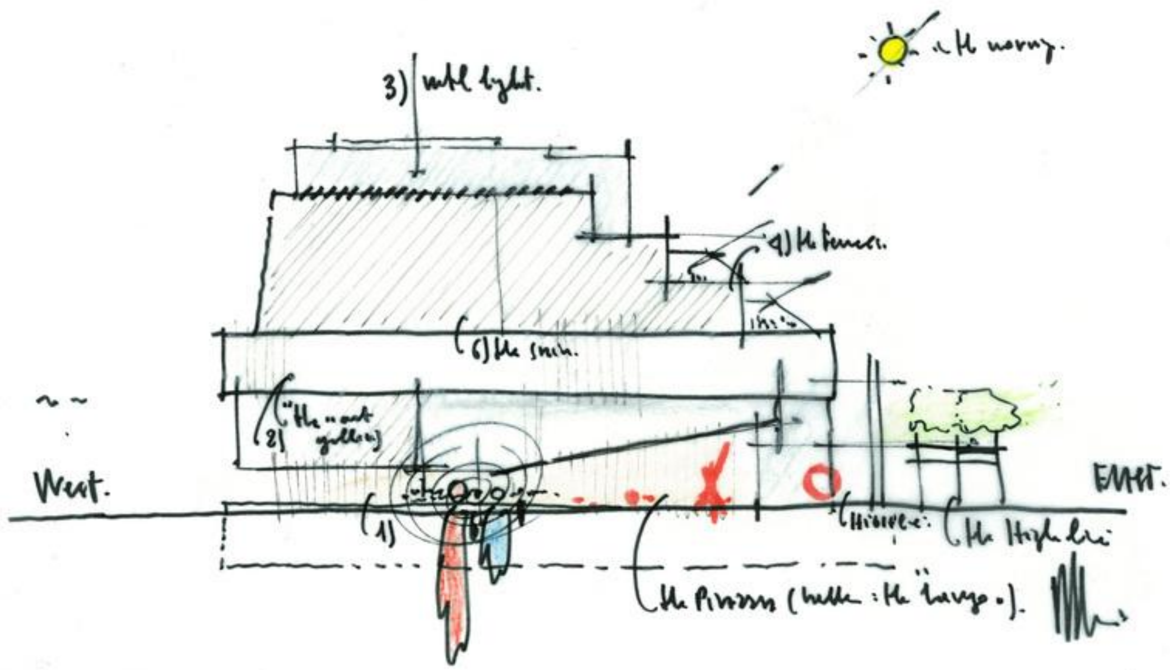
SEAN FELTON | STRUCTURAL



AMERICAN ART MUSEUM | NORTHEAST UNITED STATES

FINAL REPORT

ADVISOR: HEATHER SUSTERIC

April 3, 2013

SEAN FELTON | STRUCTURAL

AMERICAN ART MUSEUM | NORTHEAST, UNITED STATES

PROJECT TEAM

Owner Representative : Gardiner & Theobald
General Contractor : Turner

Engineers/Consultants

Civil : Philip Habib & Associates
Structural : Robert Silman Associates
MEP : Jaros, Baum & Bolles
Lighting : ARUP

Architects

Executive : Cooper, Robertson & Partners
Design : Renzo Piano Building Work Shop

GENERAL INFORMATION

Function : Museum/Mixed-Use
Size : 220,000 SF
Height : 150 ft
Number of Stories : 9 above, 2 below
Construction : 5/2011—12/2014
Cost : $266 million
Delivery Method : Design-Bid-Build (GMP)

N ELEVATION RENDERING

SE CORNER RENDERING

STRUCTURE

- Foundation consists of drilled caissons under pile caps, 36" concrete secant wall, and 24" pressure slab
- Composite floor system 3.25" concrete slab and W-shape beams
- Lateral system works with steel braced frames and specified rigid floors
- Floors supported by combination of columns, trusses, and hangers
- 30' cantilever at level 5 (SE corner)
- Levels 3 and 4 hung from level 5 in several places

MECHANICAL

- 5 architecturally exposed cooling towers
- Mechanical space in cellar, level 2, level 4, and level 9
- Combination of VAV for galleries and CAV for less controlled spaces
- Roof heating/snow melting roof system

LIGHTING/ELECTRICAL

- Lamps and windows specified for optimal color rendering (CRI > 97)
- LEDs, fluorescents
- (4) 4000 A 208Y/120V switchboards serve building

ARCHITECTURE

- Façade and interior module of 3'-4"
- Stainless steel and precast concrete clad-ding
- Exposed structural steel and MEP systems
- 50,000 SF of interior gallery space
- Step-backs for outdoor gallery space
- Skylight and architectural fabric used in level 8 gallery

CONSTRUCTION

- Secant wall poured in tandem with exca-vation
- 42" steel tubes used to stabilize secant wall during construction
- Deep wells gather site water before desedimentation, pumped back into sewer system

http://www.engr.psu.edu/ae/thesis/portfolios/2013/shf5014/index.html

ACKNOWLEDGEMENTS

I especially thank the following people and organizations for their assistance, provision, guidance and advice, support, prayers, patience, flexibility, humor, distractions, generosity, consideration, games of Dutch Blitz and Chess, and for otherwise picking up my slack over the past year.

Companies	The Owner (who wished to remain anonymous) Turner Construction Barton Malow Cooper, Robertson & Partners RPBW Robert Silman Associates
Advisors and Faculty	Professor Bob Holland Professor M. Kevin Parfitt Heather Sustersic Dr. Thomas Boothby Dr. Andres Lepage
Classmates	Sarah Bednarcik Dan Bodde Eric Buckwalter Jim Chavanic Tyler Donnell Jon Fischer Jon Gallis Adam Karlheim Nick Leonard Cheuk Tsang Andrew Voorhees
Family	Dad & Phyllis, Brandi Mom, Liz, Christina
Roommates	Bobby Frederico & James Wheeler
Friends	Carter Bowman Emily Draving Zach Jones Ian Kelmartin Kevin Moyer Abby Ott Meredith Tipton

EXECUTIVE SUMMARY

The Final Report investigates the possibility of supporting the South-Eastern corner of Renzo Piano's American Art Museum (AAM) without the use of a column at 3-M.5 (circled in Figure 1 below). Due to the monumental nature of the project, the structural alterations would need to be done in a way that minimized impacts on the architecture of the building. Though it was understood at the outset of this investigation that the weight and cost of the structural system would almost certainly increase, these effects were also to be minimized. After a thorough design and investigation of the proposed structural system and its effects on the architecture and construction of the building, this report recommends that the current structural design by Robert Silman Associates is the best solution to supporting AAM's signature cantilever.

A load path was successfully developed that did not involve a column at 3-M.5. This load path requires the use of additional trusses along the East wall of the Main Gallery space and South wall of the office spaces on Levels 3 and 4. Special consideration was taken to ensure that exposed structural steel in the gallery aligns with the carefully-developed modular façade system established by the Architect, and that the sizes of these trusses and their members did not affect the exterior envelope of AAM.

This concern for the architecture, however, adversely affects the weight and cost of the building. The proposed changes increase the weight of the influenced structural system by 50%, or nearly 100 t. Also, the foundations require greater capacity and 5 additional piles of varying strengths. In all, the cost of the structural system would increase by nearly $2 million, or 33%. Additional provisions isolated to individual members and custom cross sections will greatly increase the difficulty of the construction.

Note: All photos and renderings used with permission from RPBW.

Figure 1: Renderings of current (top-right) and proposed (bottom-right) structural systems.

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BUILDING INTRODUCTION

The American Art Museum (AAM) will serve as a replacement to the owner's current facility in the same city. Figure 2 shows AAM's new location in a more vibrant district of the city where aging warehouses, distribution centers, and food processing plants are being renovated and replaced by art galleries, shops, and offices. Now AAM stands in place of several such warehouses, and will provide a magnificent new southern boundary to the city's recently renovated elevated park, which terminates on the eastern edge of the site.

Figure 2: Arial map showing urban location along river (www.maps.google.com)

Renzo Piano's approach to AAM's design and architecture serves to reference the city's history with large cooling towers and outdoor terraces that step back towards the river on the west. These outdoor terraces will provide views into the city and space for outdoor exhibits and tall sculptures while being protected from any wind by the higher portions of the building's west side. Alternately, the large cantilevers, insets, large open spaces, exposed steel, and modular steel plate cladding show no attempt to camouflage AAM with the more historical surrounding buildings.

AAM's façade is comprised of the aforementioned stainless steel panels, pre-cast concrete, and glazing using a standard module of 3'-4" (about 1m; shown in Figure 3). The steel panels, the primary element of the façade, are 2 modules wide, or 6'-8". While most of the façade components are broken at each story, the longest panels stretch 60' on the southern wall from levels 2 to 6 and from 6 to 9.

This new facility is a multi-use building with gallery and administration space, two café/restaurants, art preservation and restoration, a library, and a 170-seat theater. Public space including the theater, classrooms, restaurants, and galleries are located on the south half of the building on the ground level and levels 5 through 8.

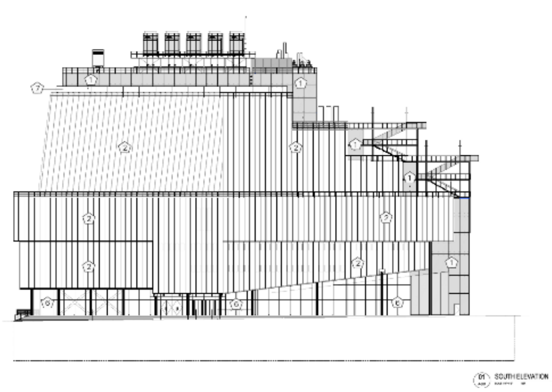


Figure 3: South Elevation showing modular façade (A-007)

Mechanical, storage, conservation, offices, and administration are dispersed on the north side at each level. The 220,000 square-foot AAM will stand 158' tall and has a guaranteed maximum price of approximately $267 million. Construction began in May 2011 and is expected to be complete in December 2014.

EXISTING STRUCTURAL SYSTEMS

OVERVIEW

AAM sits on driven steel piles filled with reinforced concrete with diameters of either 9.875" or 13.375" and grouped by pile caps. From the foundation level at 32' below grade, 10 levels rise on steel columns and trusses. Each floor is designed for steel/concrete composite bending. The lateral system consists primarily of braced frames spanning several stories. At some levels however, the floor system uses HSS diagonal bracing between joists and beams to create a rigid diaphragm that also transfers the lateral loads between staggered bracing. Moment frames are used for localized stability purposes. While masonry is used in AAM it is used for fire rating purposes only.

The building classifies as Occupancy Category III. This is consistent with descriptions of "buildings where more than 300 people congregate in one area" and "buildings with a capacity greater than 500 for adult education facilities."

FOUNDATIONS

URS Corporation published the geotechnical report in February 2011 to summarize the findings of several tests and studies performed between 2008 and 2010. They summarize that while much of the site is within the boundaries of original shoreline, a portion of the western side is situated on fill-in from construction. They explain further that the portion that was formerly river has a lower bedrock elevation and higher groundwater. Due to the presence of organic soils and deep bedrock, URS suggested designing a deep foundation system and provided lateral response tests of 13.375" diameter piles reinforced with 3"-diameter bars and socketed into bedrock.

The engineers acted on the above suggestions and others. The piles are specified with a 13.375" diameter of varying concrete fill and reinforcement to provide different strengths to remain consistent with URS Corp's lateral response tests. Low-capacity piles (9.875" diameter) are individually embedded to the pressure slab, while typical and high-capacity caissons are placed in pile caps consisting of one or two caissons. The high-capacity caissons are always found in pairs and are located beneath areas of high live load or where cantilevers are supported. For a complete layout and caisson schedule, see FO-100 in Appendix A.

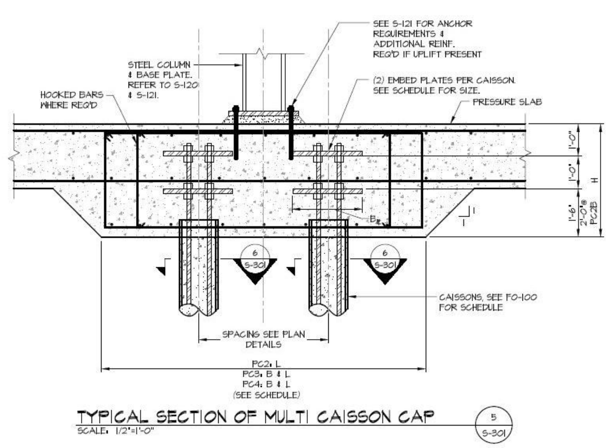


Figure 4: Pile cap section (S-301)

A pressure slab and the perimeter secant-pile walls operate in tandem to hold back the soil and groundwater below grade during construction and for the lifespan of the building. The walls vary between 24" and 36" and are set on 6'-6" wall footers and caissons. These are isolated from the pressure slab shown in Figure 4. Hydrostatic uplift led the engineers to design a 24" pressure slab, isolated from the 5" architectural slab-on-grade by a 19" layer of gravel.

Gravity System

Floor System

A surprisingly regular floor layout contrasts the obscure geometry of the building (Figure 5). The engineers managed to create a grid with spacings of roughly 20' (E-W) and 30' (N-S), where the 20' sections are divided by joists which support the floor decking running E-W. Beams that do not align with the typical perpendicular grid indicate a change of building geometry below or above. Each joist and beam/girder is designed for composite bending with the floor slab.

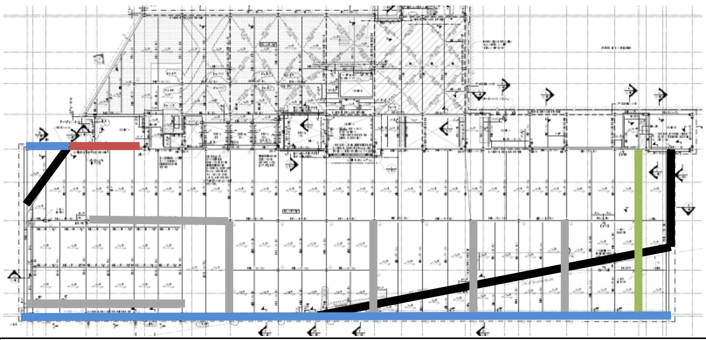


Figure 5: Level 5 framing plan showing regular layout against building footprint (S-105)
Gravity Trusses (above)
Gravity Trusses (below)
Plate Girder (d=46")
Lateral Braced Frames (part of gravity)
Outline of Building Below

Four slab/decking thicknesses are called for depending on deck span and loading, all on 3"-18 gauge composite metal deck. The most common callout is 6.25 (total thickness) lightweight concrete. This provides a 2-hour fire rating. 7.5N (normal weight) is used on level 1 for outdoor assembly spaces and the loading dock, and 9N is used for the theater floor. The roof above the level 9 mechanical space calls out 5.5.

While the layout can be considered relatively consistent, the beam sizes and spans selected suggest a much more complicated floor system. Though a typical span at 20'-30', spans often run as long as 70' on the gallery floors (levels 6-8). The shorter spans require joists as small as W14x26, but the longer spans supporting the upper gallery levels require beams as large as W40x297s for web openings. In several places welded plate girders are specified at depths from 32.5" to 72." The plate girders are used as transfer large loads and moments over cantilevers, especially from gravity trusses and lateral braced frames (Figure 6).

Framing System

Cantilevers on the south side of AAM are supported by 1 or 2-story trusses, typically running in the N-S direction. One large gravity truss runs along the southernmost column line between levels 5 and 6 to support the cantilever on the south-eastern corner of the building.

While the vast majority of columns are W12x or W14x shapes, some of the architecturally exposed steel vertical members are HSS shapes, pipes, or solid bars. Furthermore, the gravity load path goes up vertically and horizontally nearly as much as it flows directly down a column to the foundation. Figure 7 shows how large portions of the southern half of AAM's levels 3 and 4 are hung from trusses and beams on the level 5 framing system.

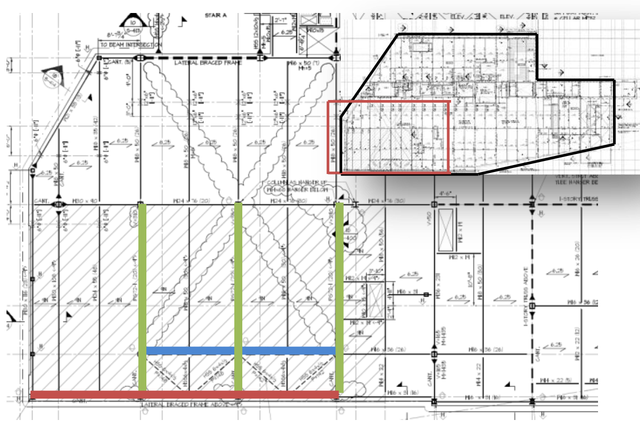


Figure 6: Level 3 framing plan showing transfer girders and lateral braced frames (S-103)
Lateral Braced Frame (above)
Lateral Braced Frame (below)
Plate Girder (d=72")

Renzo Piano's designs often expose structural steel, providing an extra constraint on the design team. One example is Column 3-M.5 which supports level 5 from the outdoor plaza below. The foundation column below grade specifies a W14x311, a typical shape for a column, but the architecturally exposed structural steel is called out as 22" diameter solid bar. A unique analysis would be required for a solid bar acting as a column, as AISC XIII does not have provisions for such a selection in its tables or specifications. Strength calculations for the optional 22" Round HSS are discussed in the Proposed Structural Design section of the Final Report.

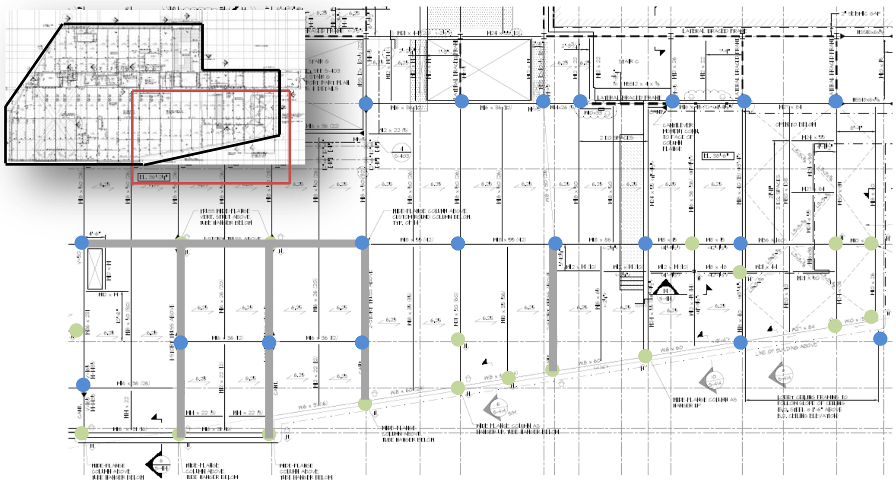


Figure 7: Level 3 framing plan showing hangers and outline of hung/cantilevered portion of building (S-103)

Gravity Truss (above)

Compression Support (single below)

Tension Support (single above)

Lateral System

AAM's lateral system is more easily understood than its gravity systems. The concentric braced frames stagger up the building, transferring lateral loads via diagonal bracing within the floor diaphragms on level 3 for the southern portion and 5 for the northern portion as shown in Figure 8. Most of the braced frames terminate at ground level, but three extend all the way down to the lowest level. The bracing members are comprised mostly of W10x, 12x, or 14x shapes in X-braces or diagonals. There are, however, HSS shapes are used with chevron-braces. An enlarged floor framing plan showing the braced frames at level 5 is provided in Figure 9 below.

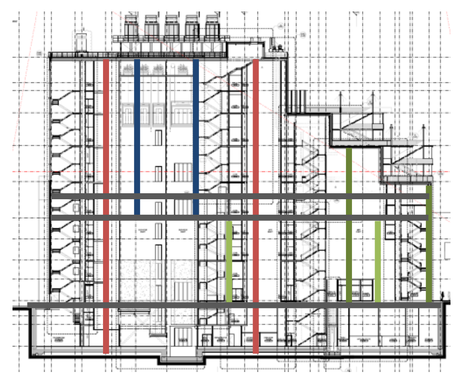


Figure 8: Section cut showing N-S braced frames at staggered heights (A-212)

Figure 9: Level 5 Framing Plan Showing Lateral System (S-105)

Lateral Braced Frame

Gravity Truss that Contributes to Lateral System

Floor System with Diagonal Bracing

DESIGN CODES & STANDARDS

The design codes listed for compliance of structural design can be inferred from drawing S-200.01 and Specification Section 014100.2.B:

- International Code Council, 2007 edition with local amendments including:
    - Building Code
    - Fire Code
- ASCE 7-05: Minimum Design Loads for Buildings and other Structures
- ACI 318 -08: Building Code Requirements for Structural Concrete (LRFD)
- AISC XIII: Specifications for Structural Steel Buildings (LRFD)
- AWS D1.1: American Welding Society Code for Welding in Building Construction

Other codes not applicable to the structural systems of the building can be found in the specifications.

MATERIALS SPECIFICATIONS

The different materials specifications are summarized in Figure 10 below. Additional information can be found on drawing S-200.0, provided1 in Appendix A.

Concrete & Reinforcement			Structural Steel			
Wt	Use	f'c (psi)	Shape	ASTM	Gr.	Fy (ksi)
LW	Floor Slabs (typ)	4000	Wide Flange	A992	-	50
NW	Foundations (walls, slab, pile caps, grade beams)	5000	Hollow Structural	A500	B	46
			Structural Pipe	A500	B	46
NW	Composite Column Alternate	8000	Channels	A36	-	36
NW	Other	5000	Angles	A36	-	36
Gr.	Use	ASTM	Plates	A36	-	36
70	Reinforcement	A185	Plates (for Girders)	A709	50	50
150	Reinforcement In Composite Members	-	Connection Bolts	A325-SC	-	80
70	Welded Wire Fabric	A185	(3/4") Anchor Bolts	F1554	36	36

Figure 10: Material specifications

DESIGN LOADS SUMMARY

GRAVITY LOADS

LIVE LOADS

Perhaps the most notable aspect of AAM's design is its live loads. Typically, one would expect to see Live Loads calculated from ASCE 7-05 minimums (ASCE 7-05 Table 4-1). The structural narrative explains that much of AAM does not fit with any ASCE 7-05 descriptions of use types, so the engineers have provided their own design loads summarized in Figure 11. Additionally the engineers created a live load plan on S-200.01 which shows areas of equal live load on each floor.

The engineers, in a desire for maximum flexibility of the gallery spaces, elected to drastically over-design the AAM-specific spaces for live loads, while being consistent with ASCE 7-05 minimums for more common areas.

Design Narrative Summary		ASCE 7 Designation	
Use	Live Load	Live Load	Description
Gallery - Typical	100	100	Assembly Area - Typical
Gallery - Level 5	200	100	Assembly Area - Typical
Testing Platform	200	150	Stage Floors
Offices	50	50	Offices
Private Assembly/Museum Use	60	n/a	n/a
Auditorium - Movable Seating	100	100	Theater - Moveable Seats
Compact Storage	300	250	Storage Warehouse - Heavy
Art Handling & Storage	150	125	Storage Warehouse - Light
Outdoor Plaza and Loading Dock	600	250	Vehicular Driveways
Stairs and Corridors	100	100	Stairs and Exit Ways
Lobby and Dining	100	100	Assembly Area - Lobby
Mech Spaces Levels 2, 9	150	n/a	n/a
Mech Spaces Cellar	200	n/a	n/a
Roof - Typical	22 + S	20	Roof - Flat
Roof - Above Gallery	122 + S	n/a	n/a

Figure 11: Comparison between Design LL and ASCE 7 Minimum LL

DEAD LOADS

Because the live loads are so high, special care seems to have been taken by the design engineers to be very precise in their dead load calculations. Similar to the live loads, the diversity of different use types and load requirements have led to a congruent variety of dead load arrangements in structural steel weight, concrete density, MEP requirements, partitions, pavers, roofing, and other finishes. A total of 37 different dead load requirements, arranged by use and location, are listed in the Dead Load Schedule on drawing S-200.01. These range from 76 PSF to 214 PSF. In all, the building has a dead weight of 23,084 k (11,500 tons) from level 1 through level 9 Roof North. Complete dead load calculations for the building are in Appendix B.

SNOW LOADS

Snow loads were calculated using the procedure outlined in ASCE 7-05. Figure 12 details the summary of this procedure, comparing the Snow Load Parameters on drawing S-200.01 to the City Building Code/ASCE 7-05.

ASCE 7-05 equation 7-1 (section 7.3) states that where the ground snow load exceeds 20 PSF, the flat roof load value must not be less than (20)I_s. 22 PSF, the design flat roof load, is not in accordance with ASCE 7's minimum according to equation 7-1 of 23 PSF. It is important to note that the step-back terraces where drifting is a concern are designed for 100-200 PSF of live load, and it is unlikely that the building will experience snow loads exceeding those live loads. Complete Calculations can be found in Appendix B.

Design Parameters		ASCE 7-05
Pg	25	25
Ct	1	1
Is	1.15	1.15
Ce	1	1
Pf	20.1	20.1
20 Is	22	23

Figure 12: Snow loads comparison

LATERAL LOADS

OVERVIEW

It was not possible to replicate the wind or seismic loads used to design AAM. With greater resources and experience, the engineers used Wind Tunnel Testing and Modal Response Spectrum Analysis as permitted under ASCE 7-05 for wind and seismic loads respectively. These processes allowed the engineers to accurately assess the lateral loading conditions using the correct geometry.

The Final Report does include an investigation of the wind and seismic loads as prescribed by ASCE 7-05. For simplification purposes, only levels 6 (elev. 88' 2") through RN (elev. 169' 10") were considered in this investigation. A series of additional simplifying assumptions allowed for an analysis using ASCE 7-05 chapter 6 for wind and chapters 11 and 12 for seismic. Although the designers determined that seismic loads controlled both base shear and overturning moment in their analyses, The N-S wind case controls base shear and seismic controls overturning in ASCE 7-05 using the Analytical Procedure for wind and Equivalent Lateral Force Procedure for seismic.

WIND LOADS

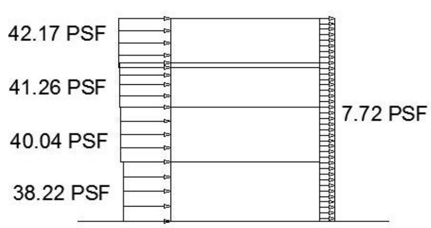
As mentioned above, the wind loads in both directions were found using Analytical Procedure (Method 2) in ASCE 7-05 chapter 6 using assumptions that simplify the geometry and environment of the building. Using the factors in Figure 14 below (calculations in Appendix B), the wind pressures were calculated between 45 PSF and 55 PSF (Figure 15). The design professionals explained that Wind Tunnel Testing returned values of between 30 PSF and 45 PSF, making the Analytical Procedure about 12PSF conservative (a difference of about 20% - 25%).

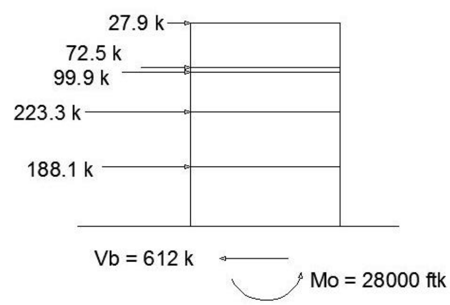
Figure 15 below summarize the revised wind load calculations. The base shears and overturning moments were found for both the North-South (Y) and East-West (X) directions by creating equivalent lateral forces at each story level. More detailed calculations provided in Appendix B show that AAM must resist wind across a much greater surface area in the N-S direction than the E-W. This difference leads to the much greater base shear (1300k which controls) and overturning moment in the N-S direction.

Wind Factors		
	E - W	N - S
G_f =	0.89	0.85
GC pi =	0.55	-
Cp =	-0.3	-0.5
Kd =	0.85	-
Kzt =	1.0	-
I =	1.15	-

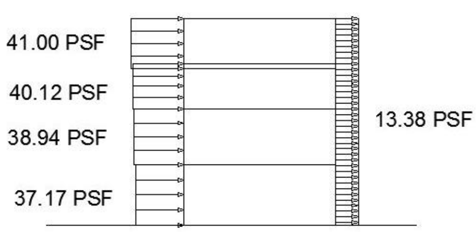
Figure 14 (Left): Wind factors for ASCE 7-05 calculations

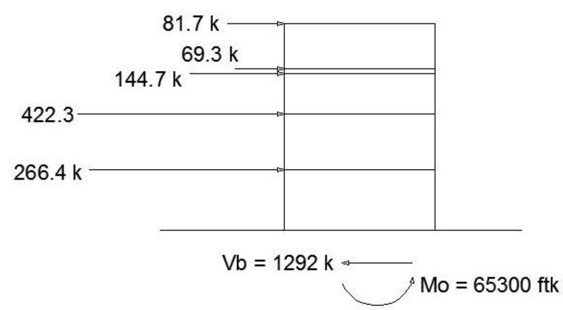
Figure 15 (Below): ASCE 7-05 Wind Pressures and equivalent lateral forces





East – West Direction





North –South Direction

Seismic Loads

The seismic loads in the Final Report were calculated using the Equivalent Lateral Force Procedure found in ASCE 7-05 chapters 11 and 12. As mentioned above, this method is in contrast to the structural engineer's Modal Response Spectrum Analysis, which is considered to have a higher degree of accuracy (ELF is more conservative). The investigation performed for the Final Report, however, uses the assumptions provided on drawing S-200.01. Figure 16 shows which values were provided by the engineers and which were supplements needed to complete the ASCE 7-05 analysis.

These values were used alongside the revised dead load calculations to find the equivalent lateral forces, base shear, and overturning moment summarized in Figure 17 below. Further calculations can be found in Appendix B. The revised base shear was found to be 1276k for floors 6-RN, much higher than the provided base shear of 946 for the whole building, which can be explained by the different procedures. The overturning moment of 158,500 ft-k controls for both wind and seismic analysis.

