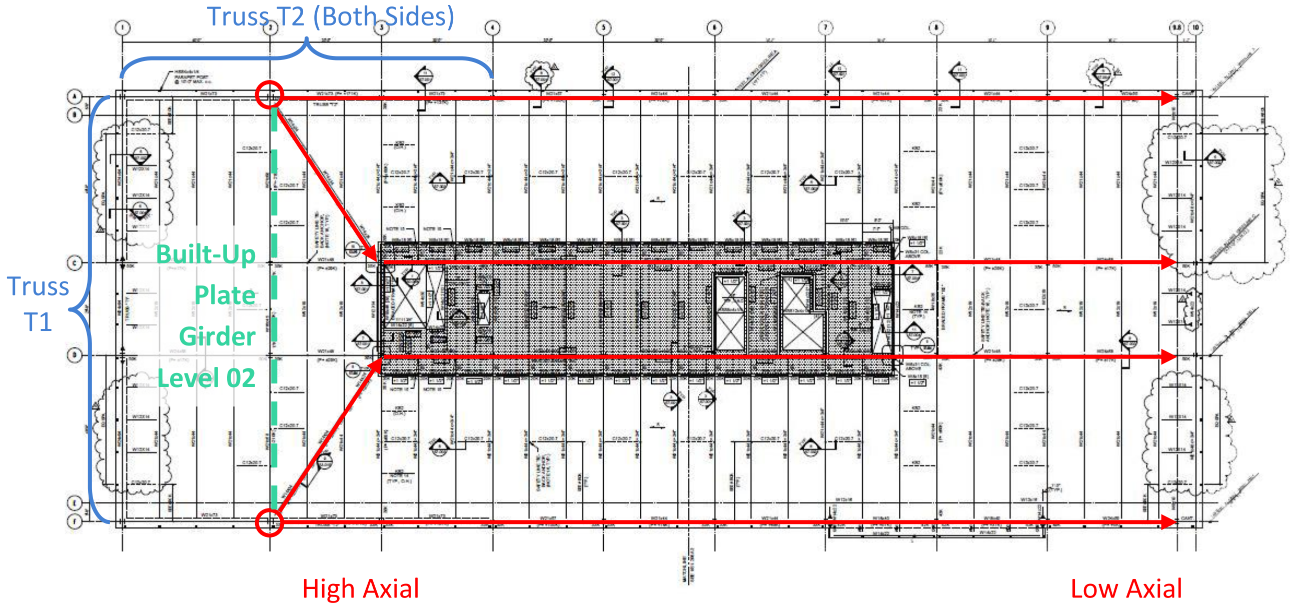


Figure 12: Roof Plan Showing Load Path of Truss System

At major connection points for both trusses, diagonal wide flange members are welded to 3/4" or 7/8" thick gusset plates. Where the truss diagonals intersect columns, the truss member stays continuous and the web is fitted with stiffeners that match the dimensions of the column it is splitting so that both members remain continuous through the connection. Columns and beams connect to girders stiffened with WT members cut to match the connecting column. Gravity beam connections inside these trusses consist of single angle, L4x4x3/8, shear tabs. At the outermost point of the cantilever, the truss system is cambered up 1 1/2" to counteract the deflection caused by dead load added after erection.