Seismic Design Criteria			
S-200.01		ASCE 7-05	
Sds	0.65	Ta (s)	0.9
Sd1	0.13	Cu	1.7
I	1.25	T (s)	1.53
R	3	TL (s)	6
W (k)	5849		
Cs	0.0602		

Figure 16: Seismic Design Criteria

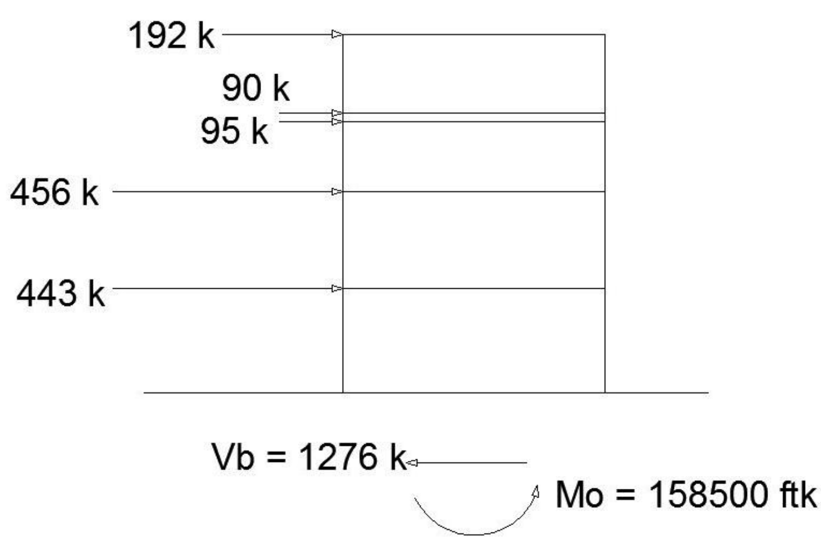


Figure 17: Equivalent Lateral Force Procedure Summary

PROBLEM STATEMENT

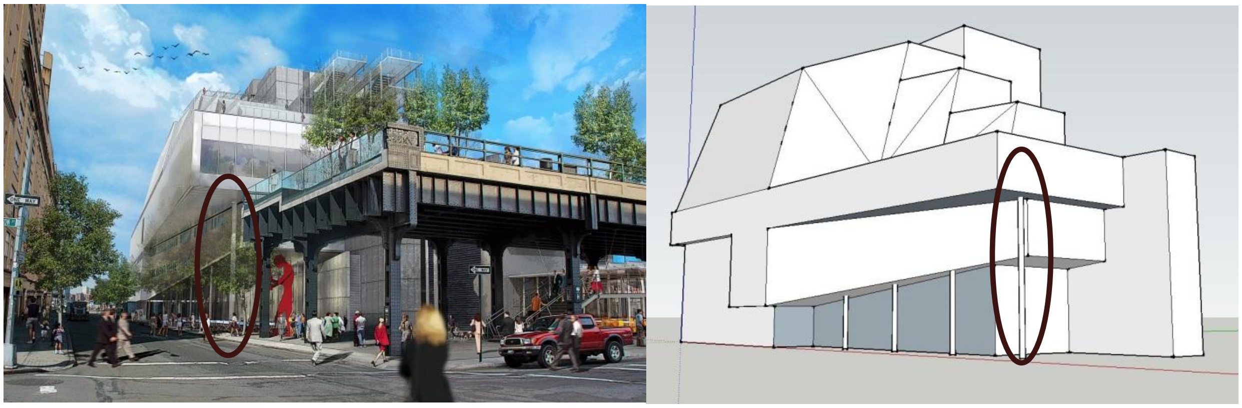


Figure 18: Rendering and Sketchup model showing column 3-M.5 from SE corner

Figure 18 above shows the geometry of AAM at the SE corner entrance and plaza space. Four architecturally exposed columns in the space run parallel to the street and coincide with the horizontal grid of AAM. Three of these columns support the mass of levels 3 and 4 above the glass-enclosed lobby. The fourth column (3-M.5, circled), however, appears to be the sole support of level 5.

A scenario has arisen in which the architect has expressed interest in removing Column 3-M.5. Architecturally, this 22" circular column carries the most delicately-balanced and most massive part of the building visible from street level. Though current design represents an effective and elegant solution to the stability of the cantilever, the architect has asked the structural engineer to consider a method which does not include the use of a column at the location of 3-M.5.

PROBLEM SOLUTION

It is for the above reasons that this thesis project will explore the possibility of supporting the level 5 cantilever without the use of a column at the location of 3-M.5. Extensive changes must be made to the building's gravity load path in ways which minimize effects on the cost, construction schedule, and architectural themes already in place.

A new load path must be introduced to redistribute the 1,800 kips carried by Column 3-M.5. This new load path will require changes to the framing of the levels below and at the cantilever level. First, a two-story truss will have to be added along the south wall (non-orthogonal) on levels 3 and 4 to act as the last support at the cantilever in both directions. Secondly, a truss must be added between levels 5 and 6 at the eastern gallery wall (currently glass). Loads will then travel through the existing frame (where possible), which will be re-analyzed to accommodate the extra loads resisted by each member.

This alternative design will be compared to the current design by analyzing changes to cost, weight, schedule, and impacts on the architecture. Finally, the data will be reviewed by the architect and owner for consideration.

PROPOSED STRUCTURAL DESIGN

LOAD PATH OVERVIEW

Before any technical design could be completed, a load path had to be established. The selection of the proposed load path follows the existing load path as closely as possible in an effort to avoid significant impacts on the architecture in place. Figure 19 shows both the existing and redesigned load paths in plan and perspective.

Floors 5 and 6 are supported by Truss 0.9 on the southernmost edge of AAM. Truss 0.9 is then simply supported, spanning between a strengthened truss at column line H and a new truss at column line N.2. Limitations discussed below in the Final Truss Design section resulted in the design of a cantilever system for Truss N.2 where Truss 0.9 is supported 26' from the nearest support at column line X. A new column was added at the location 6-N.2 to resist uplift. In order to support Truss N.2 at column line X, an additional new truss was designed along the existing exterior face. Truss X was similarly designed as a cantilevered truss supported at column lines L and J. The compression support at L is 42' from its load point due to Truss N.2, and the uplift support utilizes an existing truss at J. Finally the existing Truss J was redesigned to resist that uplift, and existing Truss L was replaced with a column at the location 3-L.

It is important to note that the cantilever supported by 3-M.5 extends 24' beyond its last support and the proposed cantilever extends 46' to its last support at 3-L. Also, for the purposes of this investigation, this alteration to the gravity system has been designed to be entirely independent of the lateral system, and therefore does not impact the rigidity of the structure or any component of the lateral system.

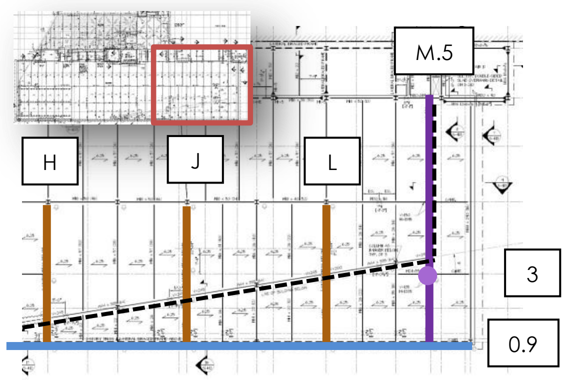
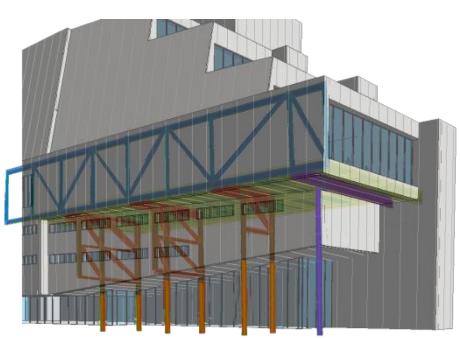
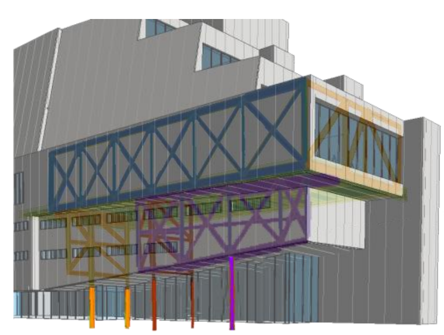
Existing Design (see also Figure 5)	Proposed Redesign
	
	

Figure 19: Plan and Perspective Comparison of Existing(left) and Proposed (right) Load Paths (S-105)

CUSTOM CROSS SECTIONS

EXISTING CUSTOM SECTIONS

As discussed in the Existing Structural Systems section above, AAM's design engineers developed 10 custom shapes to accommodate the large forces and moments created by the cantilevers. Using the information provided on drawing S-201, the strength of each shape was calculated according to AISC XIV and ACI 318-11 in an effort to utilize these custom members designed by the engineers. Furthermore, the designs provided set precedence and guidelines for the development of additional custom members where existing designs are inadequate. The complete calculations for these design strengths are provided in Appendix C.

BUILT-UP PLATE GIRDERS

Though the plate girders provided are used primarily to resist large moments (see Figures 5, 6 and 19, Existing Design), an initial investigation was performed to find both the moment and axial strengths of the plate girders based on a 20' un-braced length. This information was intended for use as a starting point should the large forces in the proposed systems require such capacities. A summary is provided in Figure 20 below.

	Moment Capacity				Axial Capacity			
Member	ϕMnx (ft-k)	Lp (in)	Lp (ft)	Limit State	KL/r	KL/r lim	ϕPn (k)	Limit State
32.5	12197	473	39	Yielding	19.4	113	8395	Torsion
33-1	12518	479	40	Yielding	19.2	113	9446	Torsion
44-1	20520	609	51	Yielding	14.7	113	9532	Torsion
46-1	12555	648	54	Yielding	SL	SL	SL	SL
46-2	29550	657	55	Yielding	13.7	113	16775	Torsion
46-3	22170	631	53	Yielding	14.1	113	9724	Torsion
72-1	45090	815	68	Yielding	10.7	113	10174	Torsion

Figure 20: Plate Girder Moment and Compression Strengths Assuming 20' Un-braced Lengths

Because each member was found to be compact for flexure, moment capacities are based on plastic section moduli which include both the flanges and web of each member. Plate girder shape PG72-1 has the highest moment resisting capacity of over 45,000 ft-k and a maximum un-braced length of nearly 70'. For compression, however, the web of shape PG46-2 proved to be slender, so it is not considered an option as a component of the proposed truss systems. Shape PG46-2 has the highest compressive strength of 16,775k failing in torsional buckling.

COMPOSITE HSS ROUND COLUMNS

In contrast to the plate girder shapes, the three HSS Round columns function are primarily designed for axial loads. Provisions specified in AISC XIV chapter I2.2 and I3.4 on composite members were used to calculate the compressive, tensile, and flexural capacities of each member, summarized in Figure 21 below. Similarly to the plate girder sections, the strengths provided in Figure 21 are used as a reference if the proposed redesign should require such strengths.

Provisions for composite sections also exist in ACI 318-11 chapter 10. While slenderness checks performed under this specification verified the non-slenderness of the sections, it was decided that the provisions in AISC XIV chapter I should govern the strength design of these members. AISC XIV Equation I2-9b is used to calculate the compressive strength of steel sections without slender elements filled with concrete. The equation uses the material properties of the concrete and reinforcement without regard for any critical loads, making the reduced stiffness provisions that could be required for slender sections in ACI 318-11 irrelevant to this strength investigation.

Shape	Lu	ϕMn	ϕPn	ϕTn
15A	25	750	2421	2295
15B	25	624	2161	1685
22	45	1714	4389	3545

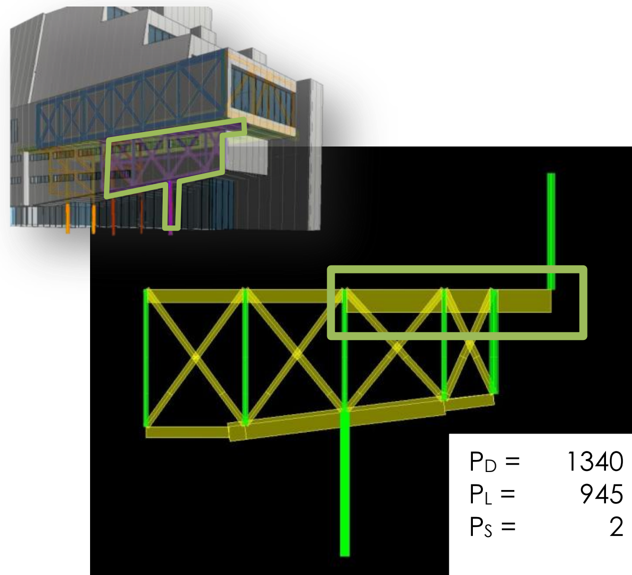
Figure 21: HSS Round Column Capacities

Proposed Custom Sections

Although the custom sections provided by the engineers are sufficient for the existing design, the design proposed in the Final Report render all existing cross sections inadequate for the largest required loads. In two locations new custom sections were developed to provide adequate strength for the proposed structural system. Complete design calculations for both proposed custom sections can be found in Appendix C.

PG56-1

Proposed plate girder PG56-1 is designed to transfer loads between Truss N.2 and Truss X (shown in Figure 22). This cross section was developed because of architectural constraints (discussed in the Architecture Considerations section below) which do not allow Truss X to extend past gridline M.5, and limit the depth of the cross section to 56". This depth constraint led to a departure from the component plate dimensions made precedent by the engineers. The web thickness and the flange dimensions were increased to provide additional capacity for combined loading conditions when used in Truss X. Final design dimensions and capacities for PG56-1 are provided in Figure 23 Below.



P_D = 1340 k
P_L = 945 k
P_S = 2 k

M_U = 34,331 ft-k
V_U = 3121 k

Figure 22: Loads on PG56-1

PG56-1 has the largest web thickness, the widest flange width, and the largest flange thickness of any established or proposed plate-girder cross sections. Though PG56-1 is designed adequately for the loads, this departure from precedent component plate dimensions could lead to adverse effects during fabrication and construction. These effects are explored further in the Construction Management Considerations section of the Final Report.

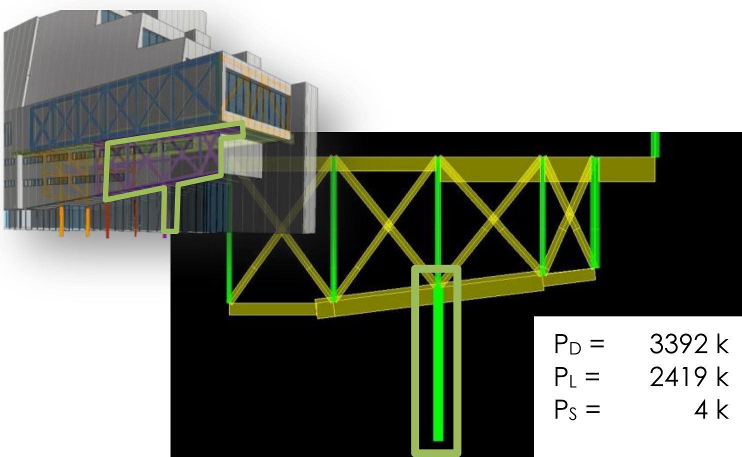
	Existing Shapes		PG56-1		Capacities		
Lb	n/a		20	ft	ϕMn	41571	ft-k
D	n/a		56	in	ϕVn	3402	k
B	18, 20	in	24	in	ϕTn	25245	k
tf	2, 4, 8	in	10	in	ϕPn	27541	k
tw	1, 2	in	2.25	in			

Figure 23: PG56-1 Design Summary

24R-1

Proposed custom section 24R-1 is designed for column location 3-L (shown in Figure 24), which is the last support for the cantilever in the proposed structural system. Because the loads applied to this column under the proposed system are so high, the current custom round column shapes are inadequate (see Figure 21 above). The proposed section is designed as a composite column using the same conditions and assumptions as the current sections described above.

Figure 24 also summarizes the design dimensions, properties, and capacities of shape 24R-1. In order to acquire sufficient axial strength, the precedent outer diameter of 22" was abandoned for 24", the wall thickness was increased from 1-1/4" to 1-3/4", the concrete strength was increased from 5,000 psi to 15,000 psi, and 14 no. 11 rebars were added for a total of 16. The yield strength of 150 ksi for the reinforcement in the composite columns is not altered from the current design, and can be found on the Custom Round Column Schedule on drawing S-120.01. Though compressive capacity was paramount to the design of 24R-1, the sizing of elements (such as the wall thickness) of the section were developed for fabrication and constructability. These considerations are discussed in the Construction Management Considerations section of the Final Report.



P_D = 3392 k
P_L = 2419 k
P_S = 4 k

P_U = 7943 k

Pipe		
Do	24	in
t	1.75	in
Concrete		
f'c	15000	psi
fy	150	ksi
no.	11	
n	16	
Capacity		
ϕPn	8272	k
ϕTn	8053	k
ϕMn	2754	ft-k

Figure 24: Loads and Capacities of 24R-1

TRUSS DESIGN AND ANALYSIS

OVERVIEW

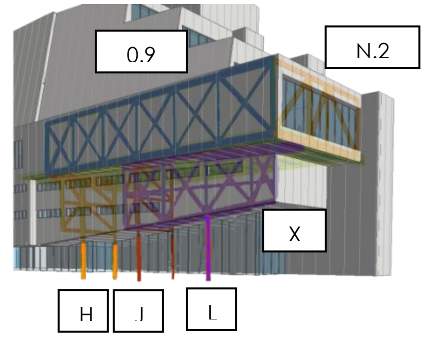


Figure 25: Truss Name Summary

The proposed truss system was designed primarily using an itterative process in ETABS. Due to the complex nature of this structural system, initial sizes were selected based first on precedence for Truss 0.9, and later judgment as the design progressed down the load path. An analytical method for selecting initial sizes was performed for a variation of Truss X used to verify ETABS's truss action (seen in the ETABS Verification section of this report below), but was not performed for other trusses due to the verification method's dependence on structural determinacy. Because the overall deflection at the 68' cantilever would be relatively large, X-braces were used where possible to provide extra stiffness and minimize deflections. This provision rendered each truss, with the exception of Truss N.2, statically indeterminate and did not allow for the use of an analytical method for selecting initial member shapes.

BASIC LOADING AND MODELING ASSUMPTIONS

Each truss was modeled independently with simple supports and major-axis moment releases for the diagonal and vertical members. Horizontal members, however, were modeled continuously except where different horizontal cross sections meet. Modeling each truss independently ensured that these simple end-releases are reliable and accurate assumptions, mitigating the effects of out-of-plane effects (i.e. torsion, minor-axis bending) from other steps of the load path at the connection sites.

IBC 2009 LRFD load combinations found in section 1605.2.1were used to determine the design loads of the proposed system. Equation 16-2 (below) was found to control in all cases for the gravity investigation.

$1.4(D + F)$ (Equation 16-1)
$1.2(D + F + T) + 1.6(L + H) + 0.5(L_r \text{ or } S \text{ or } R)$ (Equation 16-2)
$1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (f_1 L \text{ or } 0.8W)$ (Equation 16-3)
$1.2D + 1.6W + f_1 L + 0.5(L_r \text{ or } S \text{ or } R)$ (Equation 16-4)
$1.2D + 1.0E + f_1 L + f_2 S$ (Equation 16-5)
$0.9D + 1.6W + 1.6H$ (Equation 16-6)
$0.9D + 1.0E + 1.6H$ (Equation 16-7)

Loads were calculated using the Dead and Live Load Schedules found on drawing S-200.01 and applied to the Trusses appropriately. In the case of Truss 0.9 loads had to be calculated from level 6 to the roof level using tributary areas of each member supported by the truss. An additional dead load was added at the locations where columns from upper floors load Truss 0.9. A 2k point load was applied for each level supported by a column. Also, the steel panel exterior wall was estimated to have a weight of 15PSF, and was applied at typical loading points which (see Final Truss Design section below). Once modeled, the reaction from each of the Dead, Live, and Snow loads was used to load the next truss down the load path.

Additionally, the trusses are modeled such that connections are concentric. Diagonal and vertical members utilize only W14x shapes, while the top and bottom chord members use shapes determined to be efficient for both axial and bending forces. For design purposes, ETABS considers the top flanges horizontal members to be fully braced, and diagonal and vertical members to be fully un-braced if constraints are not added explicitly. Due to the preliminary nature of this investigation, P-Delta effects were not considered.

Finally, tension members were considered for yield strength only and rupture will need to be considered when designing the connections.

Deflection Considerations

Due to the nature of this investigation, the proposed trusses have been designed for strength. Deflection was considered for overall deflections at cantilevers and mid-spans in order to verify a serviceable design. The steel design analysis in ETABS considers certain deflection criteria when interpreting the adequacy of a given member which could not be modified. It is for those reasons that deflection failures of individual members were ignored in ETABS and overall deflections were checked for serviceability. Design deflection results and further discussion can be found in the Overall Deflection of the Cantilever section of the Final Report below.

FINAL TRUSS DESIGN

TRUSS 0.9

Since the floor systems of neither level 5 nor level 6 needed altering for the proposed load path, Truss 0.9 is the primary support for those levels (Figure 26). The truss is loaded along the top and bottom chords. In order to accurately model the combined member loading (bending and axial forces), Truss 0.9 is loaded every 10' according to the beam spacing on the two levels. This adherence to the existing floor framing system loads the truss at each major panel joint and the mid-spans of the top and bottom chords.

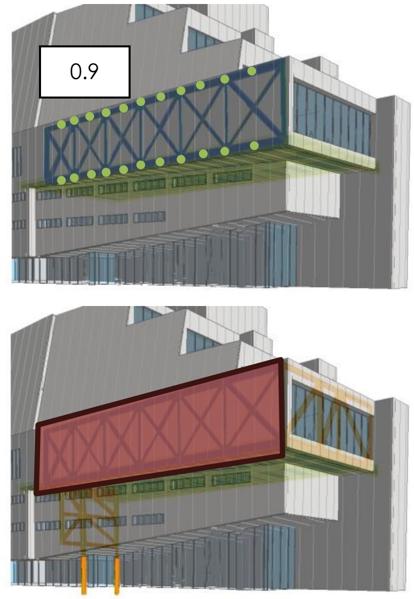


Figure 26: Truss 0.9 Location and Load Path Orientation

Truss 0.9 is supported by Trusses H and N.2 and was modeled using the conditions shown in Figure 27 below. Truss H lies between levels 3 and 5 and therefore supports Truss 0.9 at the bottom chord only. Alternately, Truss N.2 lies between levels 5 and 6 and supports Truss 0.9 at both levels. A "roller" connection was modeled at each level to provide an accurate reaction scenario. Major-axis moment releases are shown. The W14x120 between the roller supports is merely a placeholder and was not considered in the design of the truss.

Member sizes were finalized on criteria of combined loading efficiency, weight, and constructability. Proposed Truss 0.9 weighs 12.2 t more than the current truss (41.2 t) for a total of 53.4 t. More detailed weight calculations can be found in Appendix F.

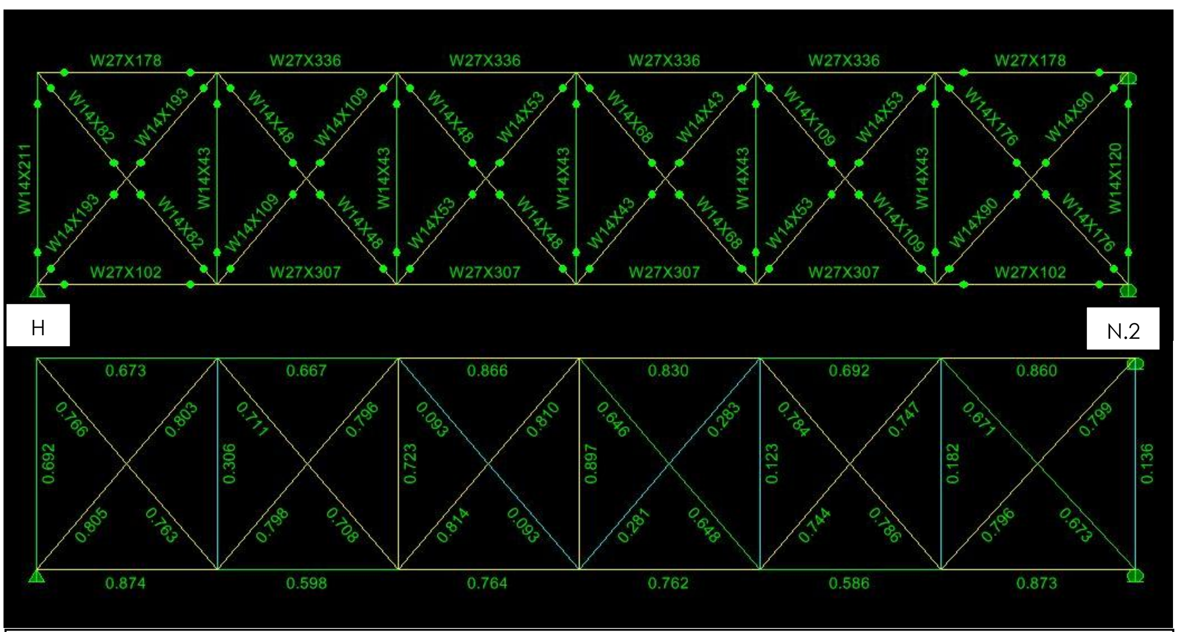


Figure 27: Truss 0.9 Modeling input and Design Results

TRUSS N.2

As the second step of AAM's proposed load path, Truss N.2 supports Truss 0.9, and cantilevers 26' past Truss X as seen in Figure 28. A tension support is added at column line 6 to resist uplift. Because Truss N.2 runs parallel to the floor framing beams, floor loads at the Eastern edge of levels 5 and 6 are applied as distributed loads. Point loads are applied at column line 0.9 according to the reactions from Truss 0.9. More detailed load calculations can be found in Appendix E.

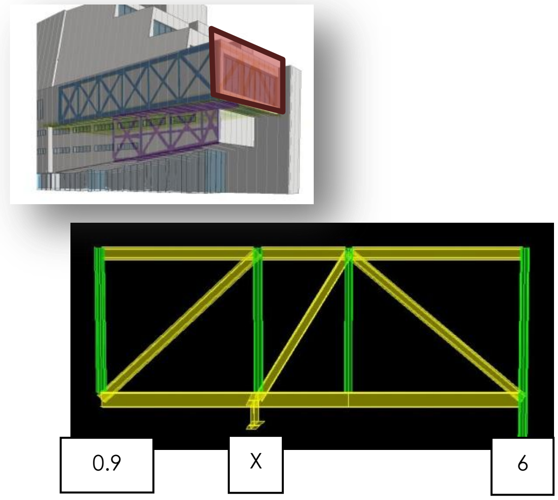


Figure 28: Truss N.2 Location and Load Path Orientation

The shape of Truss N.2 was determined by architectural constraints discussed in the Architecture Considerations section of this report, and was modeled according to the conditions shown in Figure 29 below.

Both the top and bottom chords of Truss N.2 are to be continuous sections for the entire 70' length. The truss weighs 36 t, 9.8 t heavier than the original floor framing. A more detailed weight comparison can be found in Appendix H.

The selection of Column 6-N.2 is discussed in the Impact on Foundations section below.

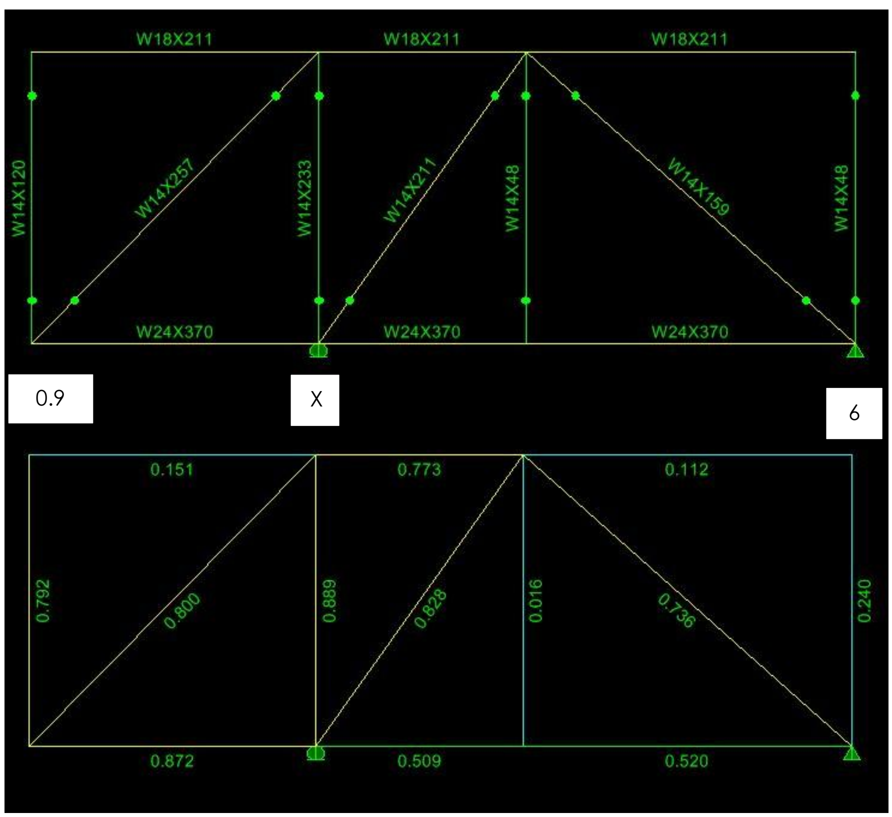


Figure 29: Truss N.2 Modeling input and Design Results

TRUSS X

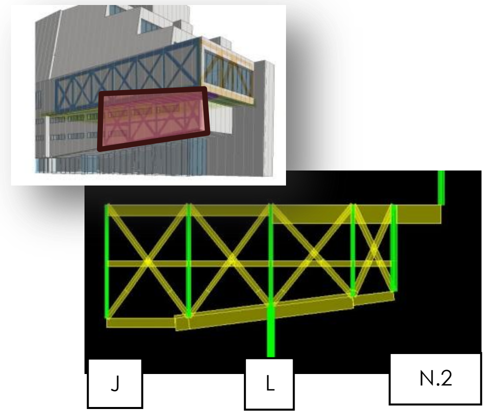


Figure 30: Truss X Location and Load Path Orientation

At over 120 t (see Appendix H), Truss X is the heaviest system of the proposed structural design. Loads from levels 5-9 are applied where Truss X supports Truss N.2 above, and is cantilevered 45' from Column 3-L (see Figure 30). Uplift is resisted by a final truss at column line J. A small distributed load was applied to the top chord at level 5, and point loads were applied at the columns on levels 3 and 4. This placement ensured an accurate model while avoiding unwanted loads applied to the diagonal members. The W16s inserted at level 4 act only as bracing for the diagonal members.