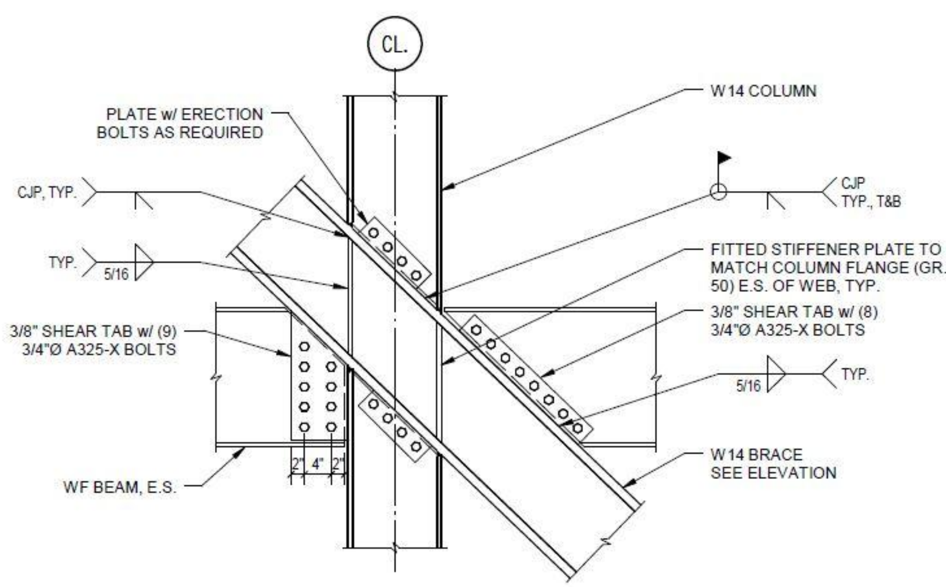


Figure 13: Truss Diagonal Joint Detail

As mentioned before, the compression members of truss T2 bear on the plate girder shown in Figure 14 below. The plate girder, A572 Grade 50 steel, is on Level 02 and spans over four columns on Level 01 which bear on post-tensioned beams in the substructure. Dimensions of the girder are shown in Figure 12 with the exception of 3/8" stiffener plates. It ties into the floor system by studs, angles, and stiffeners. Simple connections made to plate girder are typically seated connections where the bottom flange of the connecting beam has a 3/8" A572 gusset plate welded to the bottom flange. Kicker angles, typically 2L3 1/2x3 1/2x5/16, are welded to the gusset plate and the stiffeners in the plate girder to brace the girder's bottom flange against lateral-torsional buckling.

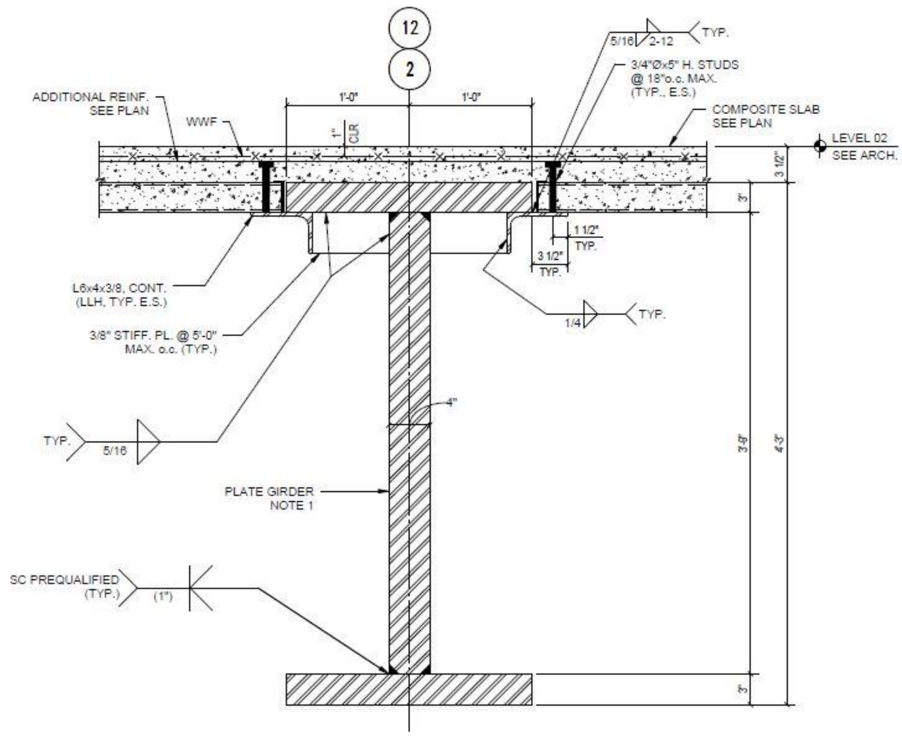


Figure 14: Plate Girder Detail

Loading

Although the load path is determined by the structure itself, design codes dictate safe loading criteria for the design of buildings.

Design Codes

Listed below are the design codes and reference standards used for the design of RGA Global Headquarters. Structurally, the chosen design method is Load and Resistance Factor Design (LRFD).

Building: International Building Code, IBC 2009 amended by Ordinance 24, 444-2010

State/County: St. Louis County Ordinances

Structural: American Society of Civil Engineers, ASCE 7-05
American Concrete Institute, ACI 318-08
American Institute of Steel Construction, AISC 360-05
Masonry: ACI 530/ASCE 5/TMS 402-08

Mechanical: International Mechanical Code, IMC 2009

Electrical: National Electrical Code, NEC 2008

Plumbing: Uniform Plumbing Code, UPC 2009

Energy: International Energy Conservation Code, IECC 2009

Design codes listed below are those used in thesis study if they differ from above:

American Concrete Institute, ACI 318-11
American Institute of Steel Construction, AISC 360-10

Load Paths

Depending on the type of load acting on the building, the building will respond to, or resolve, the forces differently.

Gravity Load Path

Gravity loads start in the roof or floor diaphragms which transfer the loads into the beams. Supporting the beams are the girders which frame into the columns. The load follows the column down the building and into the foundation. If the column line is interrupted, the load will follow the transfer girder supporting that last column into the columns supporting the transfer girder and then, into the foundation. Once the load reaches the drilled piers, it will be transferred into the bedrock.

Lateral Load Paths

Lateral loading from wind will first act upon the building's exterior skin. Connections of the curtain wall to the slab transfer the load into the floor diaphragm. From here, the load is transferred into the braced frames that take the load down to the concrete shear walls in the parking garage substructure. The shear walls are supported by grade beams that span between pier caps. The load is transferred through the pier caps, into the drilled piers, and into the bedrock.

Seismic forces on the other hand, will enter the system at the seismic base. Based on Dominic Kelly's article "Location of Base for Seismic Design" in *Structure Magazine*, the seismic base for this building is at the top of the footings. This is because the site is sloped, leaving a significant portion of the substructure exposed on the lower side of the site as seen below in Figure 15. In addition, the concrete shear walls have more mass than the braced frames above, impacting the distribution of seismic forces. The force distributes from the seismic base to the floor diaphragms on the basis of their masses. Once the force is in the floor diaphragm, it is transferred into the lateral system and down to the foundation similar to the load path for wind.

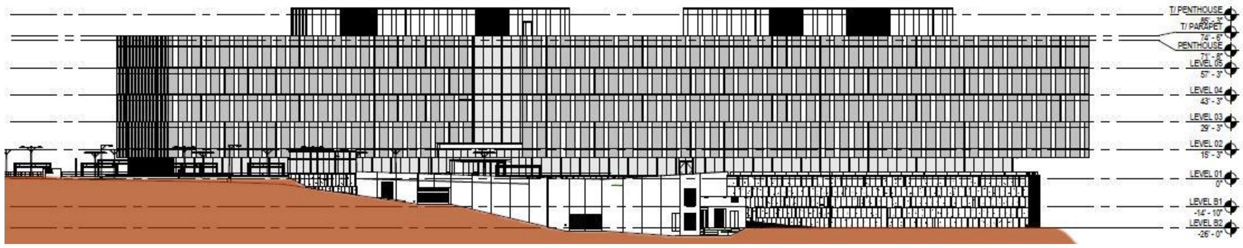


Figure 15: Architectural Elevation Emphasizing the Sloping Site

Soil Load Path

On the high grade side of the site, the parking garage walls are a tie-back retaining wall system shown below in Figure 16. Per the notes accompanying the design, the perimeter beams are designed to resist soil load only, demonstrated by the triangle load distribution. The perimeter beams are also separated by a joint to make sure load does not travel into the post-tensioned slab, which is spanning into and out of the page in the figure. In the case that there is a surcharge load, which is anything besides soil load represented by the rectangular load distribution, it will be picked up by the tie-back anchor and transferred into the residual soil. The tie-back anchor and retaining wall system are designed by a specialty firm.

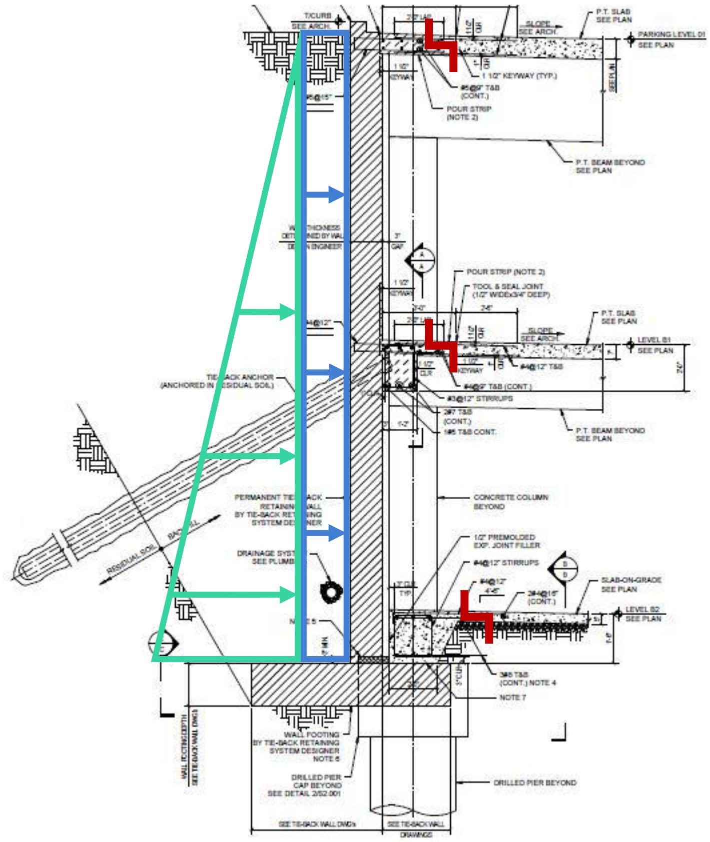


Figure 16: Tie-Back Retaining Wall

On the adjacent two sides of the parking garage, the retaining wall system switches to the foundation wall shown, in part, below in Figure 17 utilizing a temporary shoring wall for installation. The wall is spanning between floors with a fixed base provided by the rebar detail. The soil load acts on the wall and the exterior bars are put into tension while the opposite side is put into compression, creating a moment that resists rotation. Moving toward mid-height of the wall, the rebar in the interior side is larger to resist flexure from the soil. The wall is continuous over the upper parking level provided by the continuity in rebar and upper span of the wall. This joint as well as the top of the wall do not resist rotation and because of that are pinned supports.