As is further explained in the Architecture Considerations section of this report, the existing architectural envelope limited the depth of Truss X to 56". In order to transfer the loads between the load point at column line N.2 and Truss X, restricted by the envelope at column line M.5, custom section PG56-1 was designed for adequate shear and moment capacity, and is explained in further detail in Custom Cross Sections section above.

In addition to having the highest weight, Truss X is the only truss system which contains members designed for over 90% efficiency, which can be seen in Figure 31, below. Both the top and bottom chords were deemed acceptable in order to minimize truss weight. PG56-1 weighs 1909 plf, and PG46-3 weighs 748plf (see Custom Cross Sections section above), so an increase in beam size was not considered once a passible capacity was determined. Members in red in the figure which are less than 95% efficient signify a deflection failure.

The vertical member at location X-L was sized as a W14x257, and the design of Column 3-L is also discussed in the Proposed Custom Sections section of this report.

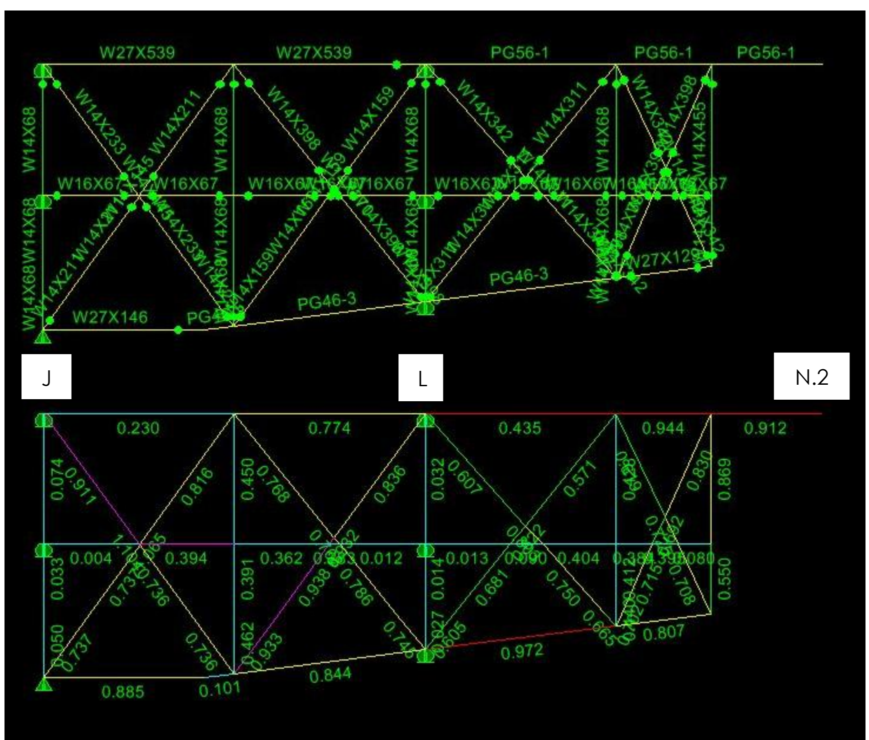


Figure 31: Truss X Modeling input and Design Results

Truss J

The primary purpose of Truss J is to resist uplift caused by the cantilevered Truss X. Figure 32 shows that Truss J is also cantilevered over a support at Column 3-J, which resists uplift, while Column 4-J resists compression. The design of Truss J's supporting columns can be found in the Impact on Foundations section of this report.

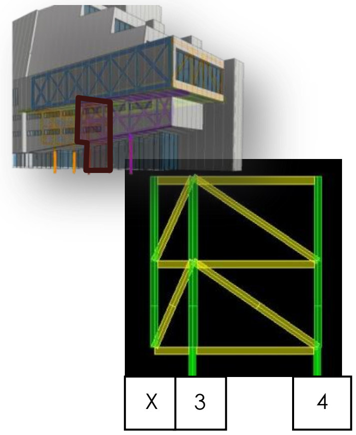


Figure 32: Truss J Location and Load Path Orientation

Because proposed Truss X spans two stories between levels 3 and 5, it was decided that its uplift support, Truss J, should also cover both stories. Also, for reasons specified in the Architecture Considerations section of this report, the position and orientation of the diagonal members was maintained.

Weighing 45 t in the current designed, the weight of Truss J could be reduced by 36 t (to 9 t) under the proposed system. Figure 33 below shows that the majority of members are W14x68s, the heaviest being a W14x145.

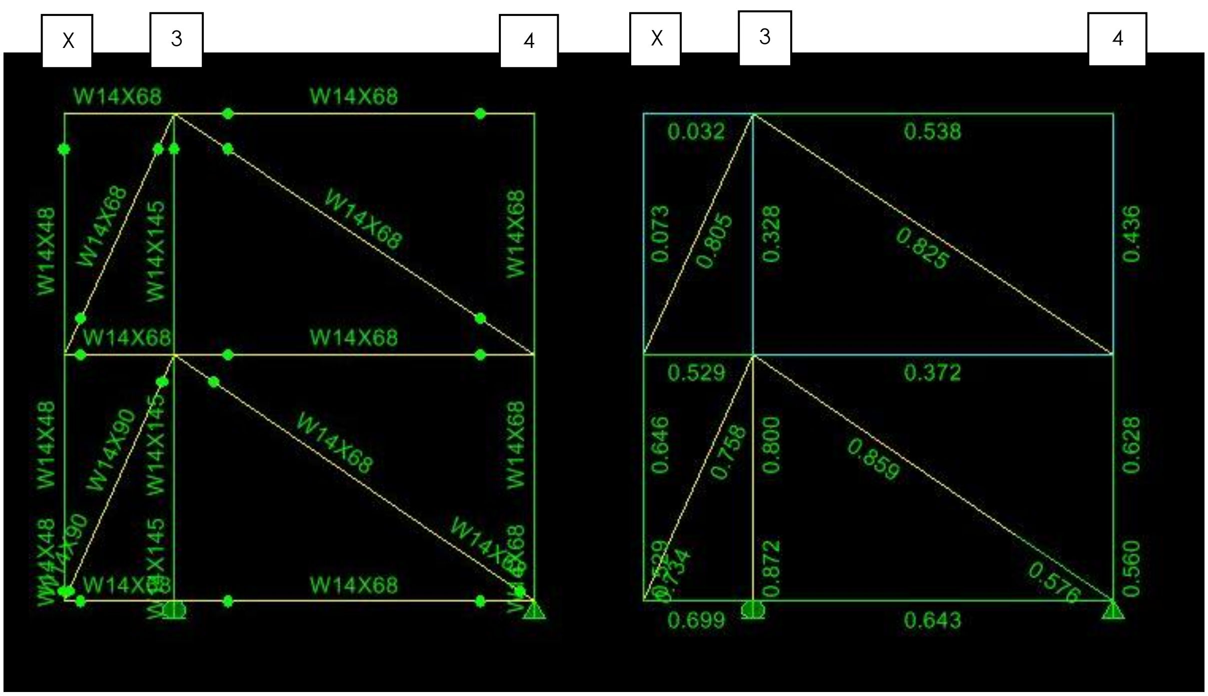


Figure 33: Truss J Modeling Input and Design Results

TRUSS H

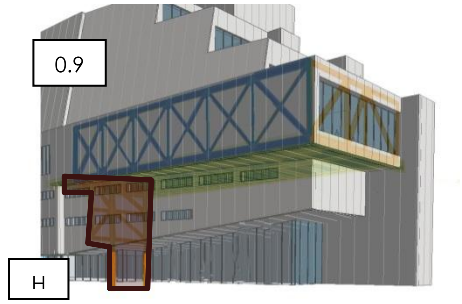


Figure 34: Truss H Location and Path

Truss H is the Eastern support for Truss 0.9 (shown in Figure 34). The architectural envelope, further discussed in the Architectural Considerations section of this report, dictate that a single beam must be cantilevered 7' from the rest of Truss H to support Truss 0.9 in a similar fashion to Truss X (see Figures 31 above, 35 below). Furthermore, the first panel of Truss H is cantilevered 12'-6" from its last support at column line 3. Also, red members in Figure 35 below signify failure by deflections.

Similar to Truss J, the shape of Truss H has not changed from the current design for reasons also discussed in the Architecture Considerations section of this Report. Loads from the Truss 0.9 above and the floor loads were reevaluated and new members were selected.

Truss H features a W14x665 at the location 3-H, the heaviest rolled Wide Flange section in the proposed structural system. Also, custom section PG46-2 was found to be adequate to carry the loads of Truss 0.9 at the top chord.

The design of the supporting columns is discussed in the Impact on Foundations section of this report.

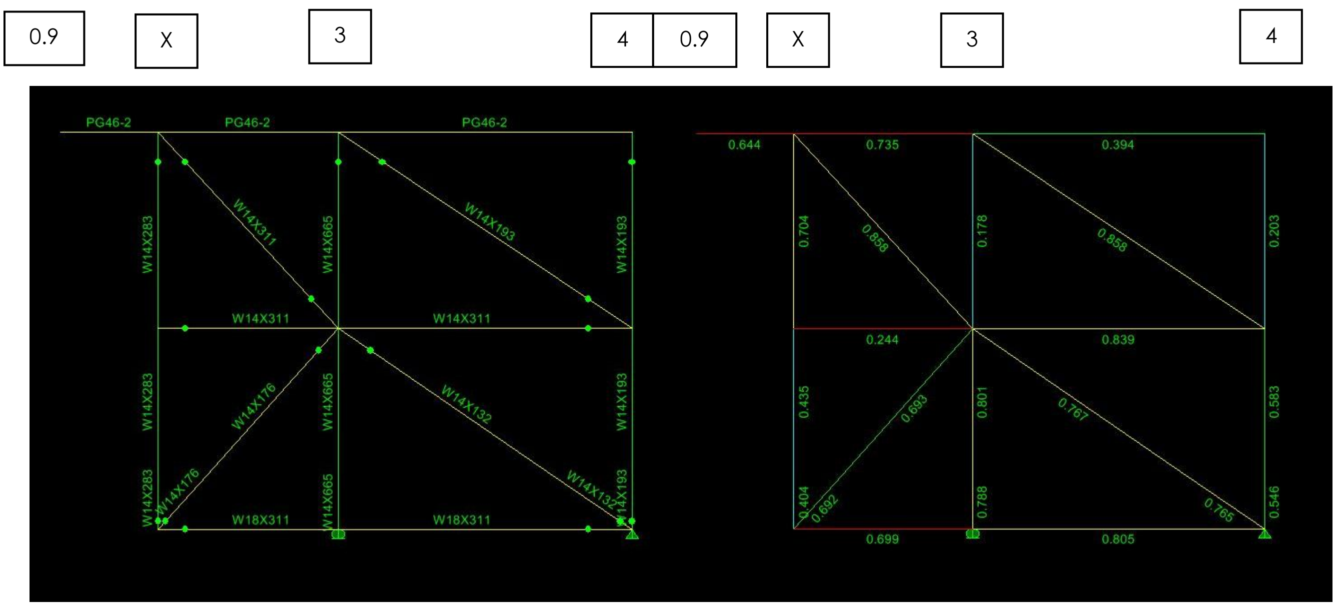


Figure 35: Truss H Modeling Input and Design Results

ETABS Verification

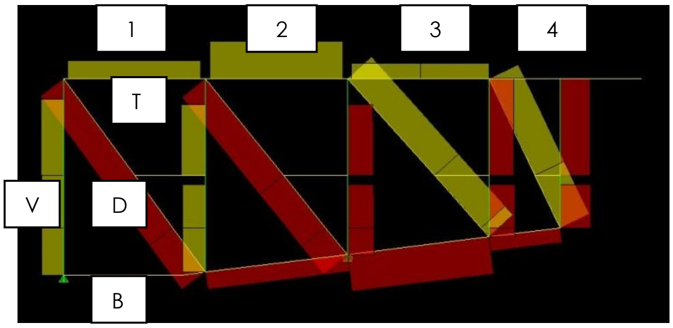


Figure 36: Nomenclature for Truss X Variation

ETABS's analysis of truss shapes was verified with hand calculations using a simplified variation of Truss X shown in Figure 36. First, hand calculations were performed to find the axial loads in each member. Trial member sizes were then selected for ETABS, and the resulting axial forces in ETABS were compared against the axial capacities of the members selected by the hand calculations. A summary is provided in Figure 37 below.

A comparison of loads to capacities was deemed to be more accurate and to better reflect efficiency because it more closely resembles the design process than a comparison of loads alone. Values highlighted green in Figure 27 reflect loads that are conservative compared to the hand values (load exceeds capacity), and the values highlighted in red reflect non-conservative ETABS loads. All load magnitudes, however, are within 10% of the selected member capacities, and therefore verify ETABS's truss analysis.

The load patterns used for the hand calculations match those used for the ETABS verification of this model but reflect an earlier iteration of the design process and do not match the loads used for the final proposed design of Truss X. More detailed calculations and selected member sizes can be found in Appendix D.

	ETABS		Hand	Error	
Frame	Shear	Axial	ϕPn	D	%D
1	2005	3393	3077	315.8	-9.31
2	2315	3724	3759	35.49	-0.95
3	2400	3568	3690	122.2	+3.42
4	1985	4635	4902	267	+5.76

Figure 37: Diagonals ETABS/Hand Comparison

IMPACT ON FOUNDATIONS

As the final stage in designing AAM's superstructure without Column 3-M.5, an analysis was performed to determine the adequacy of the current foundation design. First, the final support reactions were itemized from trusses H, J, and X, and columns were selected or designed to carry the required loads. Next, remaining loads from level 1 were added and the substructure established. Finally, the pile arrangements supporting each of the columns were re-evaluated to reflect the strength requirements of the proposed design.

Columns supporting the trusses were not considered as part of the truss system and were therefore not analyzed in ETABS. Applied loads, however reflect the ETABS reactions factored according to the load combination parameters described in the Load Path Overview section of this report. Figures 38 and 39 below show the factored loads and members selected for each of the affected columns in the superstructure and substructure respectively. Member sizes were selected based on a 25' un-braced length. More detailed calculations can be found in Appendices E (superstructure) and F (substructure).

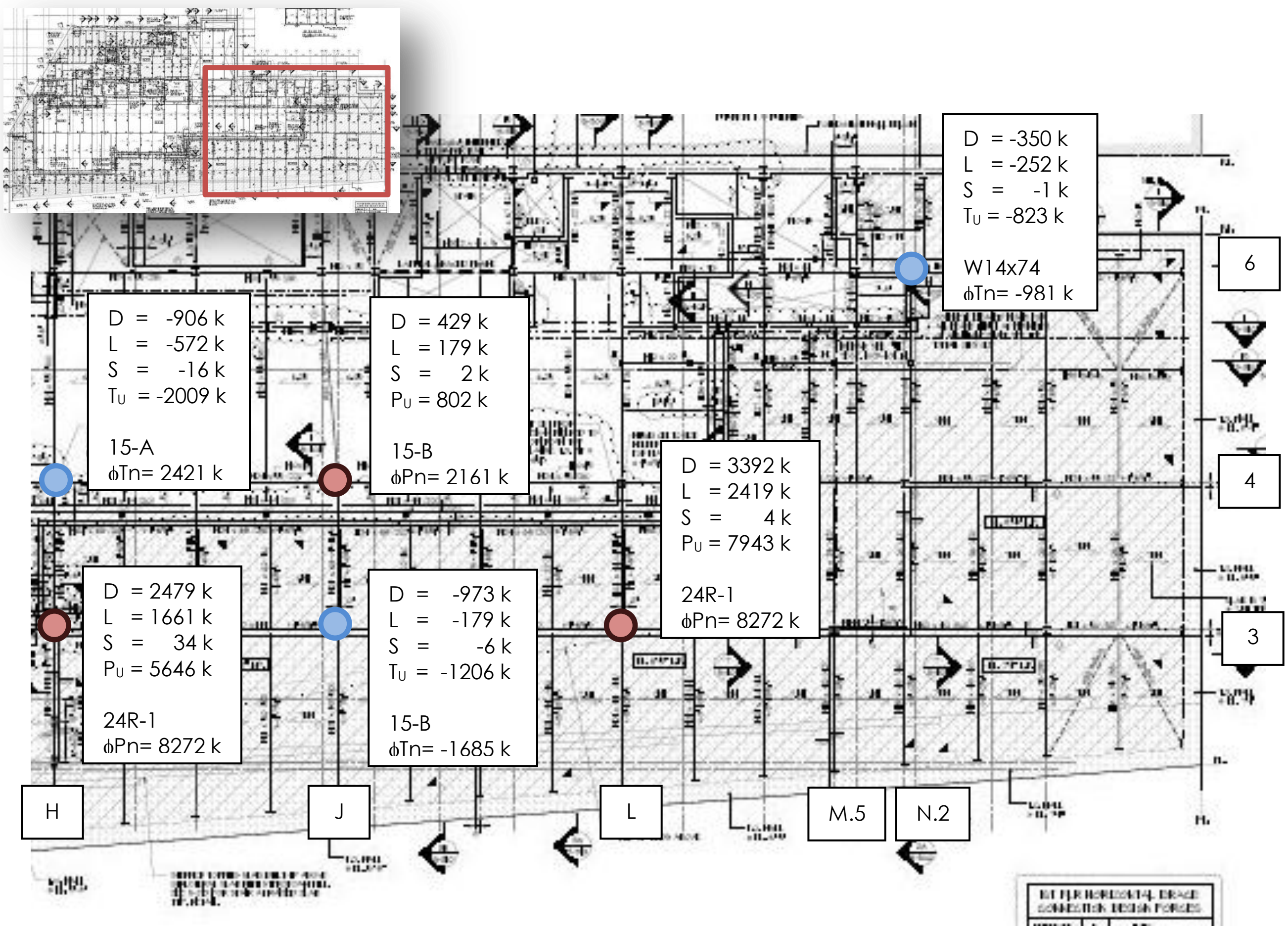


Figure 38: Truss Support Columns

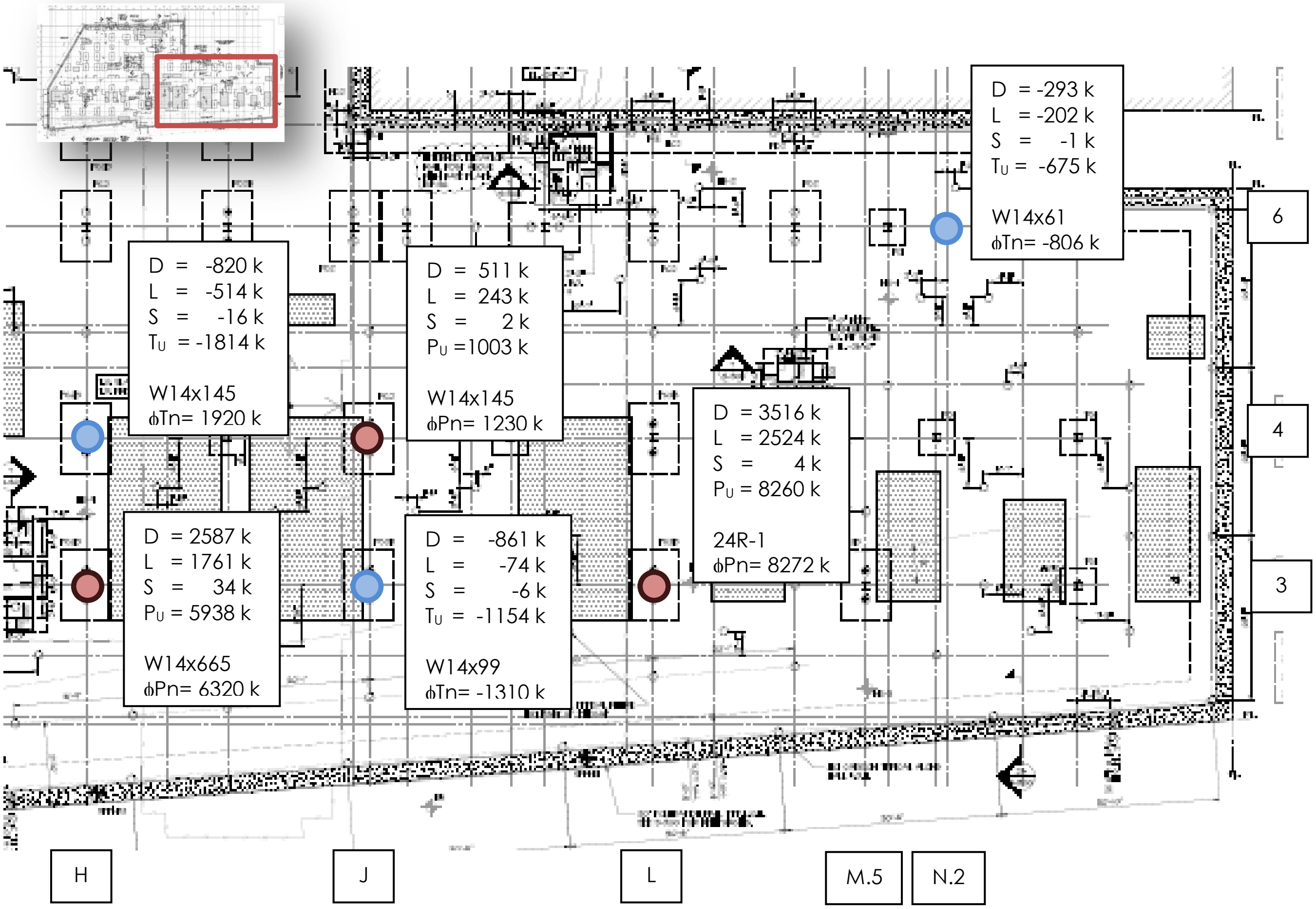


Figure 39: Substructure for Truss Support Columns
Tension Support
Compression Support

The loads shown in Figure 39 were used to analyze the capacity of the current pile arrangements before a final proposal could be issued. The capacities of assigned pile identifications are shown in Figure 40 below.

CAISSON SCHEDULE

	MARK	NOTES	CAISSON O.D.	CASING	CAISSON REINF. VERT. BARS	MIN. DEPTH OF ROCK SOCKET*	TENSION CAPACITY (TONS)	COMPRESSION CAPACITY (TONS)
1		TYPICAL, @PC/WALL	13.375"	½" THICK Fy=80 ksi	1 #24	11'-0"	151	414
2		HIGH CAPACITY	13.375"	½" THICK Fy=80 ksi	2 #24	16'-0"	227	621
3		TCI - NOT @PC/WALL	9.875"	½" THICK Fy=80 ksi	1 #24	15'-0"	151	91

Figure 40: Caisson Schedule (FO-100)

An adequate number of piles were grouped to bear the loads from each column. While the columns were designed using LRFD ultimate loads shown in Figure 39 above, the foundation drawings do not contain notation that suggests LRFD was used. The pile capacities provided are therefore assumed to be based on ASD. Itemized column loads were simply added according to IBC Equation 16-9:

D+H+F+L+S+T

Figure 41 below summarizes the pile group requirements for the proposed structural system. This report does not include provisions for changing the capacity of the piles, but rather arranges the existing pile designs such that pile groups can adequately support the loads from above. Should the proposed system be accepted by the architect, the pile caps at the new pile groups will need to be designed as they were considered out of the scope of this investigation.

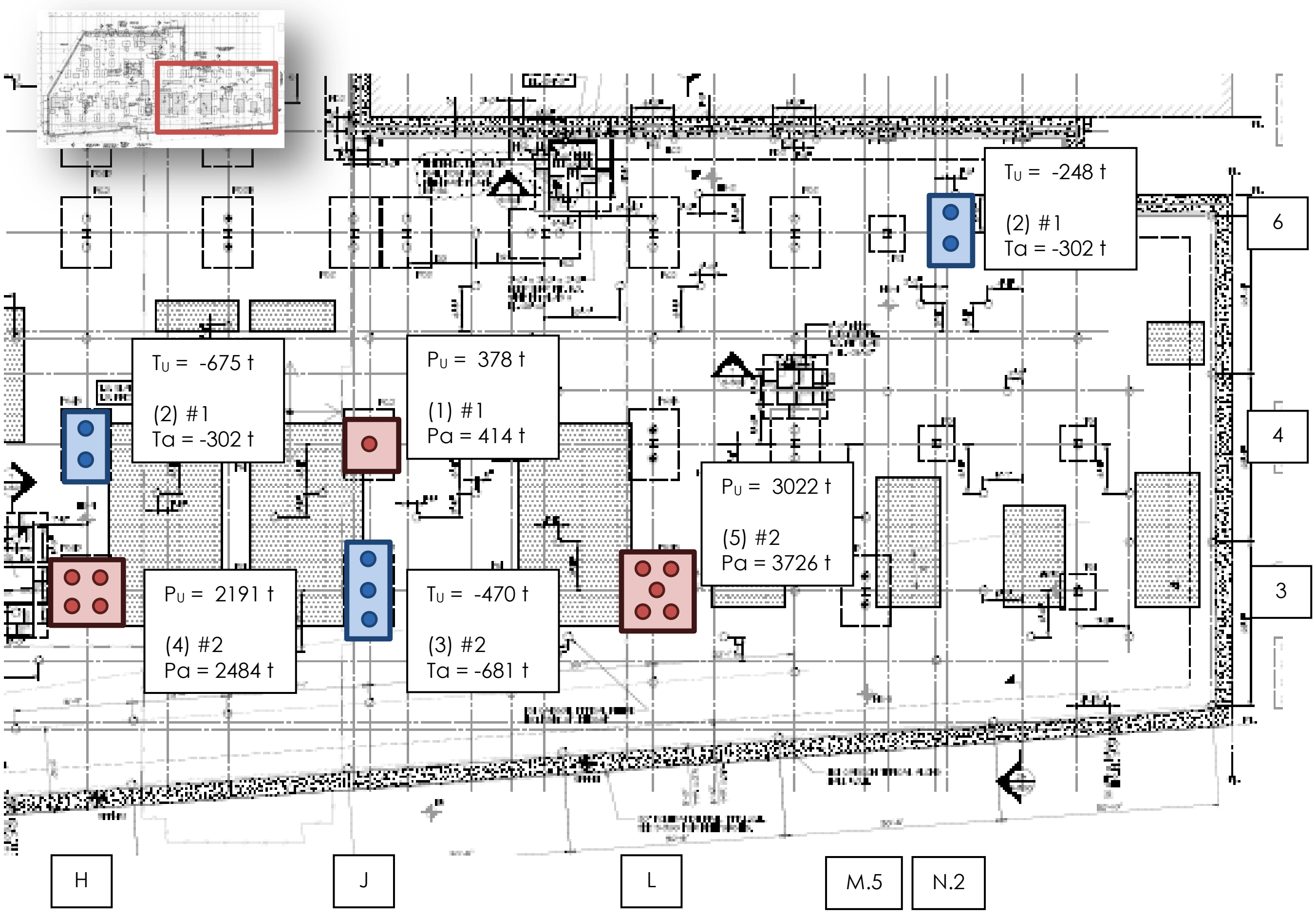
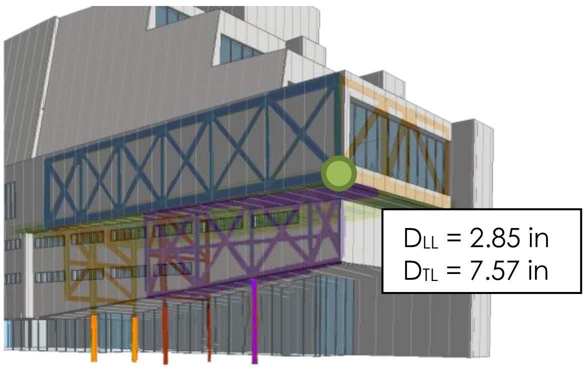


Figure 41: Pile Requirement Summary
Tension Support
Compression Support

Deflections and Serviceability

Overall Deflection of the Cantilever

Though incremental deflections were not considered in the design of the trusses, the overall deflection at the cantilever was analyzed to determine the adequacy of AAM's proposed structural system. The allowable deflection at the cantilever was performed for both Live and Total load conditions using the shortest distance, 45'-10", to the last support at 3-L. Figure 42 below shows that deflections due to live loads were deemed acceptable, while the deflections due to total load fail by approximately 3". Further Calculations can be found in Appendix G.

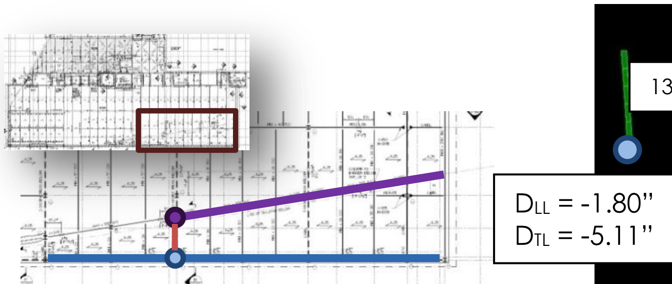


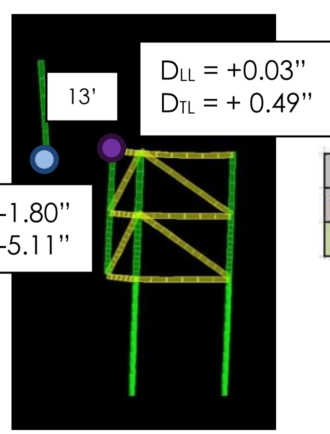
Cantilever			
LL	Dalw	TL	Dalw
2.85	3.06	7.57	4.59

Figure 42: Total Deflection Summary

Serviceability

In addition to the overall deflection of the cantilever, the trusses' close proximities also could create adverse effects on the serviceability of the structure. Figure 43 shows how the live load deflections were checked for proximity as well as span and cantilever length. At column line J level 5 experiences live load deflections in two different directions: up where Truss J supports Truss X and down where the floor is supported by Truss 0.9. The distance between these trusses is 13' (156") at this location, giving a maximum allowable LL deflection of 0.65" (l/360). In contrast to the cantilever, neither live load nort total load deflections between the highest point of Truss J and the closest deflected point at truss X pass. The deflections are so severe that the floor, wall, and ceiling materials risk damage. Furthermore, deflections of 6" over 13' would be visible under service dead and live loads.