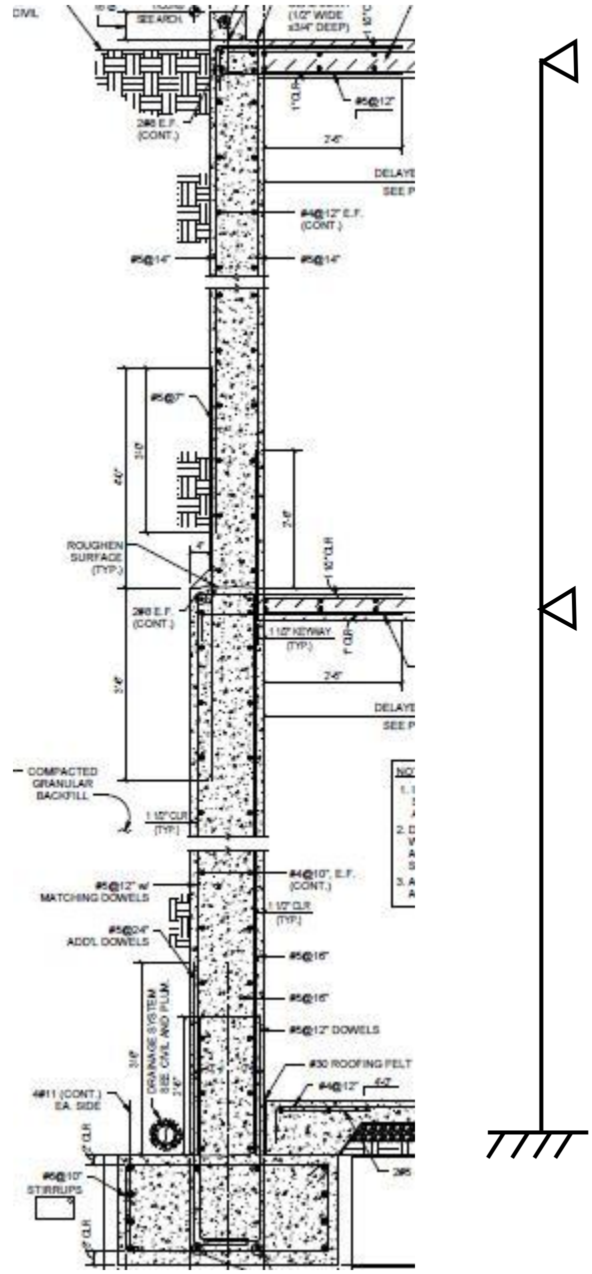


Figure 17: Retaining Wall Section with Rebar Detail (Left) and Idealized Support Condition (Right)

Load Overview

St. Louis County, MO adopted IBC 2009 into their building code which, in turn, adopts ASCE 7-05 for structural load determination. Live loads were determined using ASCE 7-05 Table 4-1. Notably, the office live load includes a 15 psf allowance for partitions. Additionally, the elevator machine room is designed for an assumed 150 psf live load similar to the storage and mechanical rooms designed for 125 psf assumed live load. On the other hand, dead loads were determined from material weights and industry standards for design minimums. Snow Loads were determined in accordance with ASCE 7-05 Chapter 7. Soil loads and seismic loads were presented in the geotechnical report provided by SCI Engineering, Inc. Soil loads are summarized in the following Table 1, by SCI Engineering, Inc.

Table 1: Soil Design Pressures from Geotechnical Report

Backfill Type	Equivalent Fluid Unit Weights	
	At-Rest Earth Pressures (pcf)	Active Earth Pressures (pcf)
Cohesive Soil	70	50
Granular Material (1-inch-minus)	60	40
Free-Draining Granular Material (1-inch-clean)	50	30

Finally, the following are seismic parameters provided by SCI Engineering, Inc.:

Seismic Design Category C F_v=1.65

Site Class C S_{DS}=0.39

F_a=1.2 S_{D1}=0.17

Summary

Through an existing conditions study of RGA Global Headquarters, the steel superstructure provides insight into braced frames and large scale truss design. The substructure features retaining wall systems, shear walls, and multiple slab systems that provide great educational value and warrant further study.

From the ground up, starting with the geotechnical recommendations, the foundation systems are primarily drilled concrete piers with some rock and soil bearing footings. Grade beams span between the pier caps and support two different types of retaining walls: a tie-back system and a vertically spanning retaining wall, as well as concrete shear walls and a slab on grade. Spanning between the substructure walls are post-tensioned concrete beams supporting one way post-tensioned concrete slabs or pan joist systems. Transitioning completely above grade, the steel superstructure is composed of composite floor systems, braced frame lateral systems, plate girders, and unique truss design.

The most influential structural feature that has been studied thus far is the cantilever truss system. The large tensile and compressive forces within the truss itself, as well as the force resolution in a series of axial forces throughout the structure, greatly impact framing decisions. In addition, the plate girder supporting the compressive loads has its own unique set of design considerations that influence how the truss loads navigate into the foundation. This system will present a significant challenge and will be very educational as well in future study of this project.

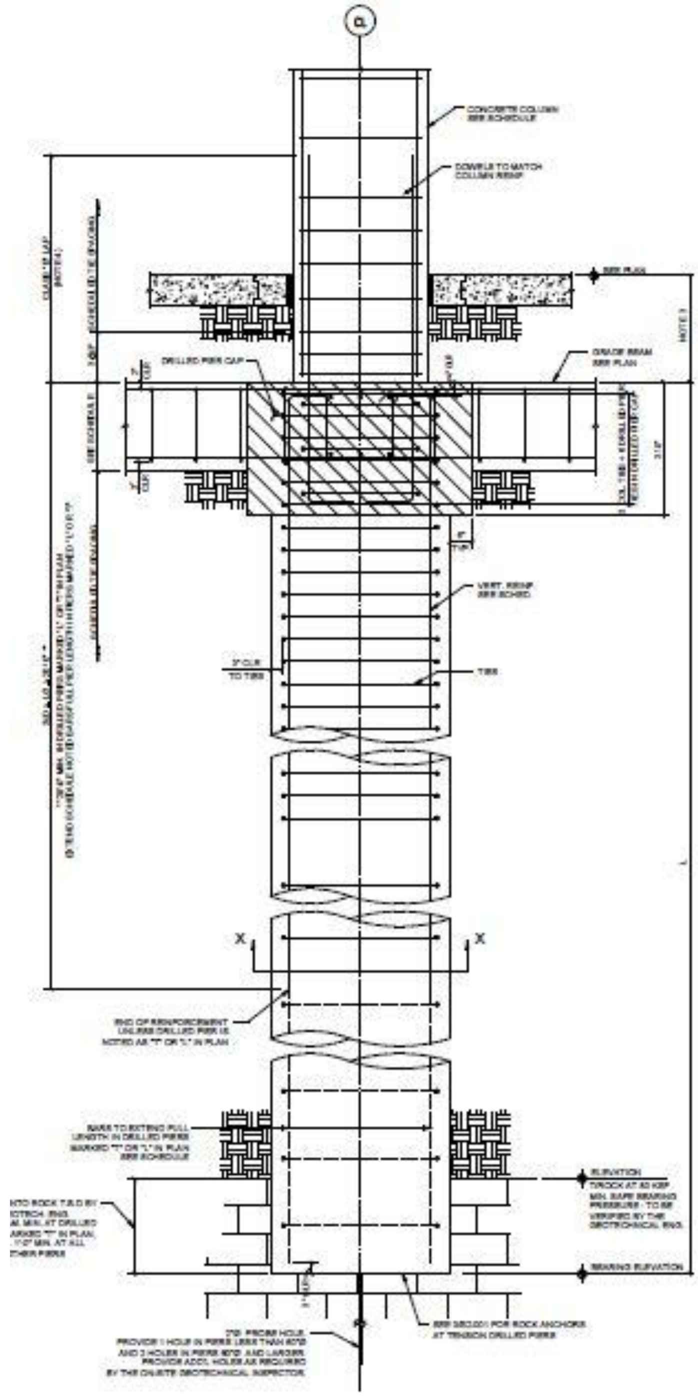
In summary, RGA Global Headquarters provides ample opportunity to study the interplay between multiple systems and structural materials as well as architectural decisions that greatly influence the design of the structural system.

Conclusions

Moving forward, several elements in the building will provide a challenge and have an impact on future analyses. One challenge is understanding and, looking toward the future, keeping the integrity of the connection between the braced frames of the superstructure and the shear walls of the substructure. Another challenge presents itself in resisting the soil loads in the parking garage because the post-tensioned slabs are not intended to take axial compression. The addition of surcharge loading from fire trucks, for example, only raises the stakes. Perhaps the most challenging, though, is the truss system supporting the 40' cantilever and the accompanying plate girder. The large amount of load and overturning action commands the framing layout of the superstructure and any modifications considered will be greatly impacted by this structural feature.