X:09			
LL	Dalw	TL	Dalw
1.83	0.65	5.60	0.98

Figure 43: Deflections between Trusses 0.9 and X,J

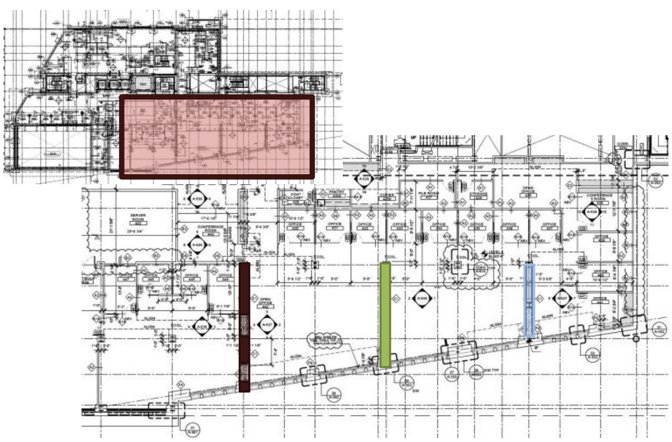
ARCHITECTURE CONSIDERATIONS

OVERVIEW

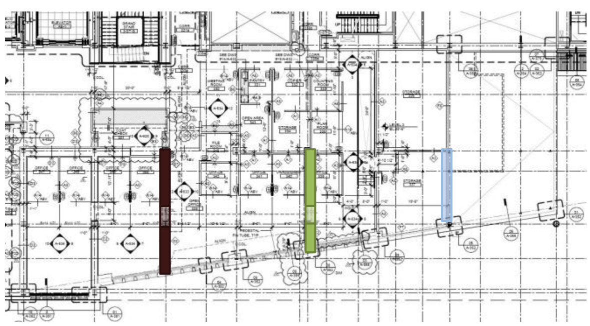
Respect for the current architectural scheme was a crucial consideration in the redesign of AAM. The office spaces on levels 3 and 4 are connected by passages through the existing truss systems. Entire systems and components such as section Truss N.2 and section PG56-1 were designed specifically to mitigate or eliminate clashes and alterations of the architecture. Some conflicts, such as Truss X's placement in front of office windows could not be avoided and will require further input from the architect.

LOWER TRUSSES: OPEN OFFICE SPACES

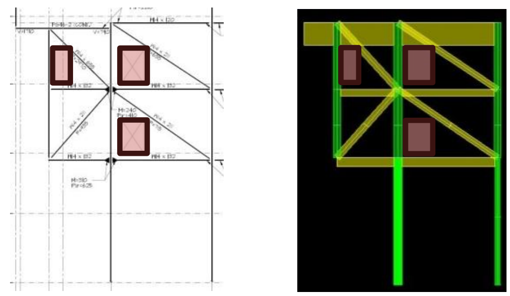
As mentioned above, the open office spaces on levels 3 and 4 are broken by gravity trusses which support the upper floors. In order to allow movement between these spaces, the web openings in the trusses were utilized by the architect. Figure 44 below displays how these openings were maintained in Trusses H and J for the proposed redesign. Additionally, Truss L was reduced to a single column, providing more flexibility for the open office space on level 4.



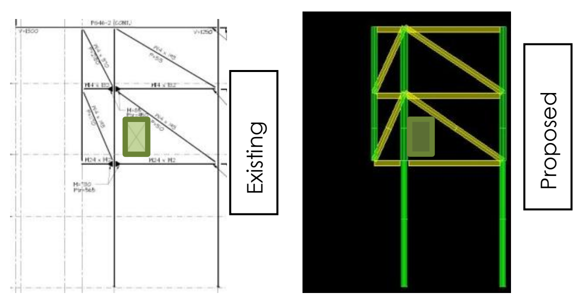
Level 4



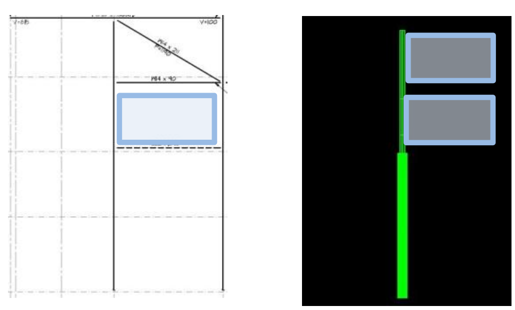
Level 3



Truss H



Truss J



Truss L

Figure 44: Openings through Gravity Trusses in Open Office Spaces on Levels 3 and 4 (Drawings A-103, A-104, S-122 and ETABS models)

TRUSS N.2: LEVEL 5 GALLERY

PLACEMENT ALONG EAST WALL

Perhaps the most notable challenges with respect to the proposed structural system conforming to the architect's vision for AAM arise from the addition of truss N.2 (shown in Figure 45 right). Located between levels 5 and 6, proposed Truss N.2 marks the East end of the main gallery space of the museum. This main gallery was designed with a 200PSF live load, twice the code minimum for assembly spaces, and boasts 16,000 uninterrupted sq. ft. of space, made possible through a 70' span. All of these exceptional structural provisions were done to provide maximum flexibility for the space's use.

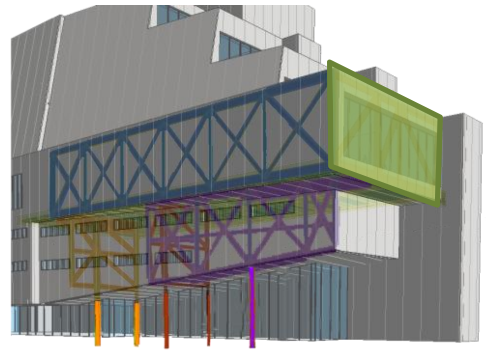


Figure 45: Location of Truss N.2

Another aspect of the uninterrupted space is the opportunity for long views which will provide relief to the public when visiting AAM. Large, uninterrupted windows were placed in the current design at the East and West walls of this main gallery. The East window will overlook the High Line park and city skyline, and the West will overlook the river and opposing shoreline (see Figure 2 in Building Introduction section). Proposed Truss N.2 is placed directly inside the East window, and would create a more obvious physical boundary between the gallery and its exterior view, while the West would appear to remain boundless to the river and beyond. Both the current and proposed designs can be seen in Figure 46 below.

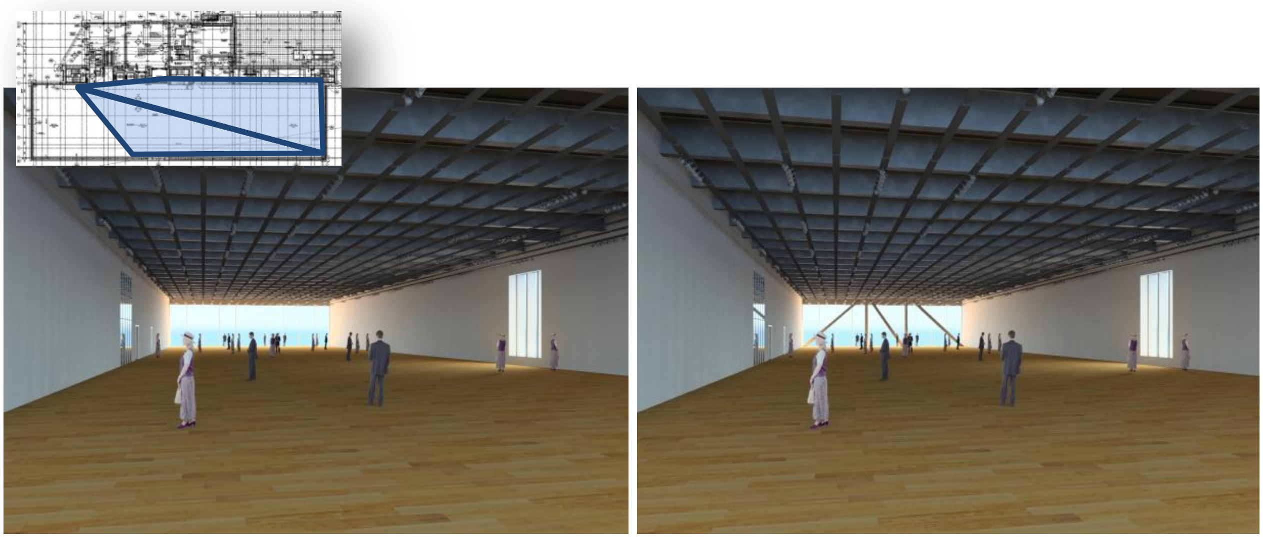


Figure 46: Interior Renderings of Level 5 Gallery Space with Current (top) and Proposed (bottom) Designs (A-105)

ARCHITECTURAL USE OF STRUCTURE

While the addition of proposed Truss N.2 may conflict with the architectural aura of the main gallery space, there is precedence for exposing structural steel both in AAM and in Renzo Piano's other projects. Figure 47 provides an elevation of the exposed bracing in the level 1 gift shop, and Figure 48 shows the use of exposed structure in another Renzo Piano building.

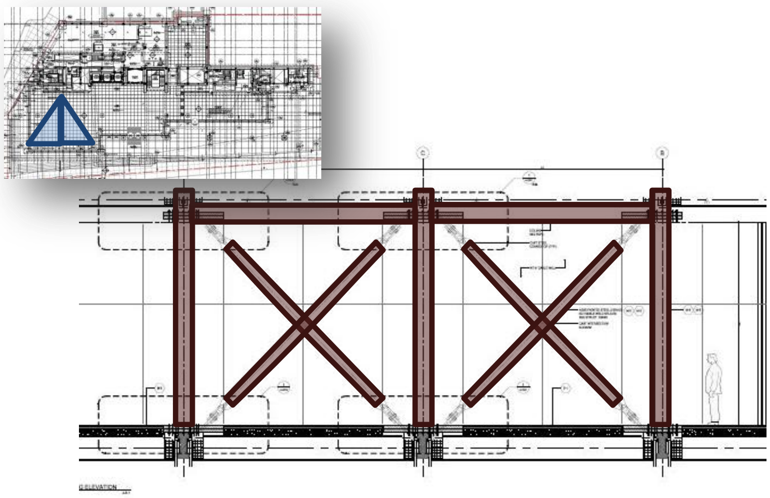


Figure 47 (Left): AESS in AAM lobby (A-399)

Figure 48 (Right): AESS used in Another Renzo Piano Building (courtesy of RPBW).

Renzo Piano's design for the Il Sole 24 Ore headquarters in Milan, Italy (Figure 48) utilizes AESS in both the interior and exterior portions of the building. Furthermore, while AAM's main public space is the level 5 gallery, Il Sole's most important space is its main lobby. Both buildings highlight their respective structures as vital to the architecture without being overbearing. This balance is achieved by using the slender, round sections, and by strictly adhering to the rhythmic architectural module.

SECTION AND MODULE INCONSISTENCIES

Truss N.2 could not be designed with the round sections described above for strength reasoned which are further discussed in the Proposed Structural Design section of the Final Report. Instead, Wide-Flange shapes were used to carry the large axial forces present within the truss.

As mentioned in the Building Introduction section above, the steel panels that dominate the façade of the building work on a 6'-8" module. AAM's entire exterior, as made evident by the East elevation shown in Figure 49 below, was composed for harmony between the glass panels and steel panels, conforming to the modular rhythm established by the architect.

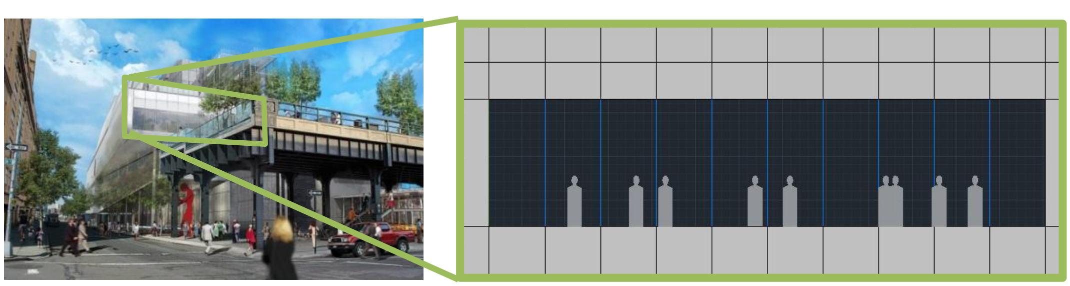
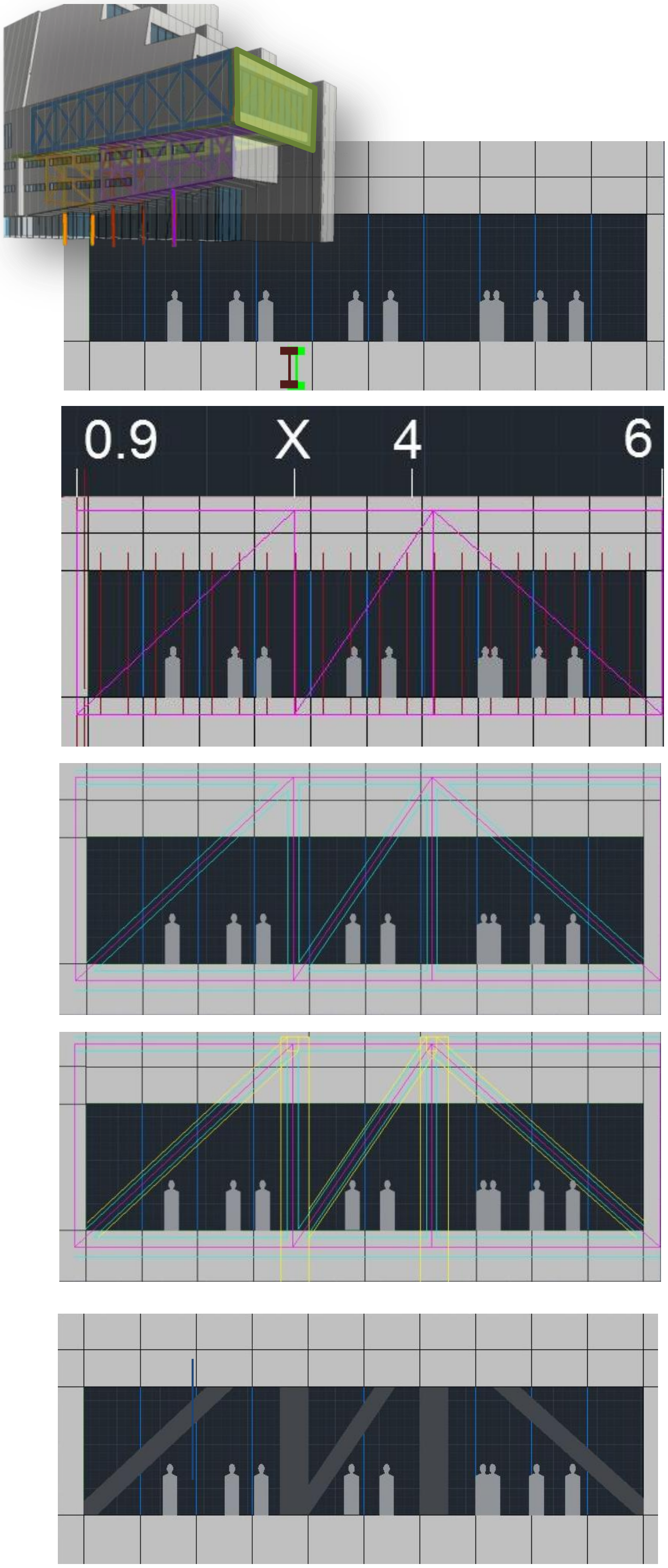


Figure 49: Current Façade Design



(A) (B) (C) (D) (E)

Proposed Truss N.2, however, cannot conform to the existing grid established by the façade and glazing panels. Figure 50-A (left) shows where Truss X intersects with and supports Truss N.2. Because column line N.2 lies slightly inward from the East wall, the intersection of the two column lines nearly coincides with the third-quarter point within the fourth wall panel (item B). Furthermore, the point of intersection creates an akward 26'-1" cantilever out to column line 0.9 from the support at Truss X.

In an effort to design a symmetric, yet rational truss for N.2, the other vertical member was placed at a more constructable 2'-6" north of column line 4 (see item B). This position was chosen because it very nearly coincides with the first-quarter point within the sixth wall panel.

While the two verticals are very close to a perfectly rational design that is consistent with the architecture, the small discrepancies of no more than 2" remain. The placement of the forward vertical member cannot be altered due to structural requirements, meaning that a symmetric and efficient design cannot be wholly reconciled to the panel system currently in place.

One option for creating the perfect alignments that are uniquely considered in Renzo Piano's architecture is an additional envelope around Truss N.2. A rational design of this envelope is shown in items D and E of Figure 50. The envelope first covers the columns on a half-panel basis, keeping and perfecting the symmetry of the truss. Secondly, the diagonal envelopes extend from the corners of the rectangles that form from the intersection of the vertical members with the top chord of Truss N.2. This rational design reinforces the rhythm of the façade and minimizes the impact of the truss within the main gallery space. Furthermore, exposing Wide Flange sections within AAM would be inconsistent with the exposed HSS braces visible on level 1. Figure 51 below shows how the enveloped truss would appear inside the main gallery.

Figure 50: Truss N.2 Module Conflicts:
(A) Intersection of Truss X and Truss N.2
(B) Proposed Truss N.2
(C) Interior Envelope Overlay
(D) Interior Envelope Schematic
(E) Proposed Alternative Truss Cover

Figure 51: Enveloped Truss N.2 inside Level 5 Gallery

PG56-1: Architectural Envelope

In order to accommodate the large structural members required in the cantilever, the architect allotted 5'-8" of space between the top of the floor on level 5 and the bottom of the architectural envelope as shown in Figure 53. When considering a 10"-thick floor system and a 2"-thick envelope structure, 4'-8" (56") remain as the absolute maximum thickness for a structural member.

The current design employs 46"-deep plate girders, which leaves an additional 10" of clearance for MEP systems. A 56"-deep member allows no additional space for MEP systems, meaning that web openings would need to be considered for actual use.

Furthermore, proposed section PG56-1 does not strictly adhere to the precedent plate components established in the current design. Both the plate thickness and width had to be increased to accommodate the loads. A section following the precedent plate sizes would need to be 76" deep to achieve the same strength as PG56-1 (see Appendix C). Adhering to the envelope limits established by the architect was the chief constraint in the design of PG56-1.

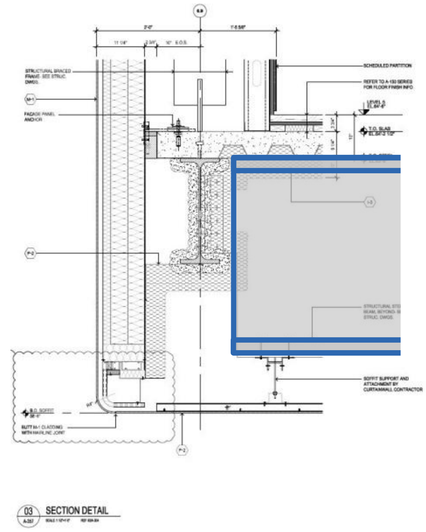
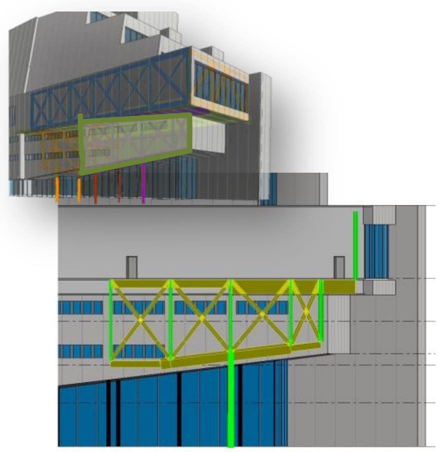


Figure 52: Section showing envelope dimensions at cantilever (3: A-357)

Plate Girder Outline



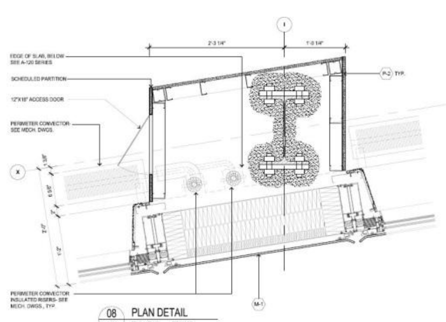
Truss X: Levels 3 and 4 Exterior Wall

As the third layer of the proposed load path and the primary support of the cantilever, Truss X carries the largest loads in AAM, thus requiring the X-bracing and two-story geometry shown. Though the horizontal spacing was held according to the current design, Figure 53 shows (left) shows how the diagonal braces could clash with the window placement in the open office spaces on levels 3 and 4.

Detail 08 on drawing A-352 (Figure 54 below) shows that the exterior face of the wall lies 24" outside of column line X, and the inside face is specified as against the fireproofing foam. The outside face will not be affected by Truss X; the widest shape, PG46-3, is 18" wide and will fit well within the exterior building envelope. Also, because the exterior face has no interference, the windows do not necessarily need to be moved or changed if a visible truss is deemed acceptable.

Figure 53 (Above): Structural/architectural conflict on levels 3 and 4.

Figure 54 (Below): Exterior Wall Detail (A-352)



Since Figure 54 is based on a W14x column, the drywall will be pushed into the office space by 2" under the proposed structural system. Because the current design is so dependent on the windows it is difficult to judge how much square footage will be lost in these spaces.

Construction Management Considerations

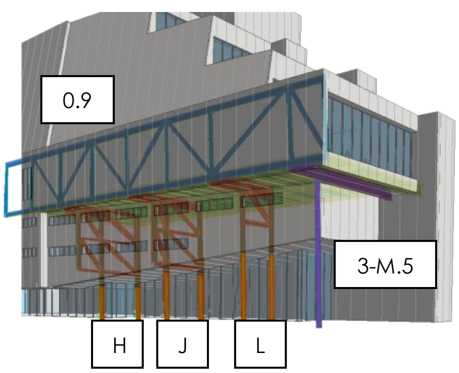
Overview

Because the structural system proposed in this report was designed to minimize effects on the established architectural scheme, the construction of AAM will be both more expensive and more difficult. As briefly described in the Proposed Structural Design section above, the weight was increased and its distribution changed. Those alterations to the superstructure also affected the number and arrangement of the piles at the foundations. Additionally, the proposed structural system consists of long-span trusses (up to 122'), which will be difficult to both transport to the site and to lift into place. Finally, both proposed custom sections PG56-1 and 24R-1 will require special consideration for the procurement of elements and construction techniques.

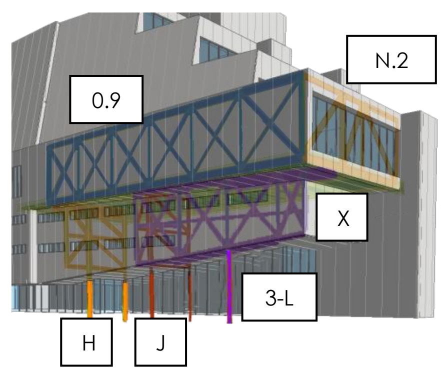
Cost

Superstructure

Cost data provided by a contact at Barton Malow Company assesses the cost of structural steel based on its overall weight, so a takeoff was performed to compare the weight of the current and proposed truss systems. The proposed structural system weighs nearly 100 t heavier than the current system. Where the heaviest element currently is Truss J at over 45 t, proposed Truss X weighs over 120 t alone. Figure 55 below summarizes the findings, and is broken down by congruent element.



Existing Design	
Truss	Weight (tons)
0.9	41.2 t
3-M.5/PG	26.2 t
Wall X	19.3 t
H	35.8 t
J	45.1 t
L	32.0 t
Total	199.5 t



Proposed Design	
Truss	Weight (tons)
0.9	53.4 t
N.2	36.0 t
X	121.6 t
H	63.9 t
J	9.3 t
3-L	12.9 t
Total	297.1 t

Figure 55: Weight Comparison of Existing (left) and Proposed (right)

Once the weight of both systems was established, the cost data was applied and an increase in cost of $2,017,824 was found for the proposed system, which is summarized in Figure 56. The starred values were provided by Barton Malow from the company's 2012 cost database. Because this data is only for weight, and does not account for the location, timeframe, or specialty items, increases were added for both the current and proposed designs for a conservative estimate. Furthermore, location factors were taken from RS Means 2012 for the correct city (which the owner requested not to be disclosed). A time factor accounts for 1% inflation because the steel framing was built early in 2013, not in 2012. Finally an Overhead and Profit factor of 15% was added to determine the total cost of each system. More detailed calculations are provided in Appendix H.

	Weight		Material			Fabricating			Install					
System	tons	lbs	*0.80	cost	Loc	*2.50	cost	Loc	*2.75	cost	Loc	Time	O&P	Total Cost
Original	199.5	399045	0.80	319236	1.04	2.68	1069442	1.670	2.78	1109346	1.139	1.01	15%	3928375
Redesign	297.1	594224	0.80	475379	1.04	2.75	1634115	1.670	2.80	1663826	1.139	1.01	15%	5946200
												Total	$	2017824

Figure 56: Superstructure Cost Comparison

FOUNDATIONS

Unlike the structural steel, no cost data was provided for the foundations, so the cost analysis for the piles was performed according to RS Means 2012. In order to use RS Means, however, the deep piles had to be taken off in terms of vertical linear feet. Figure 57 below shows a geologic section provided by the URS Geotechnical Investigation (2011). The end-bearing piles will rest on bedrock, which lies at an average depth of 90' for the site. Knowing that the bottom of the floor slab rests at a depth of 22', the piles must extend roughly 68' before being embedded into the bedrock.

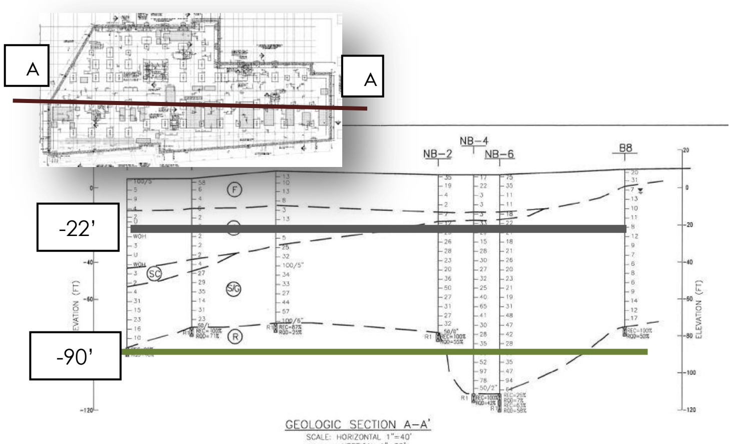


Figure 57: Geotechnical Section A-A

The Caisson Schedule on S-120.01 (see also Figure 40 in the Proposed Structural System section above) notes that each of the caisson types has a unique embedded length. Once a cost/linear foot value was established for the piles, the overall cost was determined by finding the total length of driven piles for each system. A summary is provided in Figure 58.

Comparing the number of each type of pile used determined the total cost for each foudnation system because of the differing embedment lengths. The proposed foundation system costs nearly $100,000 more than the current design.

Current	Type	n	Cost
	1	2	37722.01
	2	10	200547.39
	Total	$	238269.40
Proposed	Type	n	Cost
	1	5	94305.02
	2	12	240656.87
	Total	$	334961.90
Difference		$	96692.49

Figure 58: Foundations Cost Comparison

CONSTRUCTABILITY

TRUSSES

TRUSS 0.9

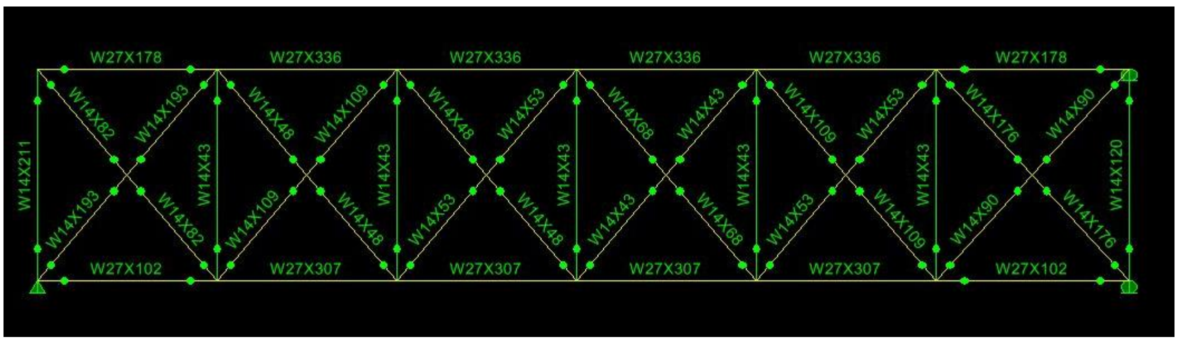


Figure 59: Truss 0.9 Constructability Concerns

Proposed Truss 0.9 spans 121.5' from gridline H to gridline N.2, is 23'-8" tall, and weighs over 53 t. It is highly unlikely, therefore, that a single crane could lift the whole truss into place. Furthermore, the city's access points, streets, and intersections are likely too low and too narrow to bring Truss 0.9 in by truck. In an effort to ease these constraints, and increase the structural efficiency of the truss, pin connections were added to separate the 4 interior panels from the 2 exterior panels (shown in Figure 59 above). This provision changes the longest span to 80', which may make truck transportation possible. If truck transportation remains impossible, however, the General Contractor will need to arrange for The Truss to be barged in on the river adjacent to the site (see Figure 2 in the Building Introduction section above).