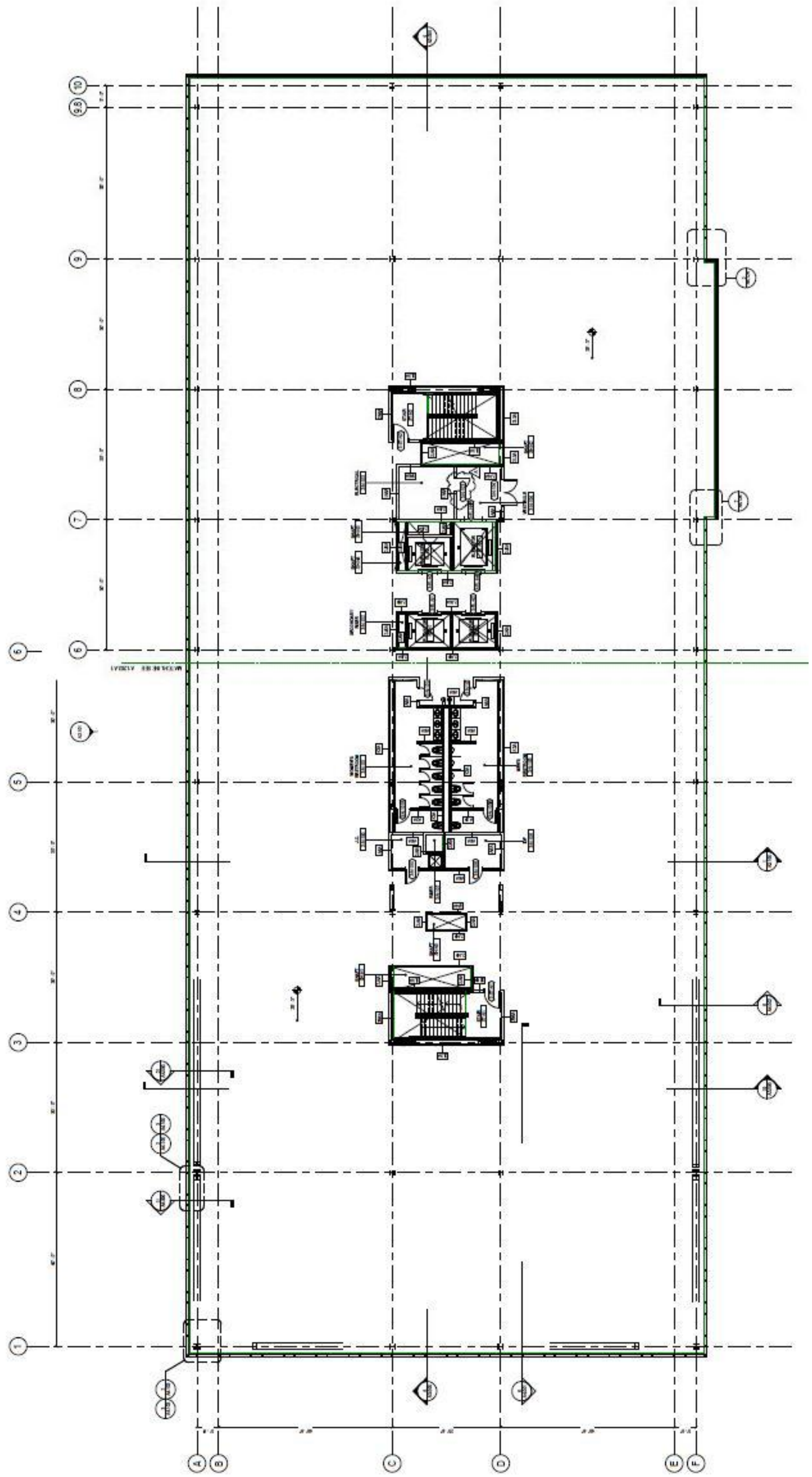
Appendix A

Drilled Pier Detail



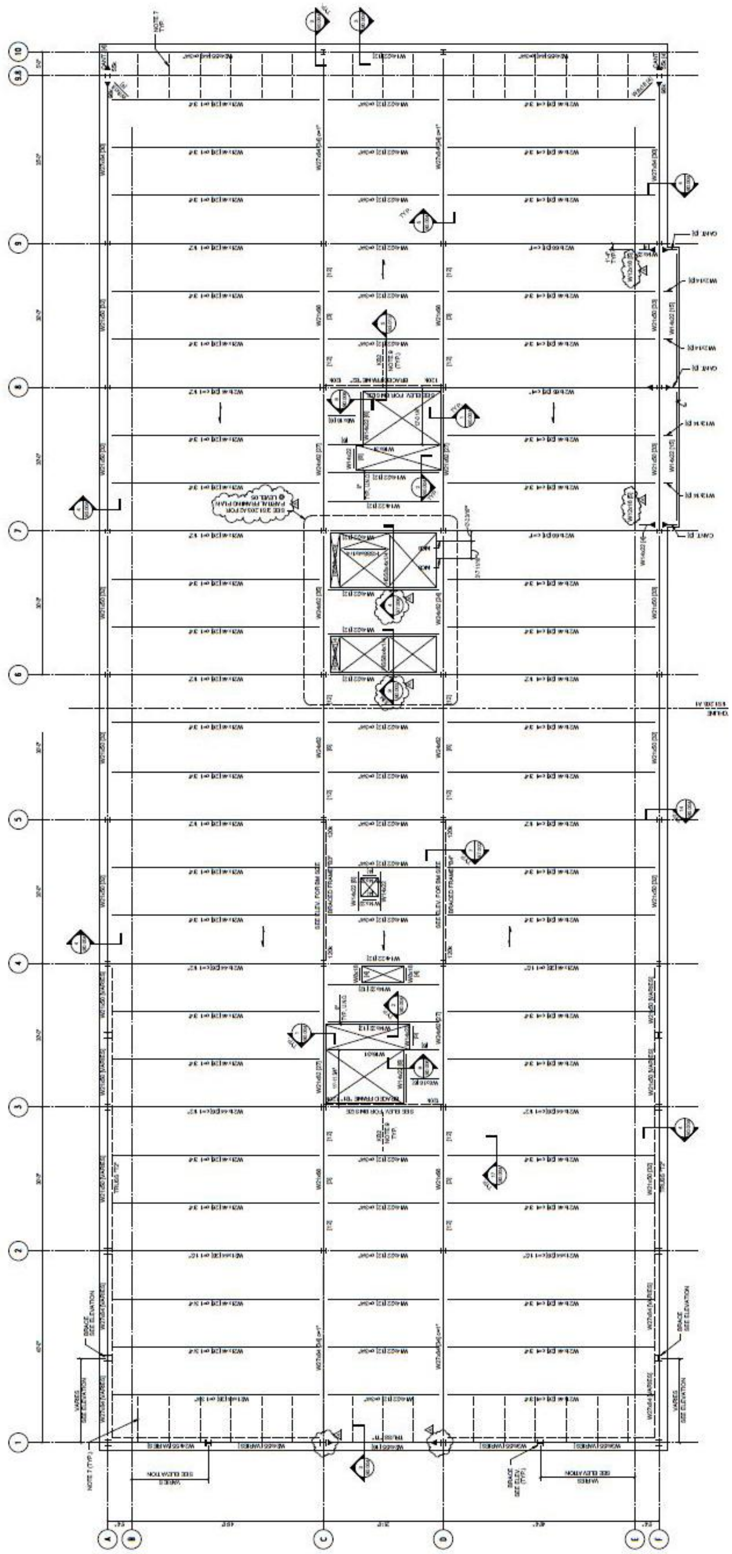
Appendix B

Architectural Typical Floor Plan



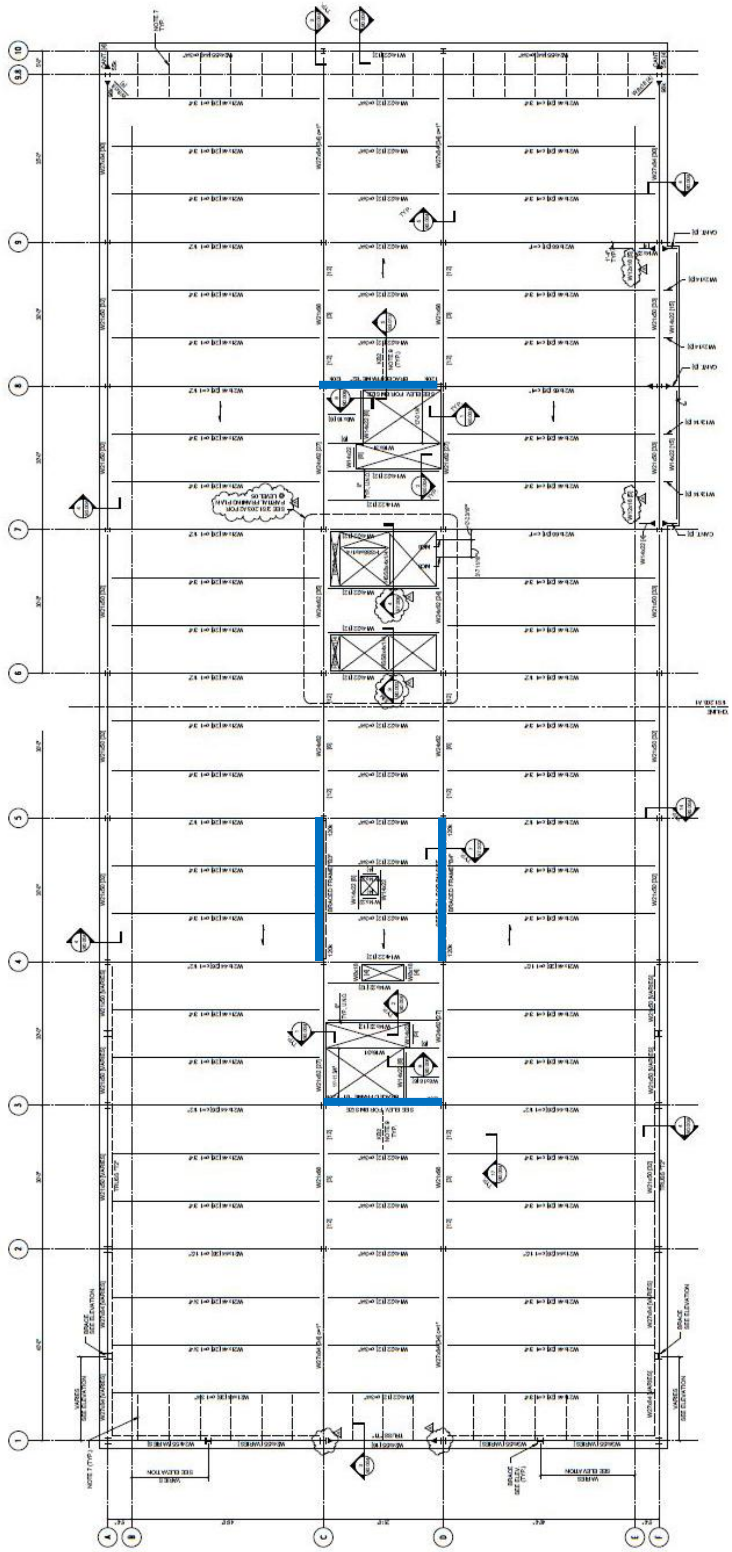
Appendix C

Structural Typical Floor Plan