Truss X

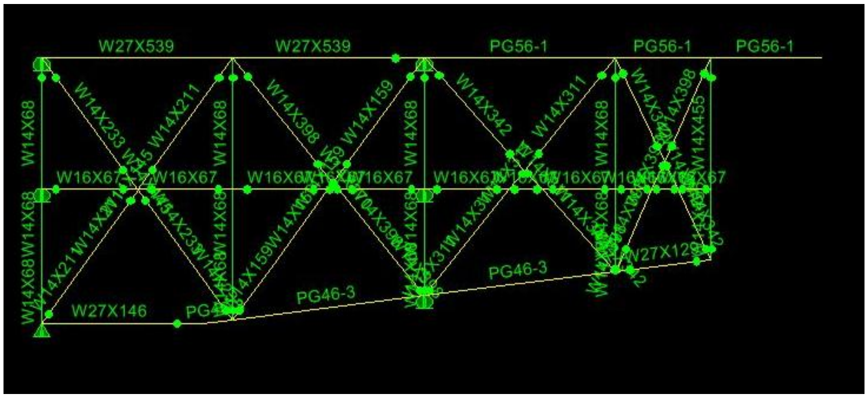


Figure 60: Truss X Constructability Concerns

In a similar fashion to Truss 0.9, Truss X weighs over 120 t and spans nearly 85' from gridline J to gridline N.2. Instead of being broken horizontally, however, Truss X is designed with pin connections at Level 4, meaning the top and bottom halves of the truss could be brought in separately (Figure 60). Also, if truck transportation is not an option, then the truss will have to be brought in by barge.

CUSTOM MEMBERS

PG56-1

As described above in the Custom Cross Sections section of this report, Figure 61 shows how the dimensions of the plates used for PG56-1 depart from the precedent plate sizes established by the engineers (found on the Plate Girder Schedule). In order to accommodate the immense loads and avoid interfering with the architectural envelope, a base dimension, B, of 24" is proposed, 4" wider than the current largest width of 20" for PG46-2. Bending requirements led to a 10"-thick, built-up flange. This does not necessarily conflict with the established design philosophy, as PG46-2 specifies (2) 4"-thick plates be welded together. PG56-1 could simply be a modification of that flange by welding an additional 1"-thick plate, or another arrangement could be established. Finally, for shear purposes the maximum established web thickness of 2" was increased to 2 ¼". Because of these provisions, increases were made to the labor, installation, and fabrication costs of the structural steel for the cost estimate.

	PG56-1		Capacities		
Lb	20	ft	ϕMn	41571	ft-k
D	56	in	ϕVn	3402	k
B	24	in	ϕTn	25245	k
tf	10	in	ϕPn	27541	k
tw	2.25	in			

BEAM MARK	BEAM DIMENSIONS			
	B	t_F	D	t_W
PG32.5	18"	4"	32.5"	2"
PG33-1	18"	4"	33"	2"
PG44-1	18"	4"	44"	2"
PG46-1	18"	2"	46"	1"
PG46-2	20"	(2) 4" **	46"	2"
PG46-3	18"	4"	46"	2"
PG72-1	16"	3"	72"	2"

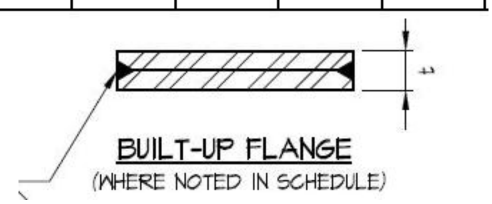


Figure 61: PG56-1 Precedence and Comparison (S-210)

24R-1

Unlike PG56-1, columns utilizing cross section 24R-1 face serious construction challenges. The section, detailed in Figure 62 below, specifies an unusually thick pipe wall of 1 ¾" with a 24" outer diameter. Attempts to contact the steel fabricator regarding this provision were unsuccessful.

The most risky specification for 24R-1, however, is the requirement for a concrete compressive strength of15,000 psi. Though this reflects an extremely high compressive strength, it is not unprecedented in the United States. The Portland Cement Association's page High-Strength Concrete (see References below) notes that compressive strengths as high as 19,000 psi have been used in large cities like Seattle. The use of 15,000 psi concrete will also likely involve more testing and regulation, as the highest-strength concrete is currently specified at 5,000 psi at the foundations.

In addition to the difficulty acquiring and ensuring such a high compressive strength, the presence of reinforcement and containment in a steel pipe make workability an issue. Extra care will need to be taken by the general contractor and subcontractors to ensure the concrete is properly placed and vibrated to ensure the capacity of the columns.

Pipe		
Do	24	in
t	1.75	in
Concrete		
f'c	15000	psi
no.	11	
n	16	
Capacity		
φPn	8272	k
φTn	8053	k
φMn	2754	ft-k

Figure 62: 24R-1 Summary

COMPARISON AND CONCLUSION

The proposed structural design contained in this report reflects a thorough investigation into the possibility of supporting the South-Eastern corner of AAM without the use of a column at 3-M.5. In order to both achieve structural adequacy under this constraint and minimize impacts to the architecture, the structural system becomes defined by its departure from common practice and precedent provisions. Figure 63 below shows that even a 50% increase in local weight, a 33% increase in cost, and radically high concrete strength specifications, AAM's proposed structural system fails in serviceability, unacceptably interferes with the window placement on levels 3 and 4, and causes serious logistical concerns during fabrication and construction. After assessing the impacts of the proposed structural system, it is recommended that AAM be constructed under the current design and specifications put forth by Robert Silman Associates.

Structural Concerns	Current Design	Proposed Design
Remove Column 3-M.5	NO	YES
No. of Steps in Load Path	2	4
Max. Element Weight	45.1 t	121.6 t
Overall Weight	199.5 t	297.1 t
Max. Pile Group	2	5
No. of Custom Sections	10	12
Columns Max.O.D.	22"	24"
Max. f'c	5,000	15,000
Max. Total Deflection	-	-7.57 in
Acceptable Deflections	YES	NO
Architectural Concerns	Current Design	Proposed Design
Gallery Interference	NO	Truss N.2
Wall X Interference	NO	Truss X
Remove Truss L	NO	YES
Maintain Web Openings	YES	YES
Maintain Building Envelope	YES	YES
Construction Concerns	Current Design	Proposed Design
No. of Long Trusses	1	2
Cost of Superstructure	$3,928,000	$5,946,000
Cost of Foundations	$238,000	335,000
Total Structural Cost	$4,166,000	$6,281,000
Total Difference		$2,115,000

Figure 63: Comparative Summary

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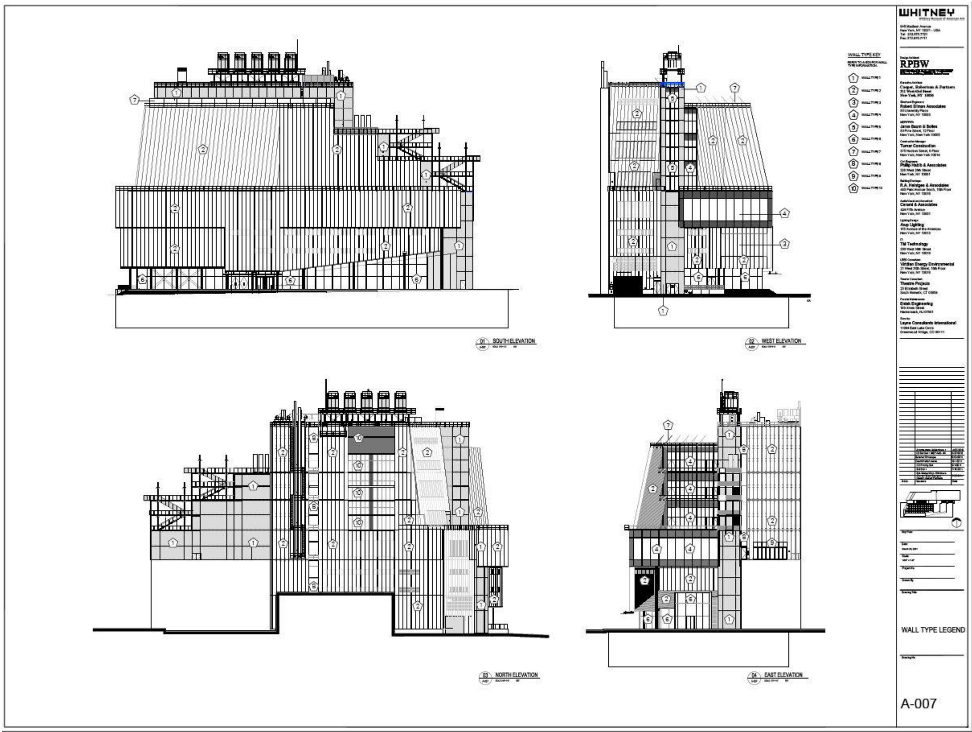
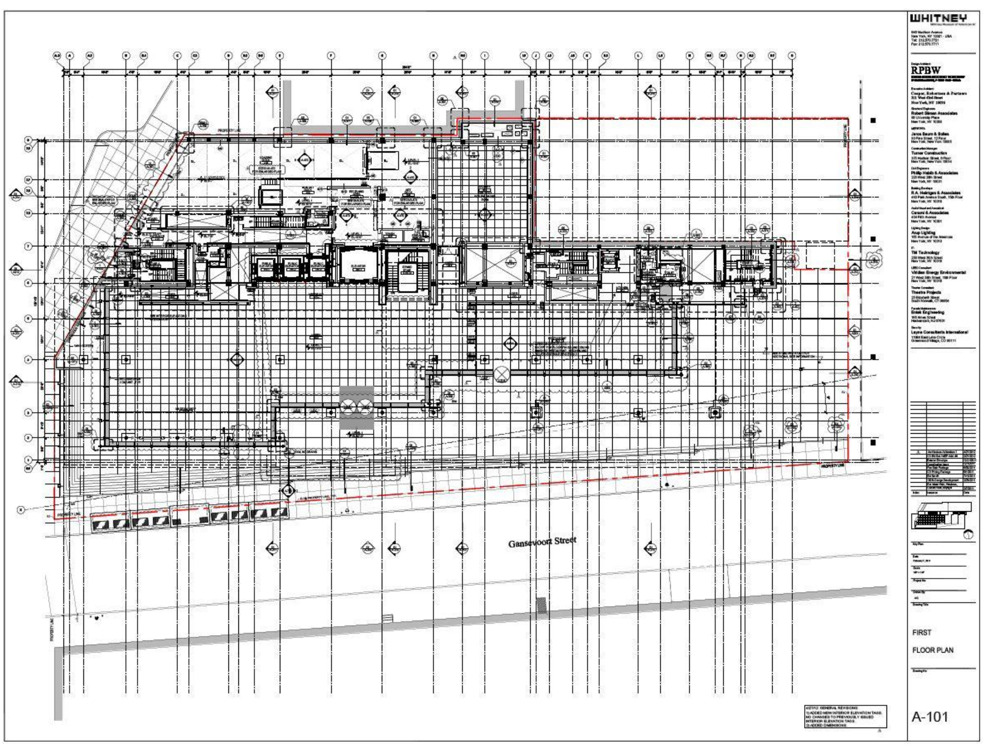
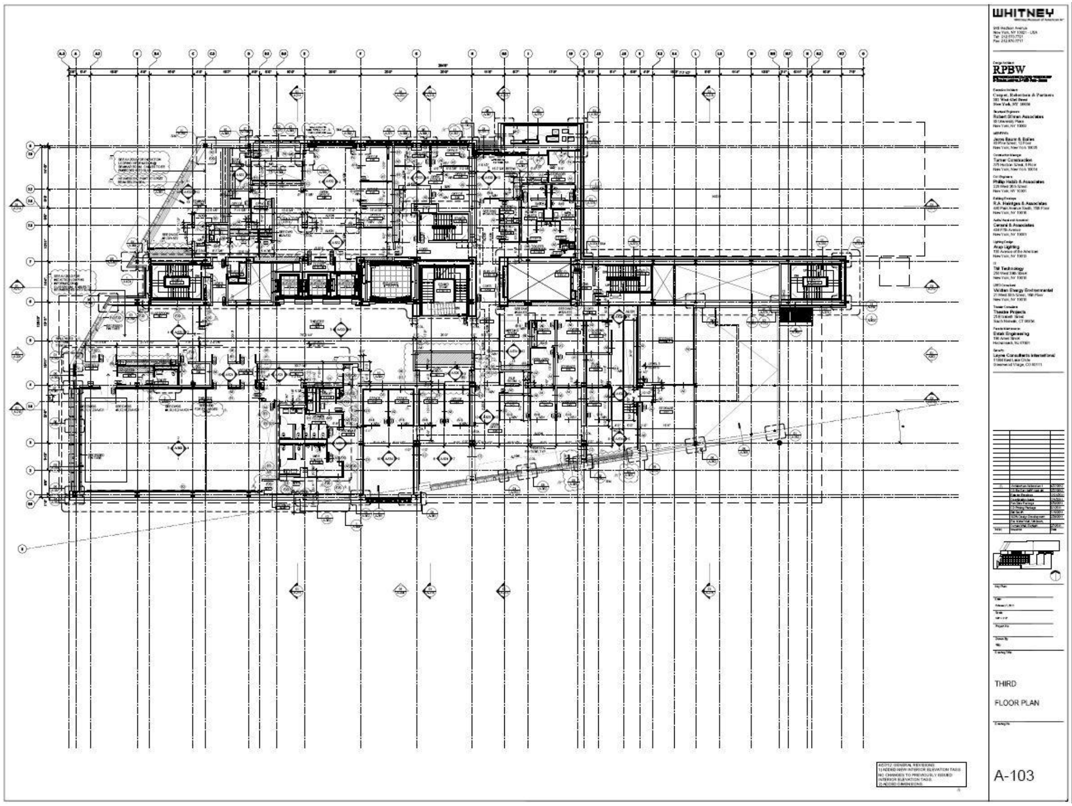
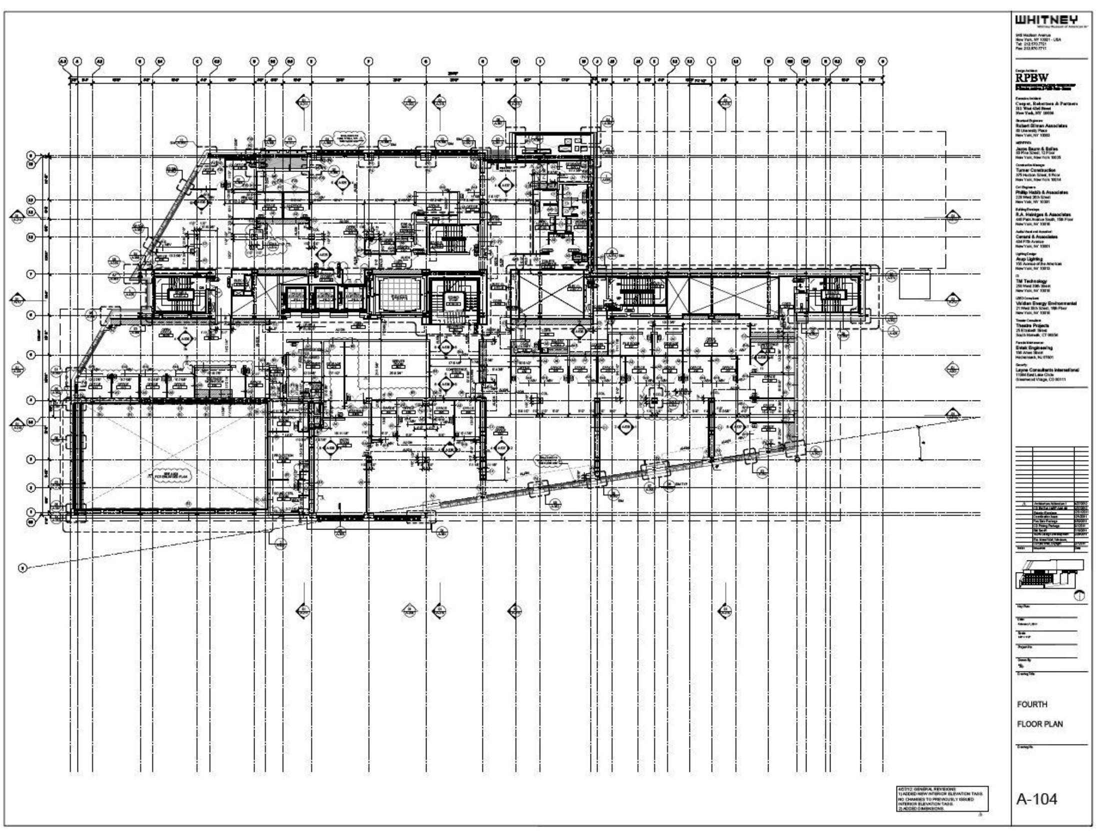
APPENDIX A: TECHNICAL DRAWINGS