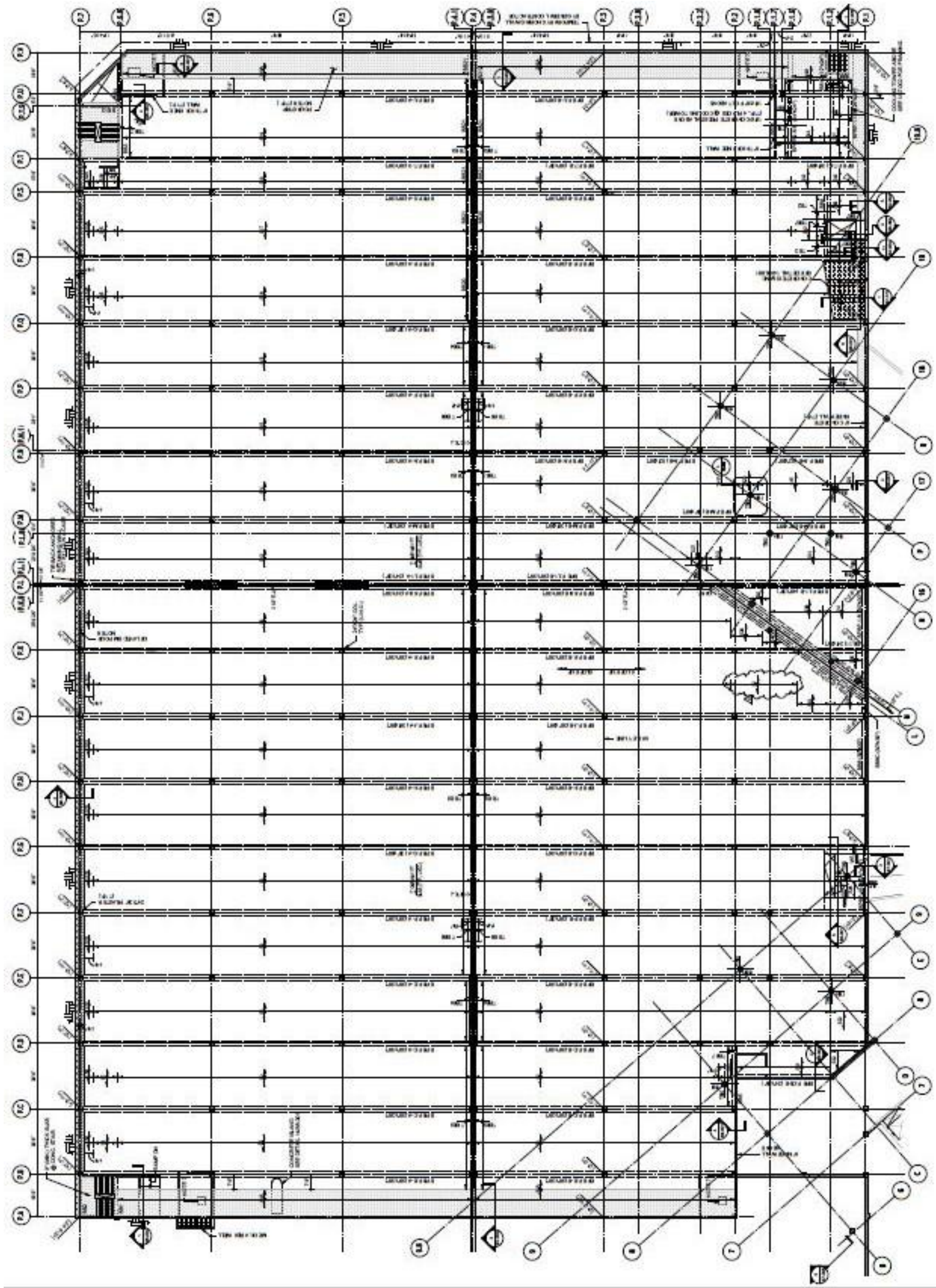
Appendix D

Braced Frame Location Plan



Appendix E

Substructure Framing Plan



Appendix F

Terrace Framing Plan