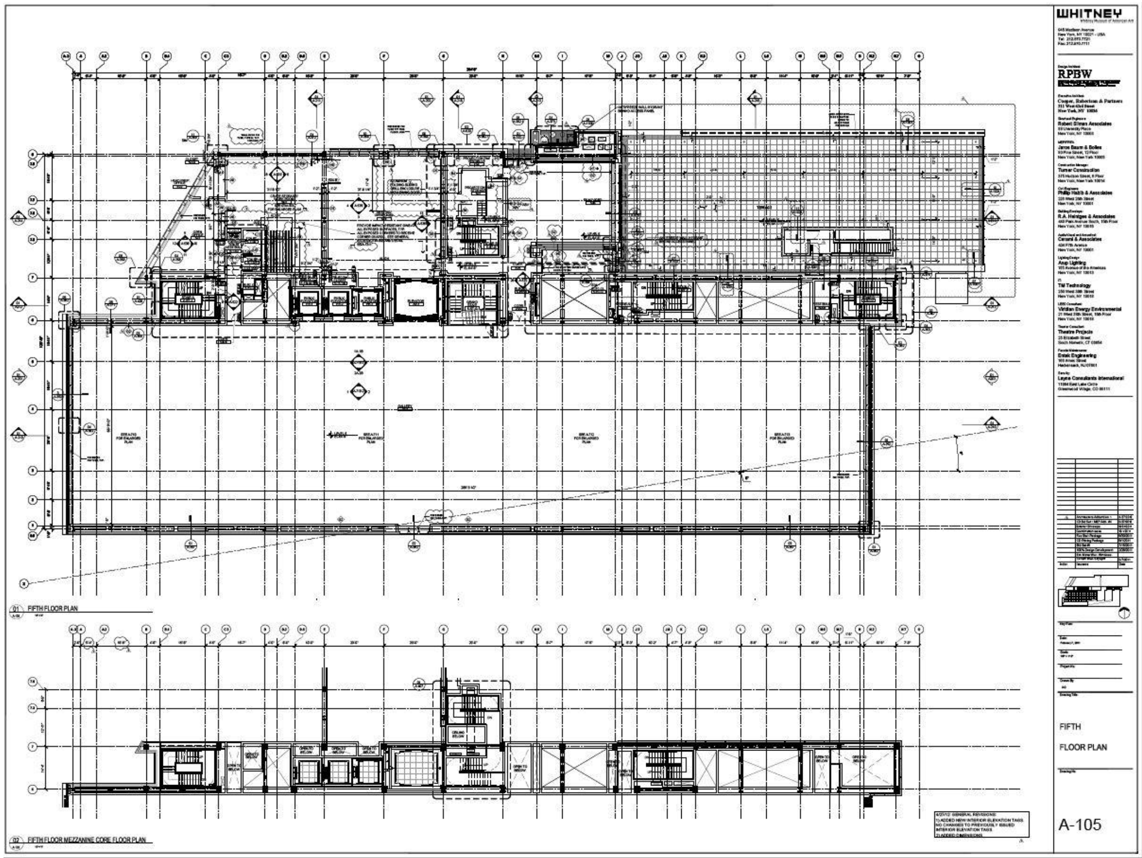
ARCHITECTURAL DRAWINGS

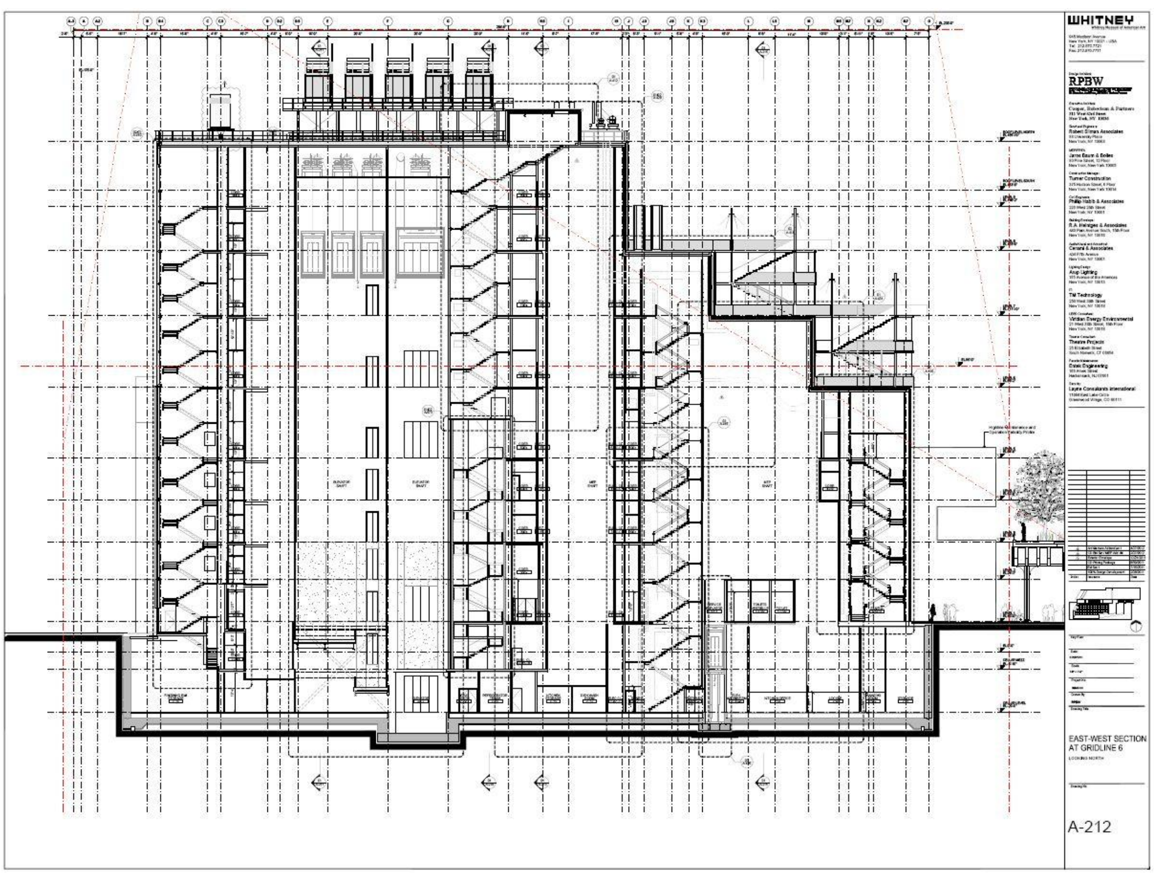


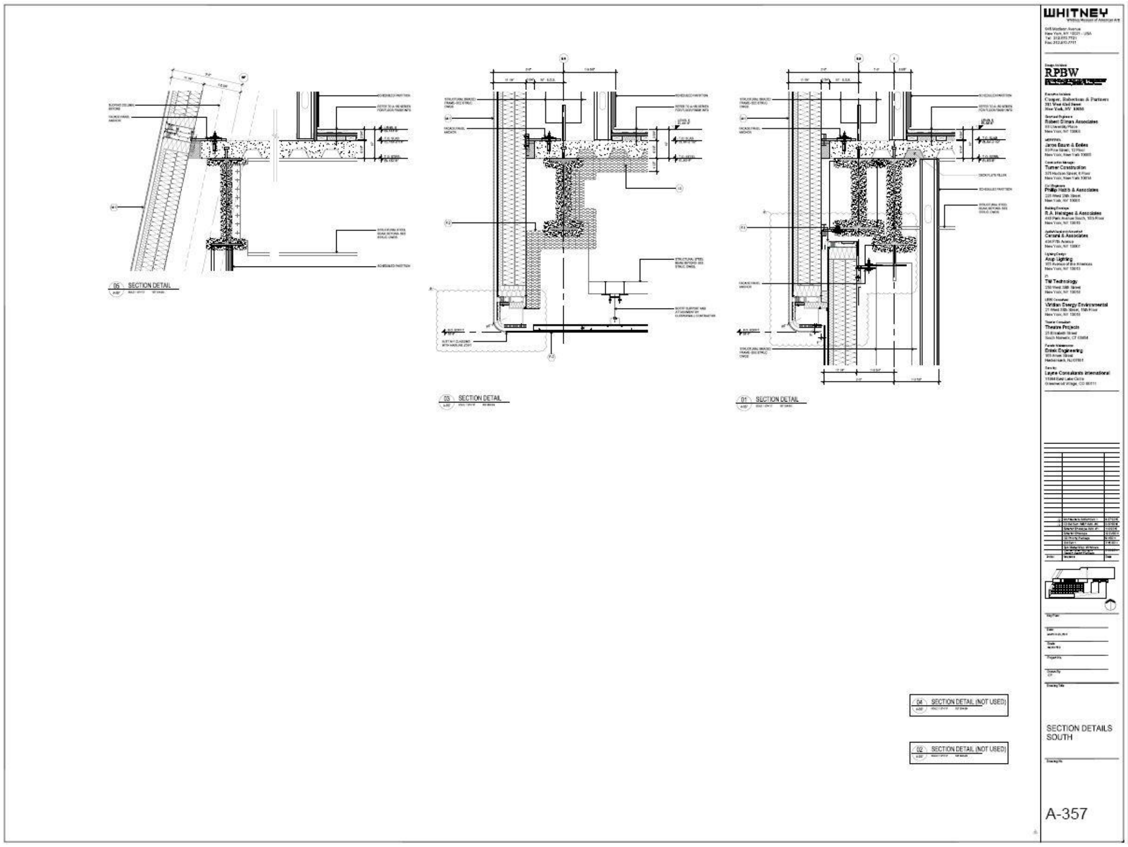


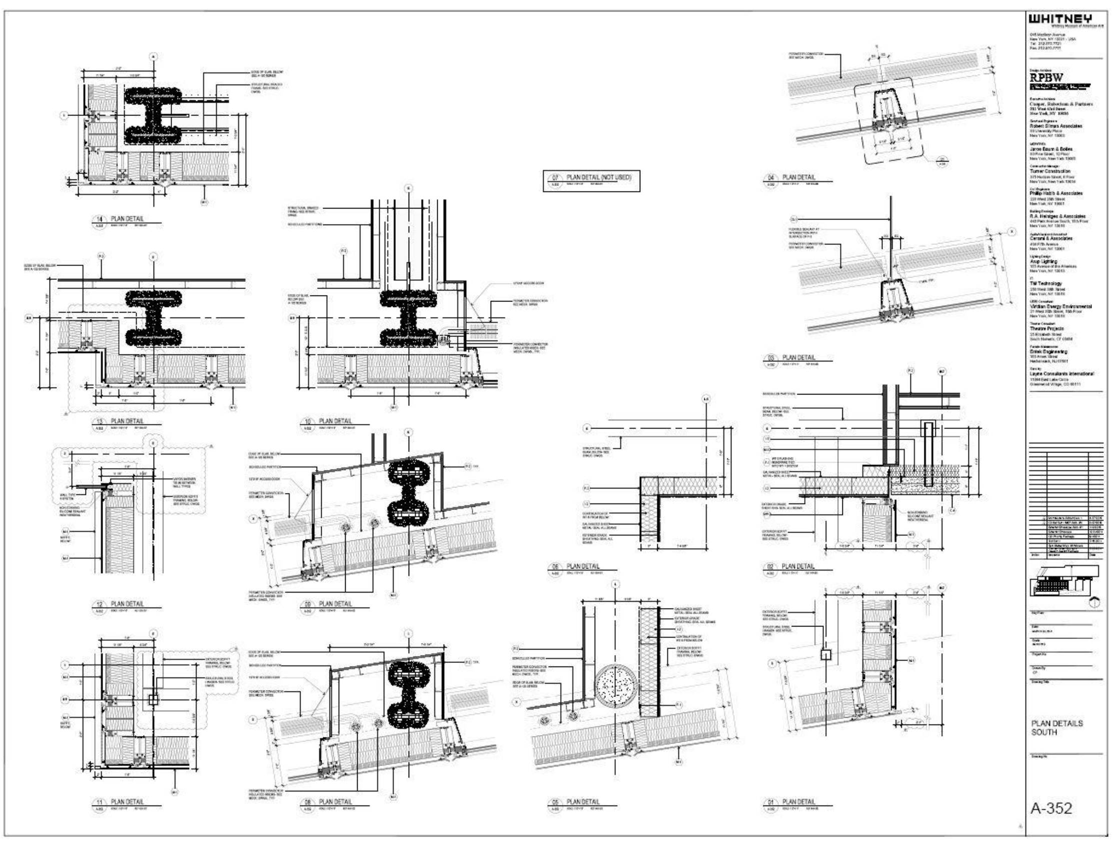

WHITNEY
RPBW
THIRD
FLOOR PLAN
A-103

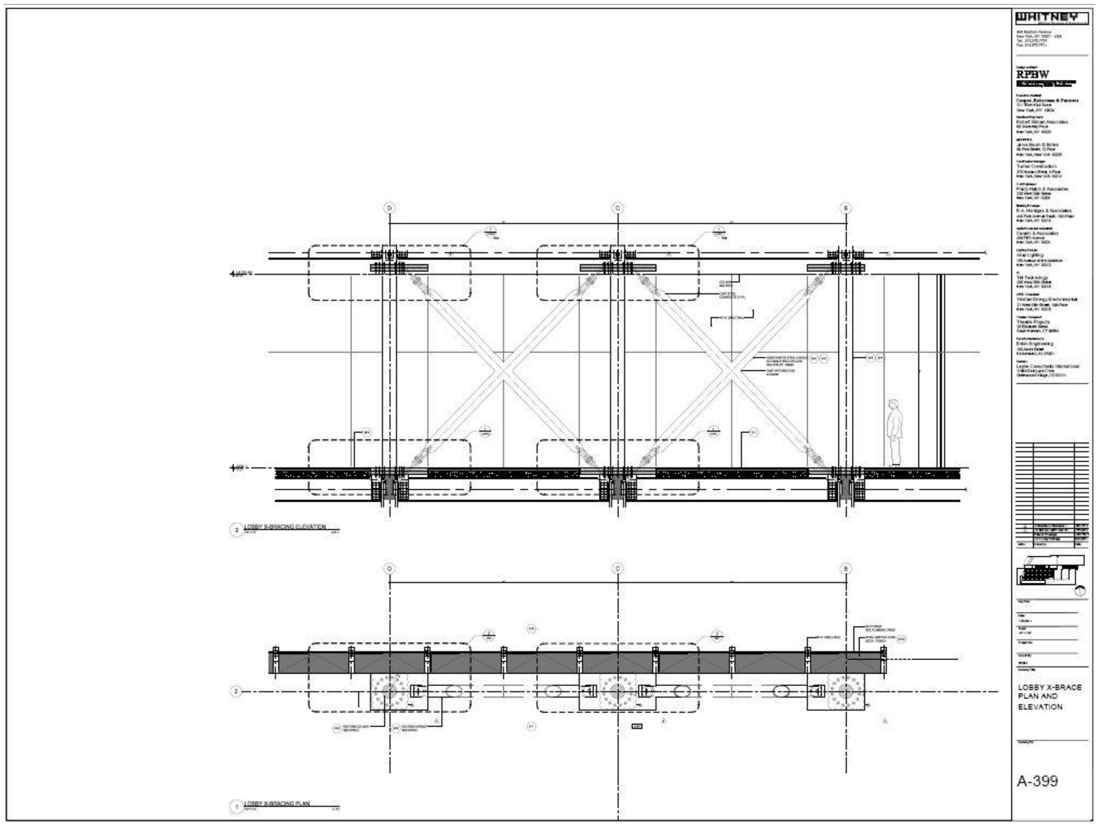

WHITNEY
RPBW
FOURTH
FLOOR PLAN
A-104


WHITNEY
RPBW
FIFTH FLOOR PLAN
A-105
01 FIFTH FLOOR PLAN
02 FIFTH FLOOR MEZZANINE CORE FLOOR PLAN

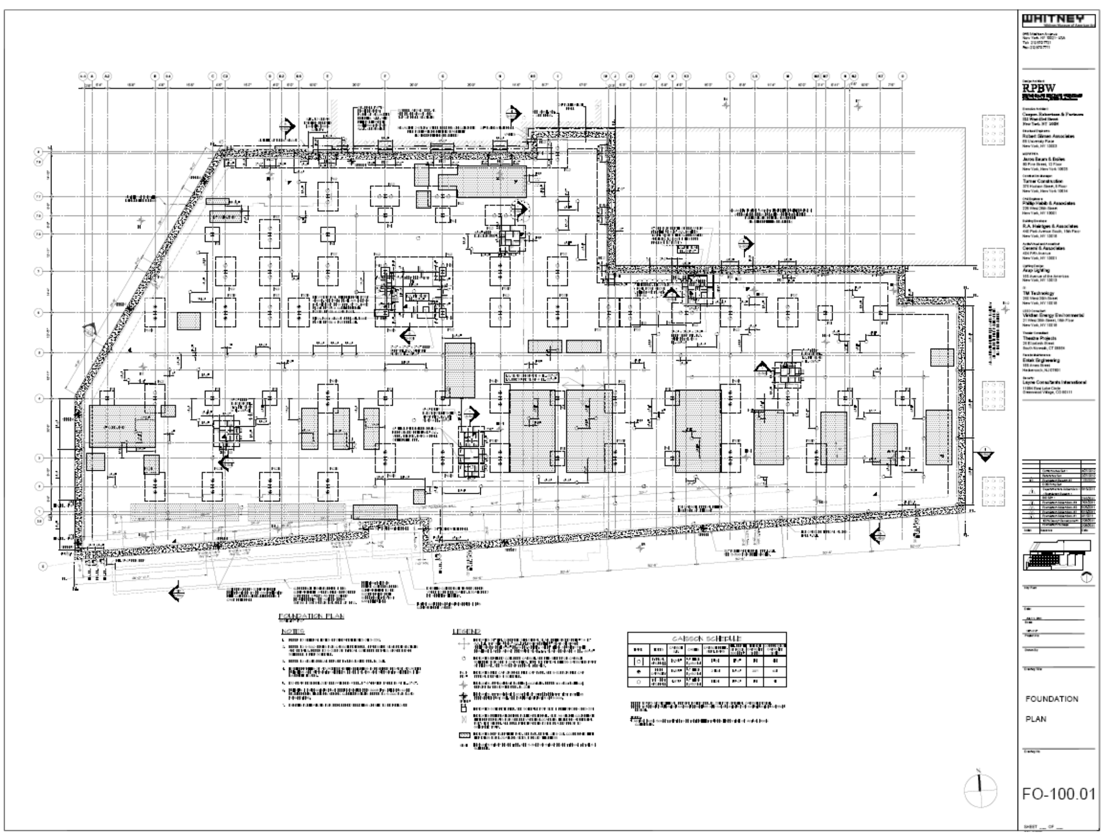

WHITNEY
RPBW
EAST-WEST SECTION AT GRIDLINE 6
A-212


WHITNEY
RPBW
SECTION DETAIL
SECTION DETAIL (NOT USED)
SECTION DETAILS
SOUTH
A-357

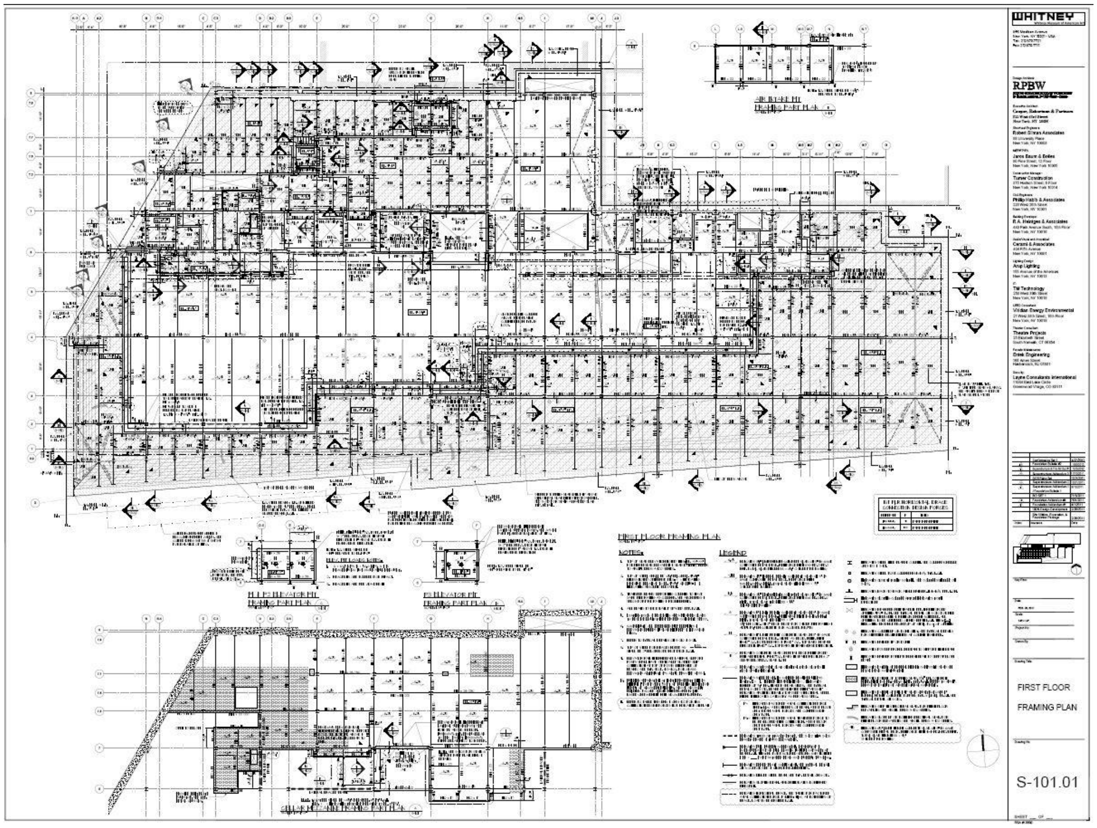

WHITNEY
RPBW
PLAN DETAIL
PLAN DETAIL (NOT USED)
PLAN DETAILS
SOUTH
A-352

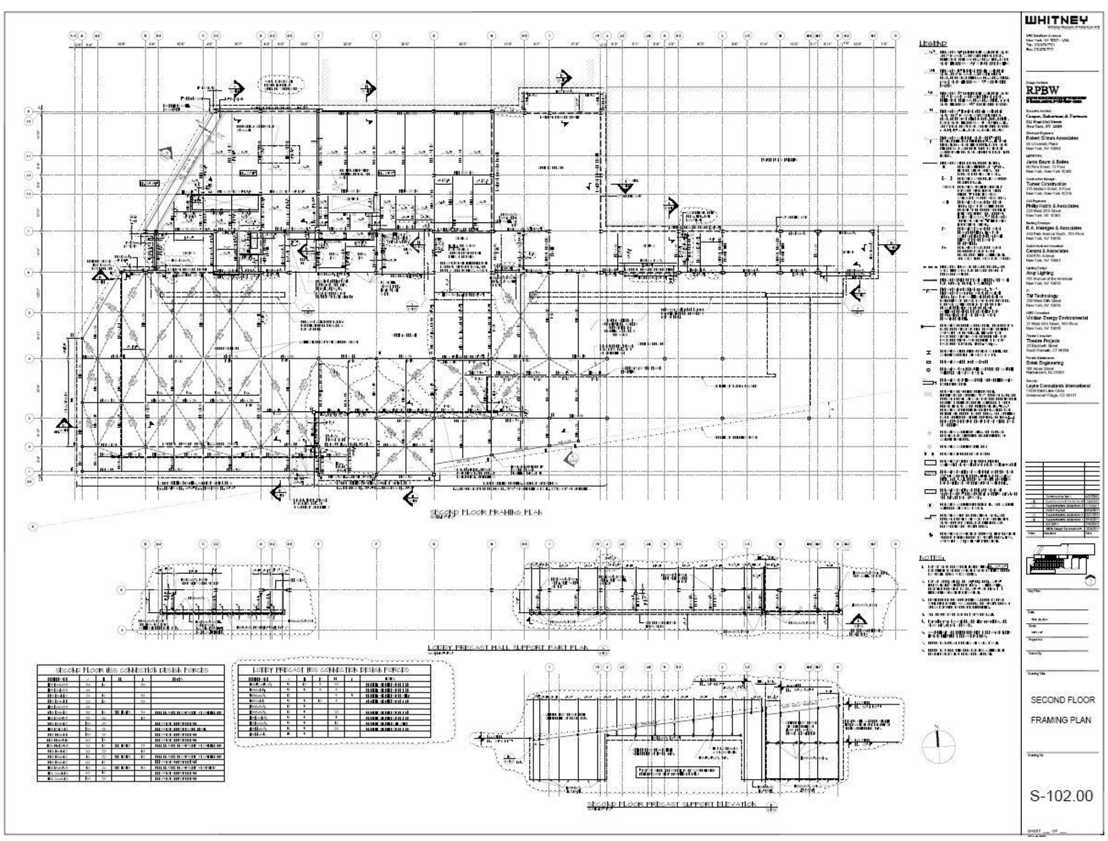


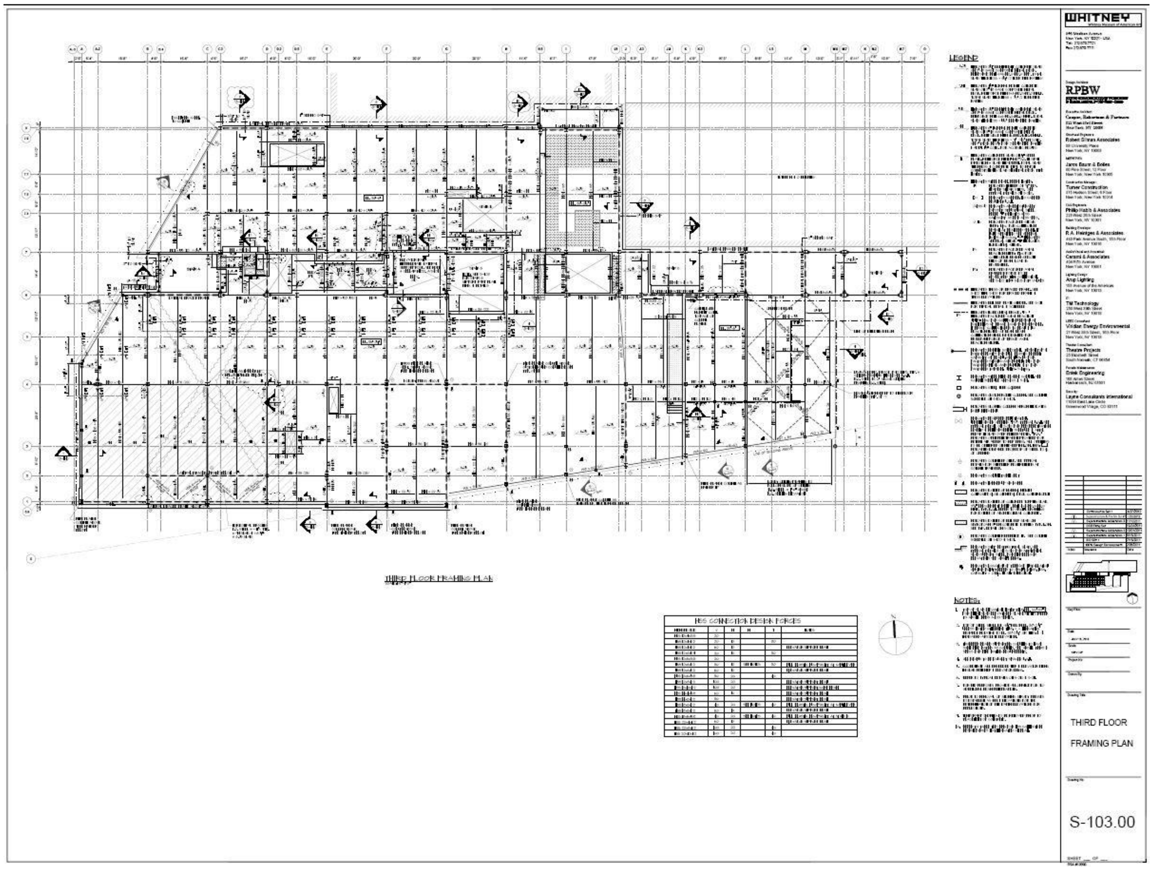
FOUNDATION DRAWINGS

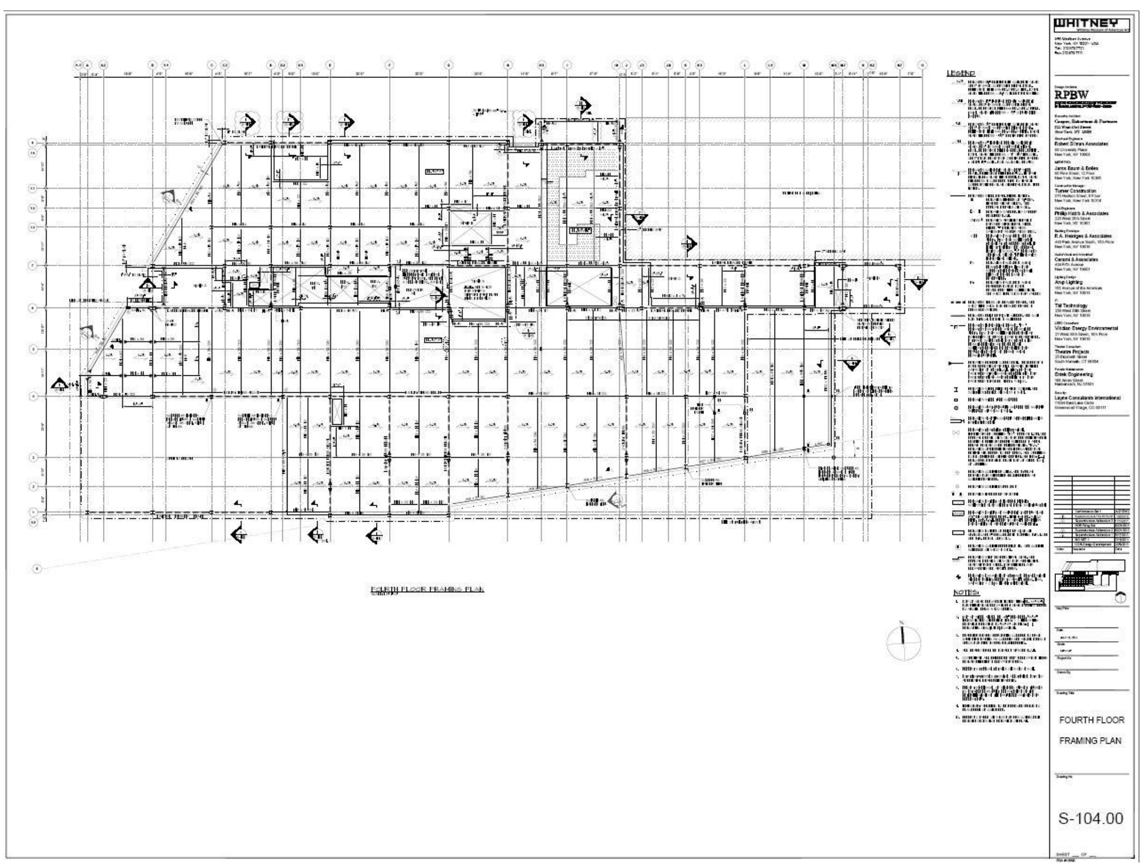


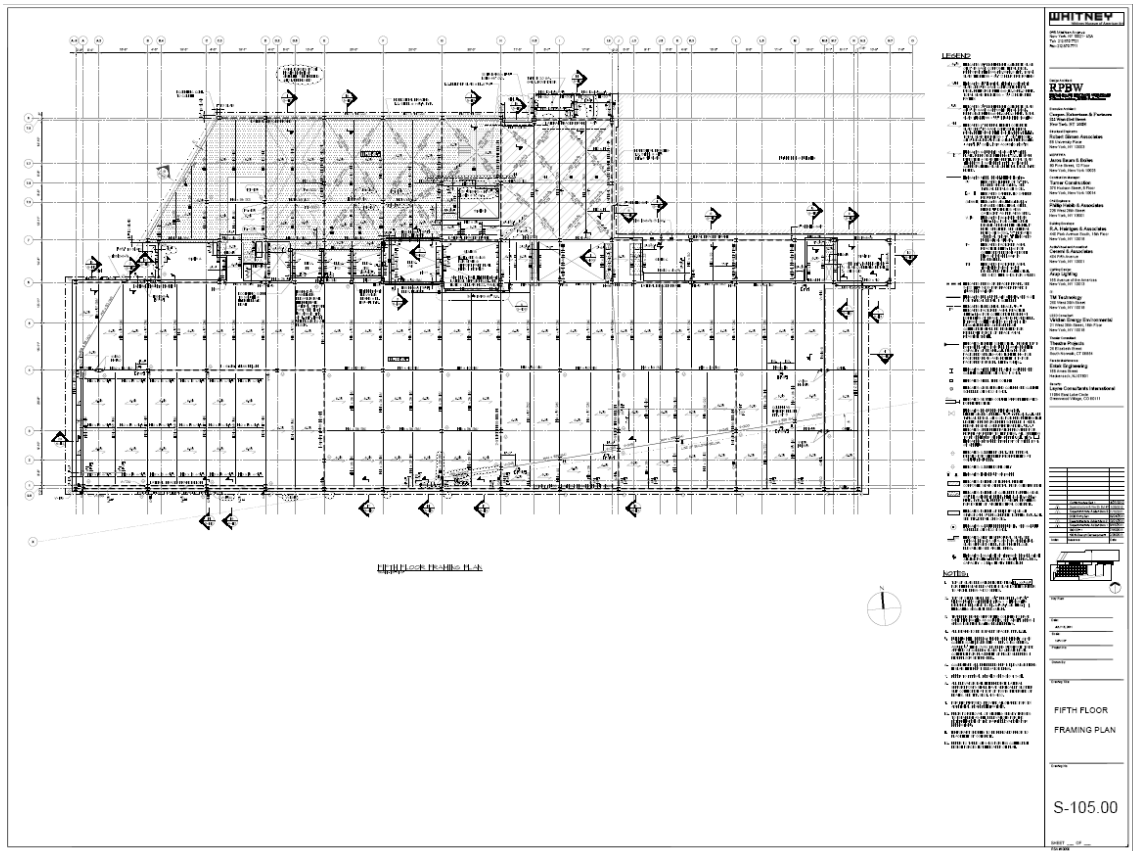
STRUCTURAL DRAWINGS

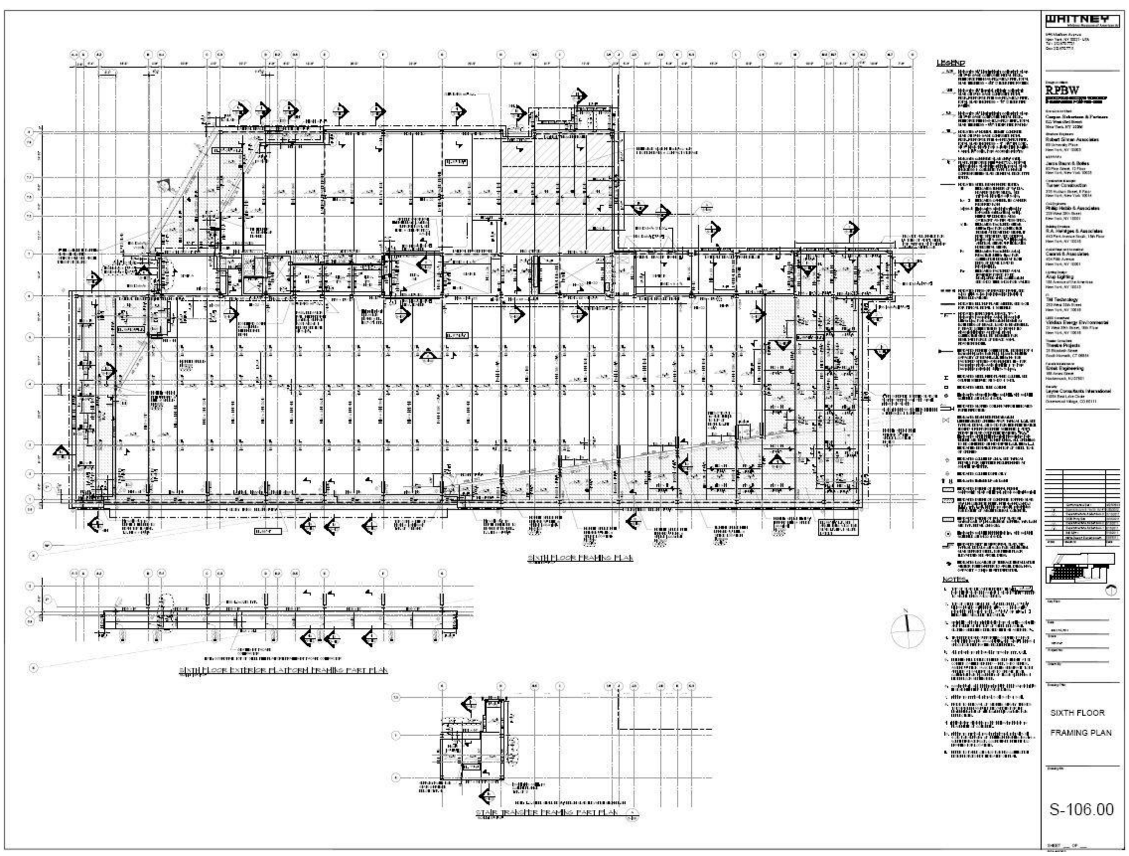


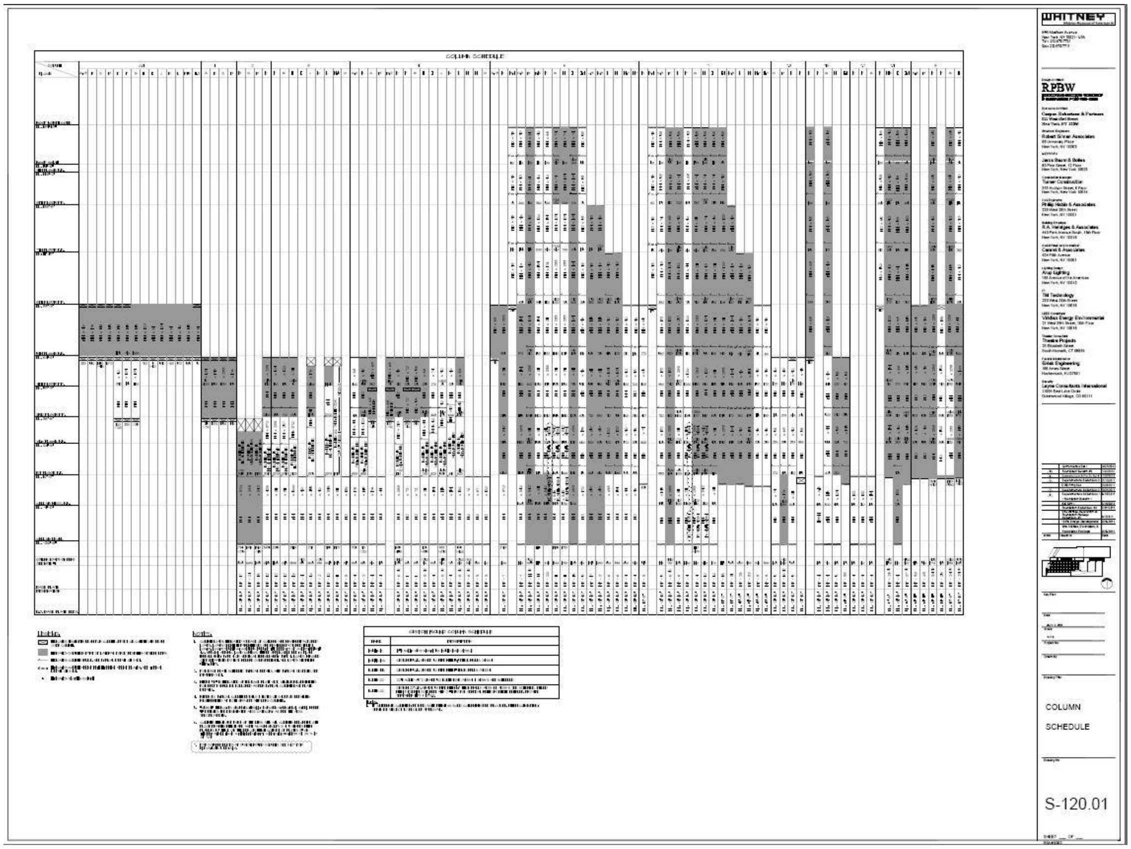


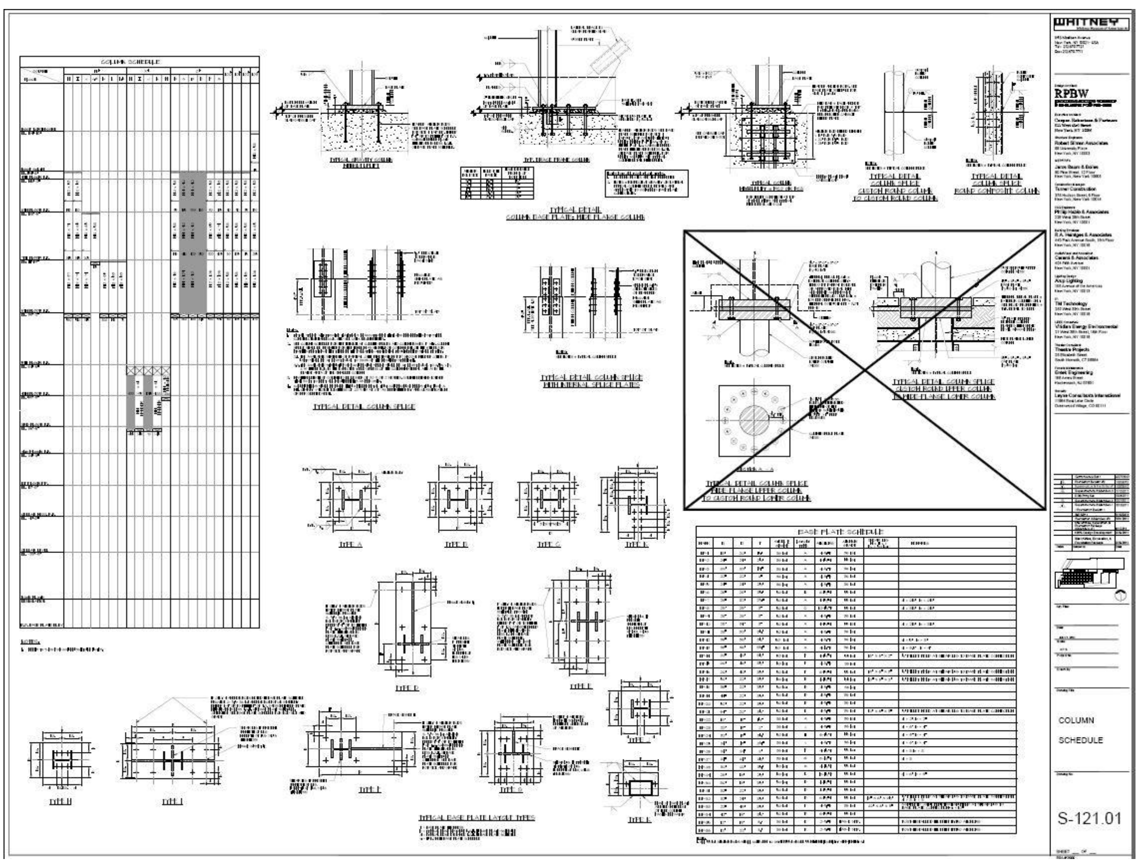

WHITNEY
RPBW
THIRD FLOOR
FRAMING PLAN
S-103.00

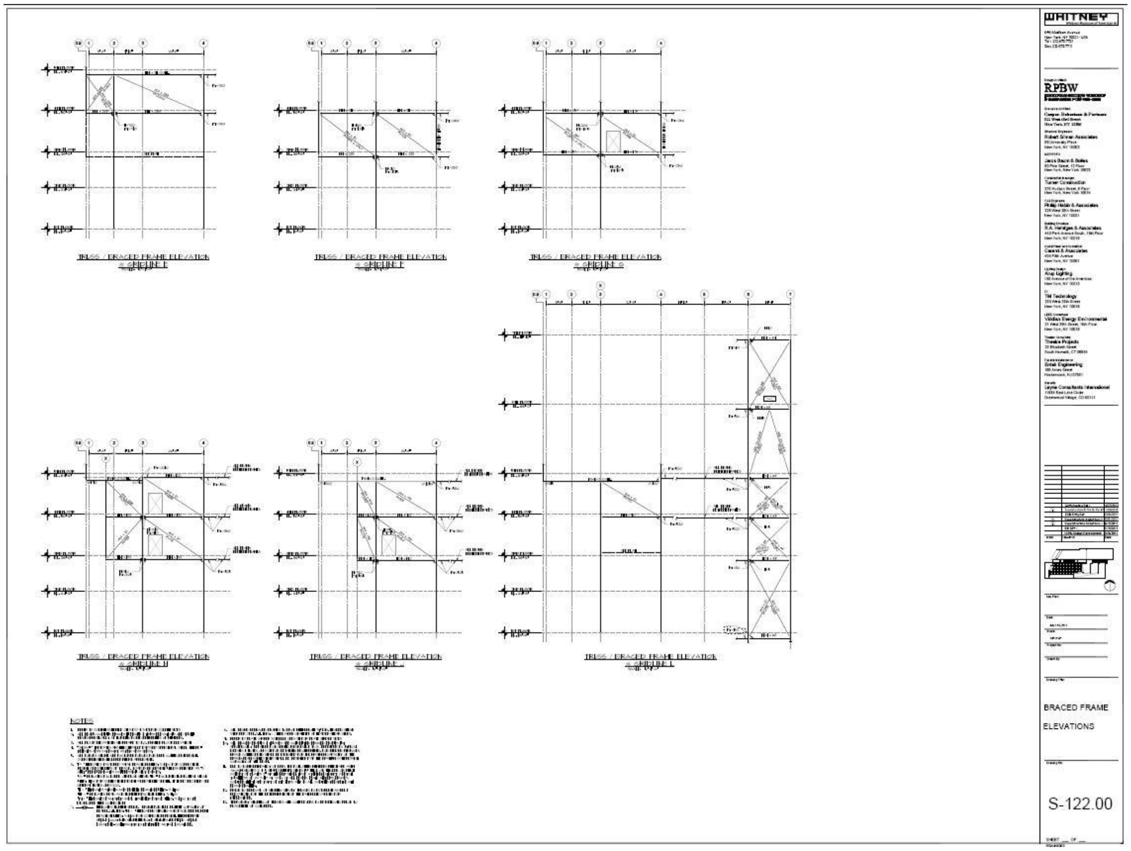

WHITNEY
RPBW
FOURTH FLOOR
FRAMING PLAN
S-104.00

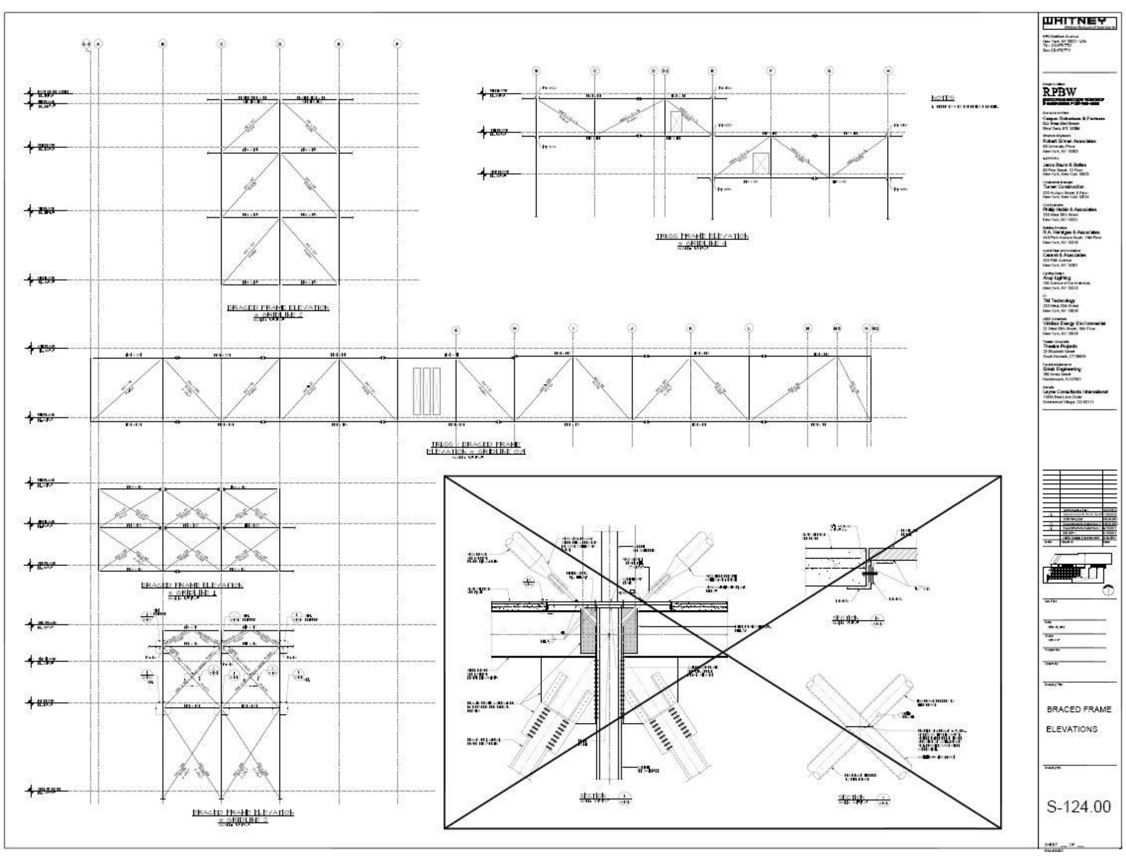

WHITNEY
RPBW
FIFTH FLOOR
FRAMING PLAN
S-105.00

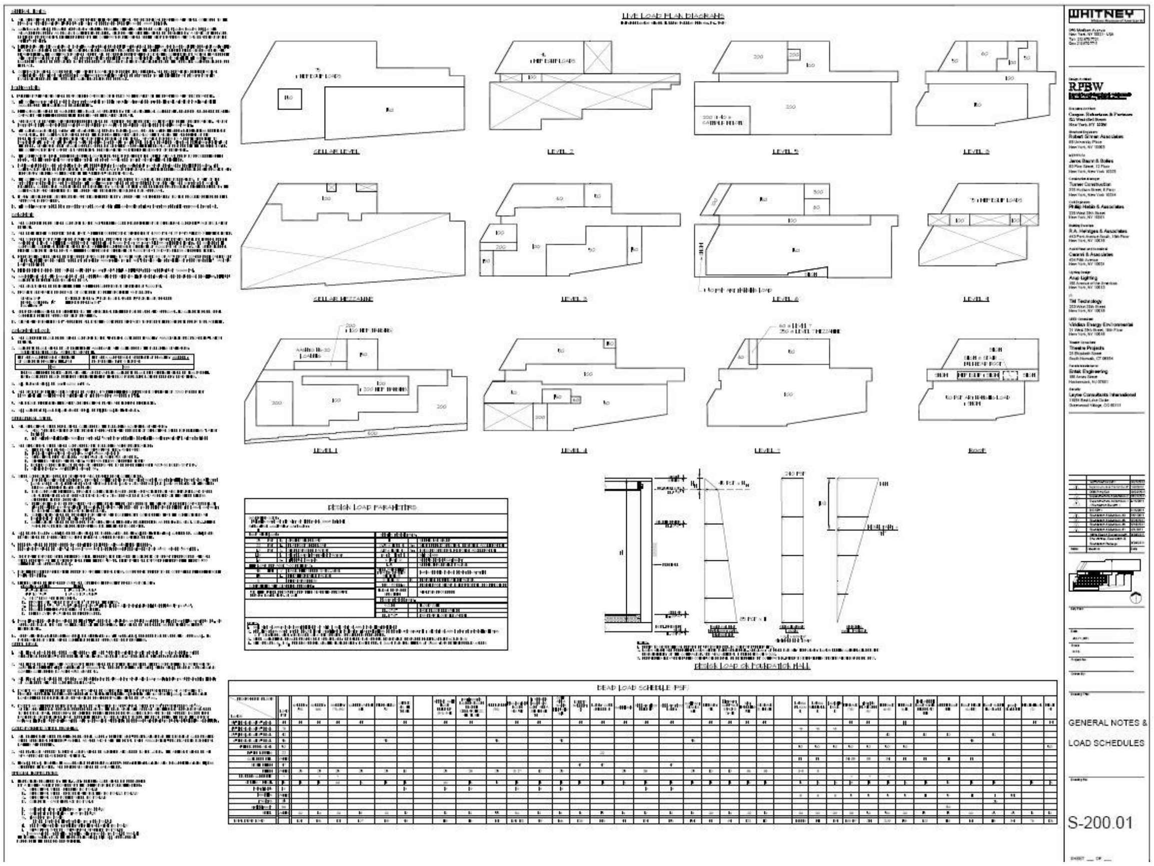

WHITNEY
RPBW
SIXTH FLOOR
FRAMING PLAN
S-106.00

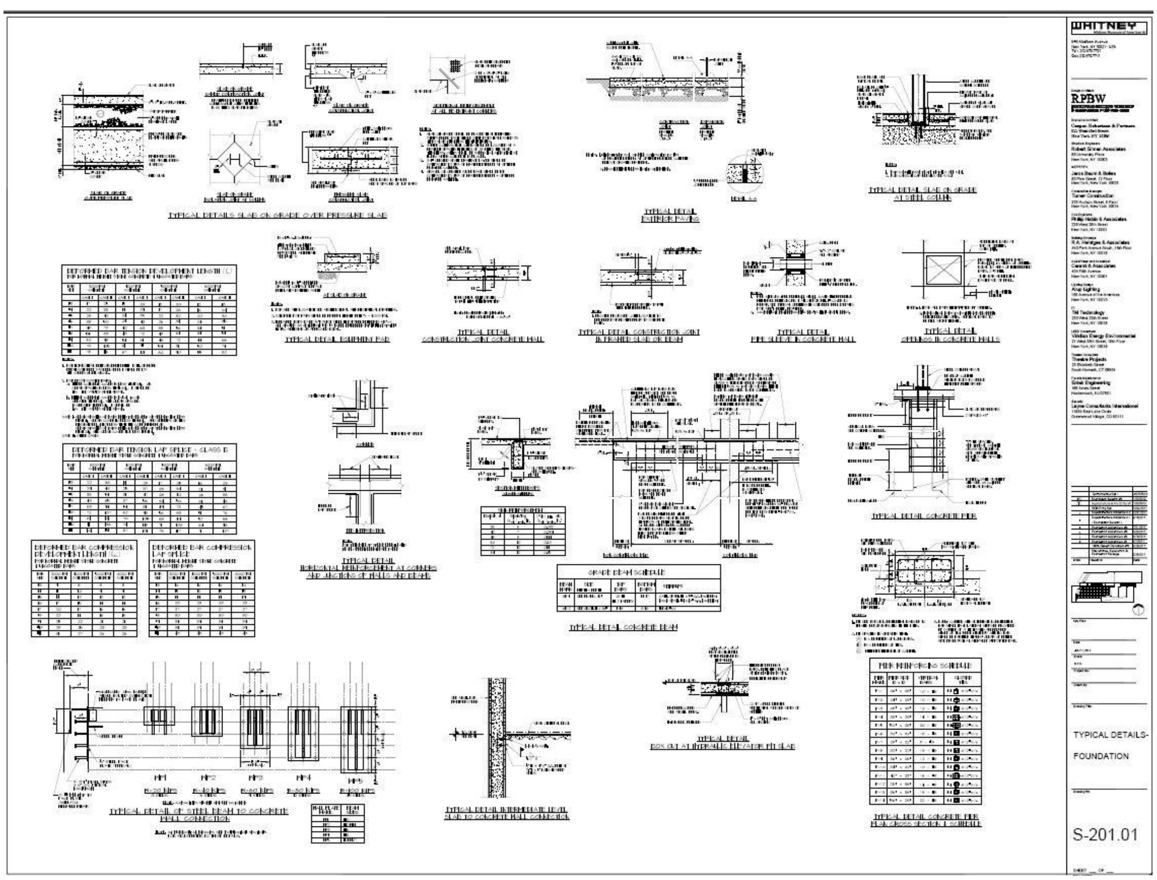

COLUMN SCHEDULE
WHITNEY
RPBW
COLUMN
SCHEDULE
S-120.01

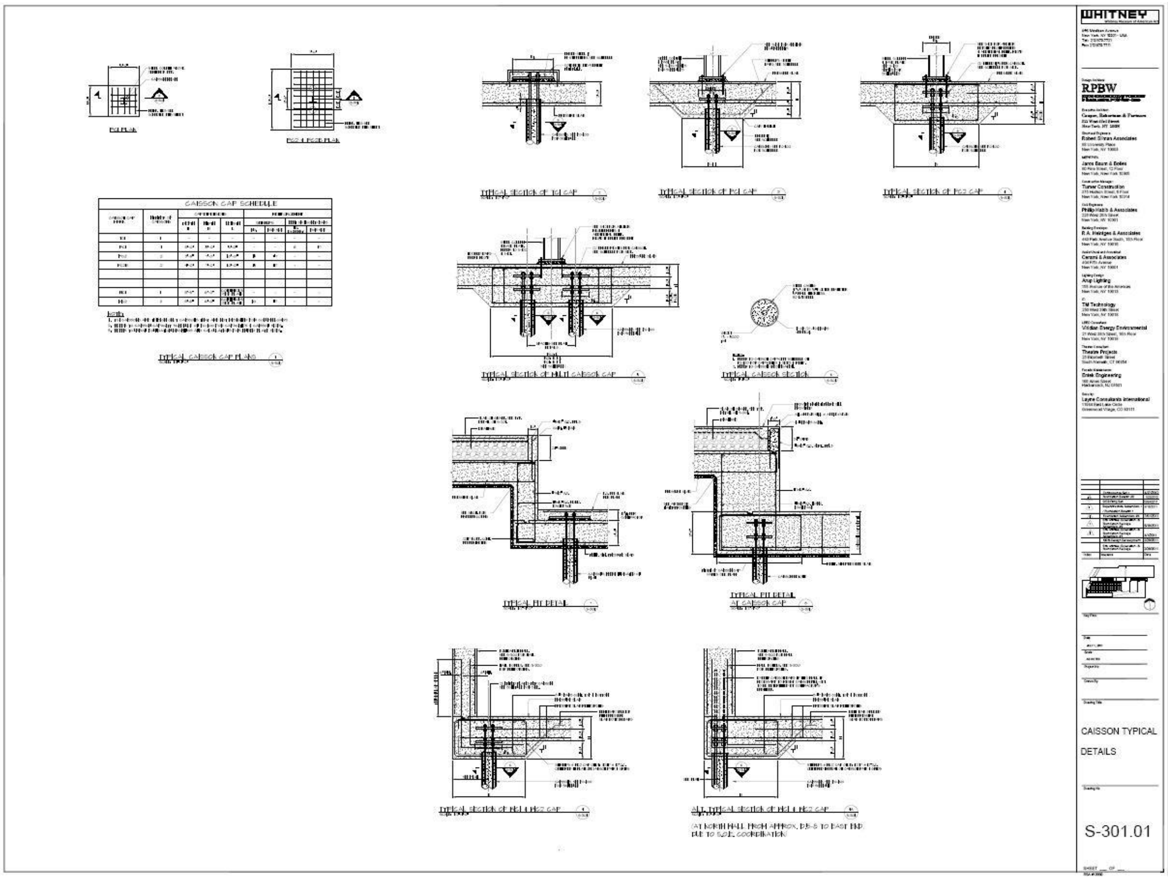

WHITNEY
RPBW
BASE PLATE SCHEDULE
COLUMN
SCHEDULE
S-121.01


WHITNEY
RPBW
BRACED FRAME
ELEVATIONS
S-122.00

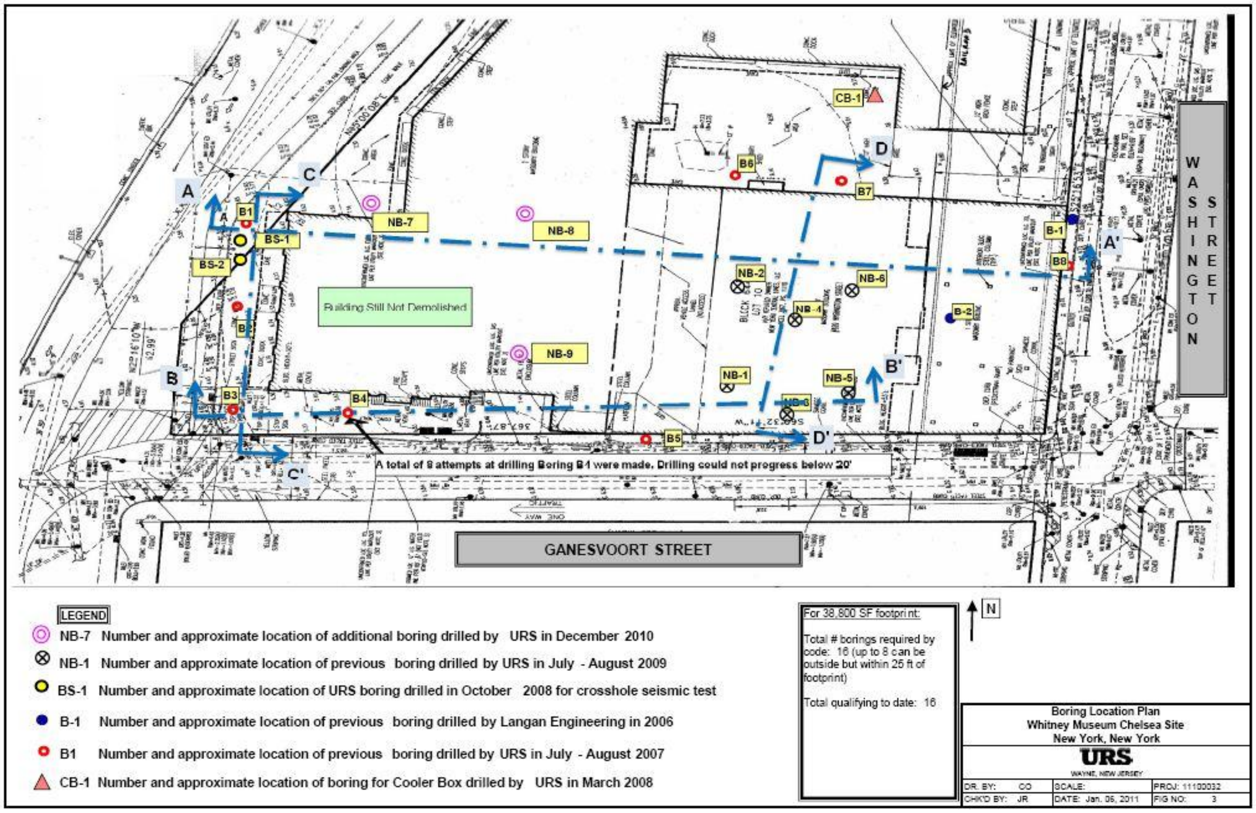

WHITNEY
RPBW
BRACED FRAME
ELEVATIONS
S-124.00

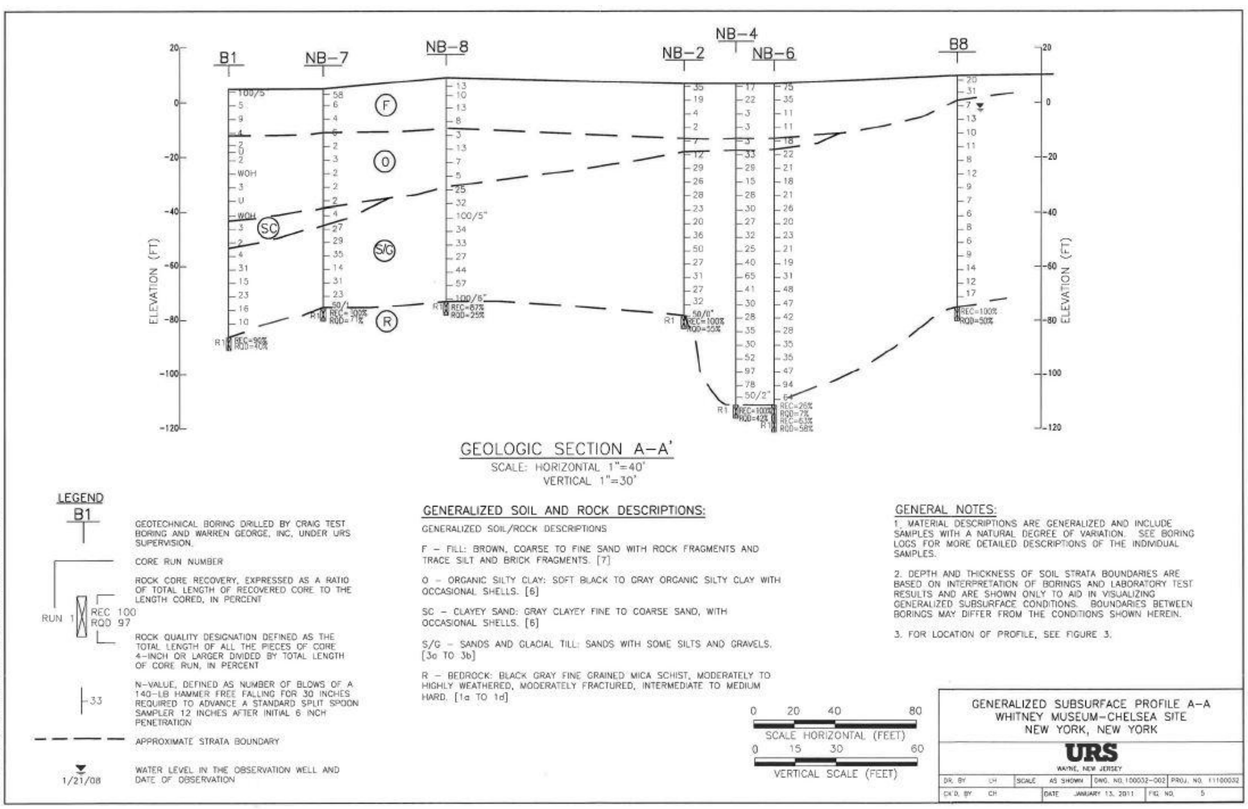

WHITNEY
RPBW
GENERAL NOTES &
LOAD SCHEDULES
S-200.01


WHITNEY
RPBW
TYPICAL DETAILS-
FOUNDATION
S-201.01


WHITNEY
RPBW
CAISSON TYPICAL DETAILS
S-301.01

GEOTECHNICAL DOCUMENTS





APPENDIX B: INTRODUCTION CALCULATIONS

BUILDING DEAD LOAD CALCULATIONS

Total Dead Load Calculations							
Level		Type		SQ in	SQ ft	Wt/SFt	Wt/flr (k)
Roof N		31		431080	2994	102	305.35
483		32		220480	1531	116	177.61
Roof S		33		154530	1073	161	172.77
628		34		128723	894	118	105.48
		35		598722	4158	84	349.25
Level 9		16		96701	672	99	66.48
500		37		495578	3442	126	433.63
Level 8		3		877728	6095	121	737.54
2002		6		119746	832	98	81.49
		7		225656	1567	118	184.91
		8		271800	1888	116	218.95
		16		415454	2885	99	285.62
		23		75238	522	112	58.52
		27		334730	2325	187	434.68
Level 7		3		1498650	10407	121	1259.28
2342		6		535436	3718	98	364.39
		8		69584	483	116	56.05
		12		123266	856	98	83.89
		16		40078	278	99	27.55
		20		83450	580	94	54.47
		21		103600	719	84	60.43
		27		335340	2329	187	435.48
Level 6		2		1897600	13178	136	1792.18
4188		4		460080	3195	107	341.87
		12		49612	345	98	33.76
		13		79600	553	166	91.76
		16		40078	278	99	27.55
		19		103640	720	154	110.84
		28		156520	1087	214	232.61
		29		988974	6868	203	1394.18
		30		149084	1035	158	163.58

Total Dead Load Calculations							
Level		Type		SQ in	SQ ft	Wt/SFt	Wt/flr (k)
Level 5		1		2830400	19656	109	2142.46
2915		5		172000	1194	158	188.72
		11		84200	585	133	77.77
		16		40078	278	99	27.55
		22		564400	3919	122	478.17
Level 4		6		2801124	19452	98	1906.32
2589		8		90400	628	116	72.82
		10		93800	651	109	71.00
		12		98340	683	98	66.93
		16		591510	4108	99	406.66
		23		84280	585	112	65.55
Level 3		6		949600	6594	98	646.26
2155		7		93200	647	118	76.37
		8		205328	1426	116	165.40
		9		458320	3183	181	576.08
		12		26000	181	98	17.69
		16		704038	4889	99	484.03
		23		243288	1690	112	189.22
Level 2		16		265600	1844	99	182.60
419		36		448300	3113	76	236.60
Level 1		14		1434000	9958	126	1254.75
4863		15		371600	2581	148	381.92
		16		222000	1542	99	152.63
		24		1222800	8492	186	1579.45
		25		384200	2668	191	509.60
		26		839400	5829	169	985.13
					Totals		
					Sq. ft		Weight (k)
					183882		23084

DL Schedule Summary (S-200.01)			
Floor Type	DL PSF	Floor Type	DL PSF
1	109	21	84
2	136	22	122
3	121	23	112
4	107	24	186
5	158	25	191
6	98	26	169
7	118	27	187
8	116	28	214
9	181	29	203
10	109	30	158
11	133	31	102
12	98	32	116
13	166	33	161
14	126	34	118
15	148	35	84
16	99	36	76
17	124	37	126
18	135		
19	154		
20	94		

SNOW LOAD CALCULATIONS

ASCE 7-05
SNOW LOADS
1/1

GROUND SNOW LOAD : 7.2

P_g = 25 PSF (FIGURE 7-1) – verified by structural notes, city BC

FLAT ROOF SNOW LOADS 7.3

$p_f = 0.7\, C_e C_t I p_g$, $p_{f\,min} = \begin{cases} I\, p_g & (p_g \le 20\text{ PSF}) \\ 20 I & (p_g > 20\text{ PSF}) \end{cases}$

EQN 7-1

EXPOSURE FACTOR 7.3.1

TERRAIN CATEGORY "C" (WIND CALCS p1, ASCE 7-05 6.5.6)
PARTIALLY EXPOSED ROOF

$C_e = 1.0$ (TABLE 7-2) – verified by structural notes

THERMAL FACTOR 7.3.2

$C_t = 1.0$ (TABLE 7-3) – verified by structural notes

IMPORTANCE FACTOR 7.3.3
OCCUPANCY III
$I = 1.15$ – verified by structural notes

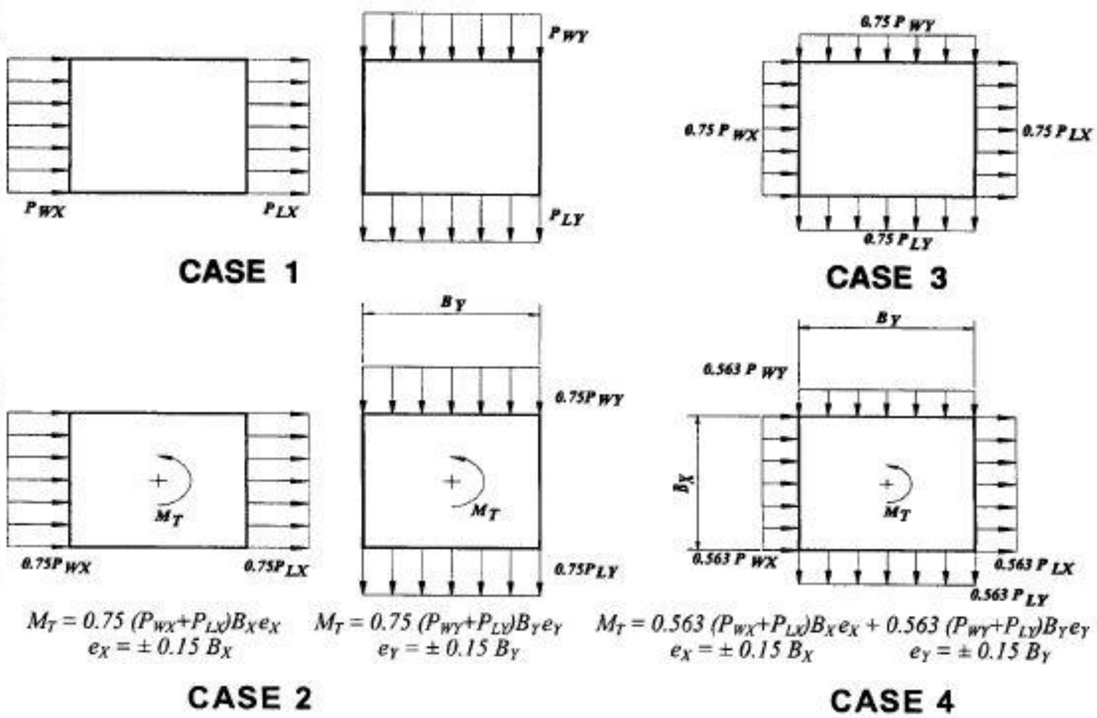
$p_f = 0.7 \cdot 1.0 \cdot 1.0 \cdot 1.15 \cdot 25 = 21$ PSF

$p_{f,\,min} = 20 I = 20 \cdot 1.15 = 23$ PSF * CONTROLS

* DRAWINGS DENOTE 22 PSF FOR p_f. LIKELY ASSUME LL WILL CONTROL

WIND LOAD CALCULATIONS

Main Wind Force Resisting System–Method 2		All Heights
Figure 6-9	Design Wind Load Cases	



Case 1. Full design wind pressure acting on the projected area perpendicular to each principal axis of the structure, considered separately along each principal axis.

Case 2. Three quarters of the design wind pressure acting on the projected area perpendicular to each principal axis of the structure in conjunction with a torsional moment as shown, considered separately for each principal axis.

Case 3. Wind loading as defined in Case 1, but considered to act simultaneously at 75% of the specified value.

Case 4. Wind loading as defined in Case 2, but considered to act simultaneously at 75% of the specified value.

Notes:

1. Design wind pressures for windward and leeward faces shall be determined in accordance with the provisions of 6.5.12.2.1 and 6.5.12.2.3 as applicable for building of all heights.
2. Diagrams show plan views of building.
3. Notation:

   P_{WX}, P_{WY}: Windward face design pressure acting in the x, y principal axis, respectively.
   P_{LX}, P_{LY}: Leeward face design pressure acting in the x, y principal axis, respectively.
   $e\ (e_X, e_Y)$: Eccentricity for the x, y principal axis of the structure, respectively.
   M_T: Torsional moment per unit height acting about a vertical axis of the building.

Wall Pressures								
E-W				qGfCp	qiGCpi	WW	LW	Pressure
Level	ht	Kz	qz	Cp	-0.55	0.8	-0.3	PSF
RN	160	1.39	33.41	23.793	-18.37	42.17	-7.72	49.89
RS	142	1.36	32.68	23.279	-17.98	41.26	-7.72	48.98
9	140	1.36	32.68	23.279	-17.98	41.26	-7.72	48.98
8	124	1.32	31.72	22.595	-17.45	40.04	-7.72	47.76
7	102	1.26	30.28	21.568	-16.65	38.22	-7.72	45.94
6	78	1.21	29.08	20.712	-15.99	36.71	-7.72	44.43

Equivalent Point Loads					
E-W	Pressure	hi	Dist Ld	Bx	Px
Level	PSF	ft	plf	ft	k
RN	49.89	10	498.9	55.8	27.9
RS	48.98	9	1348.3	53.8	72.5
9	48.98	18	1789.1	55.8	99.9
8	47.76	19	1964.3	113.7	223.3
7	45.94	23	1589.9	118.3	188.1
Vb =	611.6	k	Mover =	27902.3	kft

Wall Pressures								
N-S				qGfCp	qiGCpi	WW	LW	Pressure
Level	ht	Kz	qz	Cp	-0.55	0.8	-0.5	PSF
RN	160	1.39	33.41	22.631	-18.37	41.00	-13.38	54.38
RS	142	1.36	32.68	22.143	-17.98	40.12	-13.38	53.50
9	140	1.36	32.68	22.143	-17.98	40.12	-13.38	53.50
8	124	1.32	31.72	21.491	-17.45	38.94	-13.38	52.32
7	102	1.26	30.28	20.515	-16.65	37.17	-13.38	50.55
6	78	1.21	29.08	19.700	-15.99	35.69	-13.38	49.07

Equivalent Point Loads					
N-S	Pressure	ht	Dist Ld	By	Py
Level	PSF	ft	plf	ft	k
RN	54.38	10	543.8	150.3	81.7
RS	53.50	9	1527.9	143.8	219.8
9	53.50	18	2009.4	150.3	301.9
8	52.32	20	2209.0	191.2	422.3
7	50.55	23	1162.6	229.2	266.4
Vb =	1292.1	k	Mover =	65303.3	kft

Wind Factors		
	E - W	N - S
G_f =	0.89	0.85
GC pi =	0.55	-
Cp =	-0.3	-0.5
Kd =	0.85	-
Kzt =	1.0	-
I =	1.15	-

Inherent Moments						
	Bx	ex	Mtx	By	ey +	Mt +
Level	ft	ft	k-in	ft	ft	k-in
RN	55.8	8.4	2799	150.3	22.5	22099
RS	53.8	8.1	7014	143.8	21.6	56896
9	55.8	8.4	10039	150.3	22.5	81651
8	113.7	17.1	45681	191.2	28.7	145310
7	118.3	17.8	40072	229.2	34.4	109905

SEISMIC LOAD CALCULATIONS

Seismic Loads								E-W Direction					N-S Direction				
	Ht (ft)	hi	W (k)	wh^k	Cvx	fi	Vi	Bx	5%Bx	Ax	RS	Mz (ft-k)	By	5%By	Ay	RS	Mzy (ft-k)
RN	160	20	841	21539881	0.2278	192	192	150	7.5	22.7		32649	56	2.8	7.1		3809
RS	142	18	649	13092727	0.1385	90	282	144	7.2	70.2	13.2	8558	54	2.7	278.9	1.0	242
9	140	16	678	13281336	0.1405	95	377	150	7.5	11.6		8326	56	2.8	11.4		3040
8	124	22	1674	25746066	0.2723	456	833	191	9.6	2.3		9942	114	5.7	2.8		7333
7	102	24	2007	20876383	0.2208	443	1276	229	11.5	1.0		5077	118	5.9	1.0		2622
k	2	Σ	5849	94536394		Vb =	1276										
T	1.53	s				Mov =	158514										

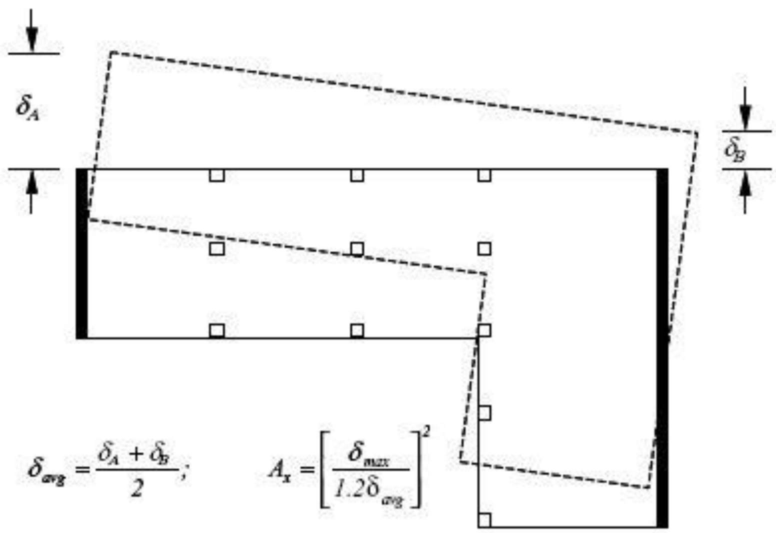


FIGURE 12.8-1 TORSIONAL AMPLIFICATION FACTOR, A_x

Amplification Factor Analysis								RS Alternative					
E-W	$(\delta_{xe})_1$	Δ_{x1}	$(\delta_{xe})_2$	Δ_{x2}	Δ_{avg}	$\Delta_{max}/\Delta_{avg}$		$(\delta_{xe})_1$	Δ_{x1}	$(\delta_{xe})_2$	Δ_{x2}	Δ_{avg}	$\Delta_{max}/\Delta_{avg}$
RN	1.057	0.236	1.054	0.134	0.1850	1.28	*						
RS	0.821	0.041	0.920	0.142	0.0915	1.55	*	0.556	0.128	0.705	0.195	0.1615	1.21
9	0.780	0.191	0.778	0.19	0.1905	1.00							
8	0.589	0.32	0.588	0.33	0.3250	1.02		0.428		0.510			
7	0.269	0.269	0.258	0.258	0.2635	1.02							
	$(\delta_{ye})_1$	Δ_{y1}	$(\delta_{ye})_2$	Δ_{y2}	Δ_{avg}	$\Delta_{max}/\Delta_{avg}$		$(\delta_{ye})_1$	Δ_{y1}	$(\delta_{ye})_2$	Δ_{y2}	Δ_{avg}	$\Delta_{max}/\Delta_{avg}$
RN	0.183	0.053	-0.238	-0.102	-0.0245	4.16	*						
RS	0.130	0.01	-0.136	-0.016	-0.0030	5.33	*	0.308	0.135	0.274	0.100	0.1175	1.15
9	0.120	0.041	-0.120	-0.061	-0.0100	6.10	*						
8	0.079	0.042	-0.059	-0.043	-0.0005	86.00	*	0.173		0.174			
7	0.037	0.037	-0.016	-0.016	0.0105	3.52	*						
N-S	$(\delta_{xe})_1$	Δ_{x1}	$(\delta_{xe})_2$	Δ_{x2}	Δ_{avg}	$\Delta_{max}/\Delta_{avg}$		$(\delta_{xe})_1$	Δ_{x1}	$(\delta_{xe})_2$	Δ_{x2}	Δ_{avg}	$\Delta_{max}/\Delta_{avg}$
RN	0.185	0.032	0.183	0.032	0.0320	1.00							
RS	0.153	0.007	0.151	0.006	0.0065	1.08		0.062	0.023	-0.045	-0.004	0.0095	2.42
9	0.146	0.03	0.145	0.03	0.0300	1.00							
8	0.116	0.038	0.115	0.037	0.0375	1.01		0.039		-0.041			
7	0.078	0.078	0.078	0.078	0.0780	1.00							
	$(\delta_{ye})_1$	Δ_{y1}	$(\delta_{ye})_2$	Δ_{y2}	Δ_{avg}	$\Delta_{max}/\Delta_{avg}$		$(\delta_{ye})_1$	Δ_{y1}	$(\delta_{ye})_2$	Δ_{y2}	Δ_{avg}	$\Delta_{max}/\Delta_{avg}$
RN	0.123	-0.829	0.978	0.218	-0.3055	2.71	*						
RS	0.952	0.053	0.760	0.042	0.0475	1.12		1.039	0.284	1.087	0.341	0.3125	1.09
9	0.899	0.245	0.718	0.198	0.2215	1.11							
8	0.654	0.342	0.520	0.306	0.3240	1.06		0.755		0.746			
7	0.312	0.312	0.214	0.214	0.2630	1.19							

Seismic Design Criteria			
S-200.01		ASCE 7-05	
Sds	0.65	Ta (s)	0.9
Sd1	0.13	Cu	1.7
I	1.25	T (s)	1.53
R	3	TL (s)	6
W (k)	5849		
Cs	0.0602		

	Amp Factor Maximums			RS Alternative		
	δ_{EW}	Δ_{avg}	$\Delta_{max}/\Delta_{avg}$	δ_{EW}	Δ_{avg}	$\Delta_{max}/\Delta_{avg}$
RN	1.057	0.185	5.71			
RS	0.920	0.092	10.05	0.705	0.1615	1.21
9	0.780	0.191	4.09			
8	0.589	0.325	1.81			
7	0.269	0.264	1.02			
	δ_{NS}	Δ_{avg}	$\Delta_{max}/\Delta_{avg}$	δ_{NS}	Δ_{avg}	$\Delta_{max}/\Delta_{avg}$
RN	0.978	-0.31	3.20			
RS	0.952	0.05	20.04	1.087	0.3125	1.09
9	0.899	0.22	4.06			
8	0.654	0.32	2.02			
7	0.312	0.26	1.19			

APPENDIX C: CUSTOM MEMBER STRENGTHS

MEMBERS SPECIFIED IN CURRENT DESIGN

PLATE GIRDERS

Plate Element Dimensions (in)						
Shape	B	tf	nf	D	tw	h
32.5	18	4	1	32.5	2	24.5
33-1	18	4	1	33	2	25
44-1	18	4	1	44	2	36
46-1	18	2	1	46	1	42
46-2	20	4	2	46	2	30
46-3	18	4	1	46	2	38
72-1	16	3	1	72	2	66
L	20	ft				
Weld size	0.5	in				

Material Propoerties		
ϕ	0.9	
E	29000	ksi
Fy	50	ksi
G	11200	ksi
K	1.0	

Note: Dimensions taken from drawing S-211

Section Properties																
	Individual Flange								Web		Total					
	A	x	Ax	Ax2	y	Ay2	Iox	Ioy	A	Ax	A	Ixx	Iyy	rx	ry	WT (plf)
32.5	72	14.25	1026	14620.5	5	1458.0	96.0	1944.0	49	600	193	29433	6804	12	6	656
33-1	72	14.5	1044	15138.0	5	1458.0	96.0	1944.0	50	625	194	30468	6804	13	6	659
44-1	72	20	1440	28800.0	5	1458.0	96.0	1944.0	72	1296	216	57792	6804	16	6	734
46-1	36	22	792	17424.0	5	729.0	12.0	972.0	42	882	114	34872	3402	17	5	388
46-2	160	19	3040	57760.0	5	4000.0	853.3	5333.3	60	900	380	117227	18667	18	7	1291
46-3	72	21	1512	31752.0	5	1458.0	96.0	1944.0	76	1444	220	63696	6804	17	6	748
72-1	48	34.5	1656	57132.0	4	768.0	36.0	1024.0	132	4356	228	114336	3584	22	4	775

Tensile Strength	
	ϕTn (k)
32.5	8685
33-1	8730
44-1	9720
46-1	5130
46-2	17100
46-3	9900
72-1	10260

Flexural Strength												
	Flange Compactness				Web Compactness				Yielding		Max Unbr Length (Lp)	
	bf/2tf	λp	λr		h/tw	λp	λr		Z	ϕMn (ft-k)	in	ft
32.5	2.25	9.2	24.1	C	12.3	90.6	137.3	C	3252.5	12197	214	18
33-1	2.25	9.2	24.1	C	12.5	90.6	137.3	C	3338	12518	214	18
44-1	2.25	9.2	24.1	C	18.0	90.6	137.3	C	5472	20520	202	17
46-1	4.50	9.2	24.1	C	42.0	90.6	137.3	C	3348	12555	197	16
46-2	2.50	9.2	24.1	C	15.0	90.6	137.3	C	7880	29550	253	21
46-3	2.25	9.2	24.1	C	19.0	90.6	137.3	C	5912	22170	201	17
72-1	2.67	9.2	24.1	C	33.0	90.6	137.3	C	12024	45090	143	12

Compressive Strenth																						
	Flange Slenderness					Web Slenderness				Flexural Buckling				Torsional Buckling					Compressive Strength			
	b/2tf	Kc'	Kc	λr		h/tw	λr			KL/r	KL/r Limit	Fe flex	Fcr flex	ho	Cw	J	Fe tors	Fcr tors	Fcr	Pn	ϕPn	
32.5	2.25	1.14	0.76	13.44	NS	12.25	35.88	NS	NS	19.4	113	758	48.6	29	1381637	460	332	46.9	46.9	9060	8154	k
33-1	2.25	1.13	0.76	13.44	NS	12.50	35.88	NS	NS	19.2	113	780	48.7	29	1430541	461	329	46.9	46.9	9103	8193	k
44-1	2.25	0.94	0.76	13.44	NS	18.00	35.88	NS	NS	14.7	113	1330	49.2	40	2721600	491	294	46.6	46.6	10059	9053	k
46-1	4.50	0.62	0.62	12.11	NS	42.00	35.88	S	SLENDER	-	-	-	-	-	-	-	-	-	-	-	-	
46-2	1.25	1.03	0.76	13.44	NS	15.00	35.88	NS	NS	13.7	113	1533	49.3	38	6738667	3515	536	48.1	48.1	18273	16445	k
46-3	2.25	0.92	0.76	13.44	NS	19.00	35.88	NS	NS	14.1	113	1439	49.3	42	3000564	496	290	46.5	46.5	10235	9211	k
72-1	2.67	0.70	0.70	12.86	NS	33.00	35.88	NS	NS	10.7	113	2492	49.6	69	4265856	328	211	45.3	45.3	10323	9291	k

HSS ROUND SECTIONS

22R

Section Properties

Concrete			Reinforcement			Pipe					
wt	145	pcf	fy	150	ksi	fy	46	ksi	Do	22	in
f'c	8000	psi	Esr	29000	ksi	Es	29000	ksi	t	1.25	in
						wt	490	pcf			
			Callout	11					td	1.16	
Ag	299	in2	Ai	1.56	in2	Isx	64638.9	in4	Di	19.5	in
Ec	5098	ksi	n	2		Zx	496.8	in3	Dd	21.8	
Ig	7098	in4	Asr	3.12	in2	ρ	0.202	OK	Ast	75.5	in2

Slenderness Checks

AISC XIV Chapter I

Pipe Slenderness			λp	λr	Max		
D/t	18.92	<	95	120	195	C	Compression
			57	195	195	C	Flexure

ACI 318-11 Chapter 10

Composite Shape Slenderness					
EcIg	36184968	rcomp	27.5		
EcAg	1522576	K	1.0		
EsIsx	1874528059	L	45		
EsAsx	2188385	KL/r	19.7	22	KL/r Limit
		SLENDERNESS NOT CONSIDERED			

AISC Chapter I

Compression Capacity		
NO SLENDER ELEMENTS		
FyAs	3471	
C2	0.95	
AsrEs/Ec	17.75	
Pno	5852	
φPn	4389	k
Tensile Capacity		
Asfy	3471	k
AsrFysr	468	k
Tno	3939	
φTn	3545	k
Flexural Capacity		
SLENDERNESS NOT CONSIDERED		
Yielding		
Mp=FyZ	22855	in-k
φMn	1714	ft-k

15A

Section Properties

Concrete			Reinforcement			Pipe					
wt	145	pcf	fy	150	ksi	fy	46	ksi	Do	15	in
f'c	8000	psi	Esr	29000	ksi	Es	29000	ksi	t	1.25	in
						wt	490	pcf			
Ag	123	in2	Callout	11					td	1.16	
Ac	123	in2	Ai	1.56	in2	Isx	18763	in4	Di	12.5	in
Ec	5098	ksi	n	0		Zx	217.5	in3	Dd	14.83	
Ig	1198	in4	Asr	0	in2	ρ	0.289	OK	Ast	49.9	in2

Slenderness Checks

AISC XIV Chapter I

Pipe Slenderness			λp	λr	Max		
D/t	12.90	<	95	120	195	C	Compression
			57	195	195	C	Flexure

ACI 318-11 Chapter 10

Composite Shape Slenderness					
EcIg	6109839	rcomp	18.6		
EcAg	625648	K	1.0		
EsIsx	544119416	L	25		
EsAsx	1447008	KL/r	16.1	22	KL/r Limit
		SLENDERNESS NOT CONSIDERED			

AISC Chapter I

Compression Capacity		
NO SLENDER ELEMENTS		
FyAs	2295	
C2	0.95	
AsrEs/Ec	0.00	
Pno	3228	
φPn	2421	k
Tensile Capacity		
Asfy	2295	k
AsrFysr	0	k
Tno	2295	
φTn	2066	k
Flexural Capacity		
SLENDERNESS NOT CONSIDERED		
Yielding		
Mp=FyZ	10006	in-k
φMn	750	ft-k

15B

Section Properties

Concrete			Reinforcement			Pipe					
wt	145	pcf	fy	150	ksi	fy	46	ksi	Do	15	in
f'c	8000	psi	Esr	29000	ksi	Es	29000	ksi	t	1	in
						wt	490	pcf			
Ag	133	in2	Callout	11					td	0.93	
Ac	133	in2	Ai	1.56	in2	Isx	15865	in4	Di	13	in
Ec	5098	ksi	n	0		Zx	180.7	in3	Dd	14.86	
Ig	1402	in4	Asr	0	in2	ρ	0.235	OK	Ast	40.7	in2

Slenderness Checks

AISC XIV Chapter I

Pipe Slenderness			λp	λr	Max		
D/t	16.13	<	95	120	195	C	Compression
			57	195	195	C	Flexure

ACI 318-11 Chapter 10

Composite Shape Slenderness					
EcIg	7147648	rcomp	18.7		
EcAg	676700	K	1.0		
EsIsx	460092980	L	25		
EsAsx	1180272	KL/r	16.0	22	KL/r Limit
			SLENDERNESS NOT CONSIDERED		

AISC Chapter I

Compression Capacity		
NO SLENDER ELEMENTS		
FyAs	1872	
C2	0.95	
AsrEs/Ec	0.00	
Pno	2881	
ϕPn	2161	k

Tensile Capacity		
Asfy	1872	k
AsrFysr	0	k
Tno	1872	
ϕTn	1685	k

Flexural Capacity		
SLENDERNESS NOT CONSIDERED		
Yielding		
Mp=FyZ	8314	in-k
ϕMn	624	ft-k

PROPOSED ADDITIONAL MEMBERS

PG56-1

Material Properties

fy	50	ksi
E	29000	ksi
G	11200	ksi
dens	490	PCF
	0.284	pci

Plate Properties

Lb	20	ft
D	56	in
B	24	in
tf	10	in
tw	2.25	in
hw	36	in
ho	46	in

Section Properties

Single Flange		Web		Total		
A	240	A	81	A	561	in2
x	23	x	0.563	Ixx	128960	in4
Ax	5520	Ax	45.56	rx	15.2	in
Ax2	126960			Zx	11085.6	in3
Ixo	2000					
y	6			Iyy	20160	in4
Ay	1440			ry	6.0	in
Ay2	8640					
Iyo	11520			J	16136.7	
				Weight	1909	PLF

Compression Design

Flange Slenderness				
B/2tf	K'c	Kc	λr	
1.20	1.000	0.76	13.4	NS
Web compactness				
hw/tw	λr			
16	35.9	NS		
SLENDERNESS NOT CONSIDERED				
Limit State 1		Flexural Buckling		
	K	1.0		
	L	240		
	KL/r	15.8	113.4	KL/r Limit
	Fe	1142	ksi	E3-2
	Fcr	49.1	ksi	
Limit State 2		Torsional Buckling		
	Cw	1E+07	in6	
	Fe	1567	ksi	
	Fcr	49.3	ksi	
ϕPn	27541	k		

Flexure Design

Flange Compactness			
B/2tf	λp	λr	
1.20	9.2	24.1	C
Web Compactness			
hw/tw	λp	λr	
16	90.6	137.3	C
SLENDERNESS NOT CONSIDERED			
Limit State 1		Yielding	
	Mp	554278.1	in-k
Limit State 2		LTB	
	Lb	240	in
	Lp	254.1	in
LTB DOES NOT APPLY			
ϕMn	41570.86	ft-k	

Tension Design

Yielding		
ϕTn	25245	k
RUPTURE MUST BE CONSIDERED		

Shear Design

Traverse Stiffeners		
hw/tw	hw/tw Limit	
16	59.2	
NO STIFFENERS REQUIRED		
kv	5	
Cv = 1.0		
	hw/tw Limit	59.2
	Cv	1.0
Vn	3780	
ϕVn	3402	k

PG56-1 Equivalent Cross Section

Material Properties

fy	50	ksi
E	29000	ksi
G	11200	ksi
dens	490	PCF
	0.284	pci

Plate Properties

Lb	15	ft
D	76	in
B	20	in
tf	8	in
tw	2	in
hw	60	in
ho	68	in

Section Properties

Single Flange		Web		Total		
A	160	A	120	A	440	in2
x	34	x	0.5	Ixx	185813	in4
Ax	5440	Ax	60	rx	20.6	in
Ax2	184960			Zx	10940	in3
Ixo	853.33					
y	5			Iyy	9333.33	in4
Ay	800			ry	4.6	in
Ay2	4000					
Iyo	5333.3			J	6986.67	
				Weight	1497	PLF

Flexure Design

Flange Compactness			
B/2tf	λp	λr	
1.25	9.2	24.1	C
Web Compactness			
hw/tw	λp	λr	
30	90.6	137.3	C
SLENDERNESS NOT CONSIDERED			
Limit State 1		Yielding	
	Mp	547000	in-k
Limit State 2		LTB	
	Lb	180	in
	Lp	195.2	in
LTB DOES NOT APPLY			
ϕMn	41025	ft-k	

Tension Design

Yielding		
ϕTn	19800	k
RUPTURE MUST BE CONSIDERED		

Shear Design

Traverse Stiffeners			
hw/tw	hw/tw Limit		
30	59.2		
NO STIFFENERS REQUIRED			
kv	5		
Cv = 1.0			
	hw/tw Limit		59.2
	Cv		1.0
Vn	4560		
ϕVn	4104	k	

Compression Design

Flange Slenderness				
B/2tf	K'c	Kc	λr	
1.25	0.730	0.7303	13.2	NS
Web compactness				
hw/tw	λr			
30	35.9	NS		
SLENDERNESS NOT CONSIDERED				
Limit State 1		Flexural Buckling		
	K	1.0		
	L	180		
	KL/r	8.8	113.4	KL/r Limit
	Fe	3731	ksi	E3-2
	Fcr	49.7	ksi	
Limit State 2		Torsional Buckling		
	Cw	1E+07	in6	
	Fe	889	ksi	
	Fcr	48.8	ksi	
ϕPn	21488	k		

24R-1

Section Properties											
Concrete			Reinforcement			Pipe					
wt	145	pcf	fy	150	ksi	fy	46	ksi	Do	24	in
f'c	15000	psi	Esr	29000	ksi	Es	29000	ksi	t	1.75	in
						wt	490	pcf			
			Callout	11					td	1.63	
Ag	330	in2	Ai	1.56	in2	Isx	111388.7	in4	Di	20.5	in
Ec	6981	ksi	n	16		Zx	798.3	in3	Dd	23.755	
Ig	8669	in4	Asr	24.96	in2	ρ	0.255	OK	Ast	113.1	in2

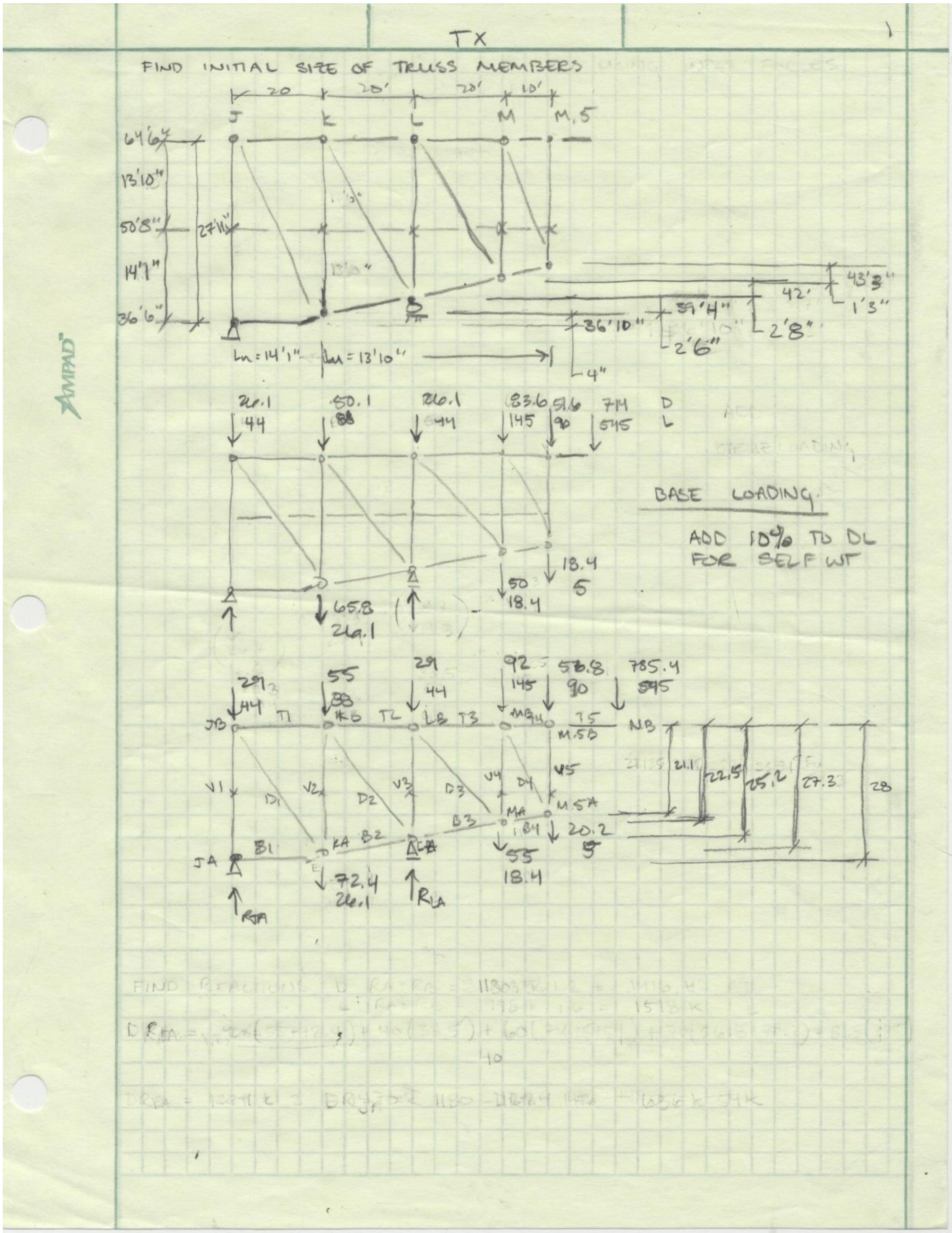
AISC Chapter I		
Compression Capacity		
NO SLENDER ELEMENTS		
FyAs	5204	
C2	0.95	
AsrEs/Ec	103.69	
Pno	11030	
ϕPn	8272	k
Tensile Capacity		
Asfy	5204	k
AsrFysr	3744	k
Tno	8948	
ϕTn	8053	k
Flexural Capacity		
SLENDERNESS NOT CONSIDERED		
Yielding		
Mp=FyZ	36722	in-k
ϕMn	2754	ft-k

Slenderness Checks							
AISC XIV Chapter I							
Pipe Slenderness			λp	λr	Max		
D/t	13.71	<	95	120	195	C	Compression
			57	195	195	C	Flexure

ACI 318-11 Chapter 10					
Composite Shape Slenderness					
EcIg	60520963	rcomp	29.4		
EcAg	2304189	K	1.0		
EsIsx	3230271344	L	25		
EsAsx	3280962	KL/r	10.2	22	KL/r Limit
		SLENDERNESS NOT CONSIDERED			

APPENDIX D: ETABS VERIFICATION

HAND CALCULATIONS



TX 2

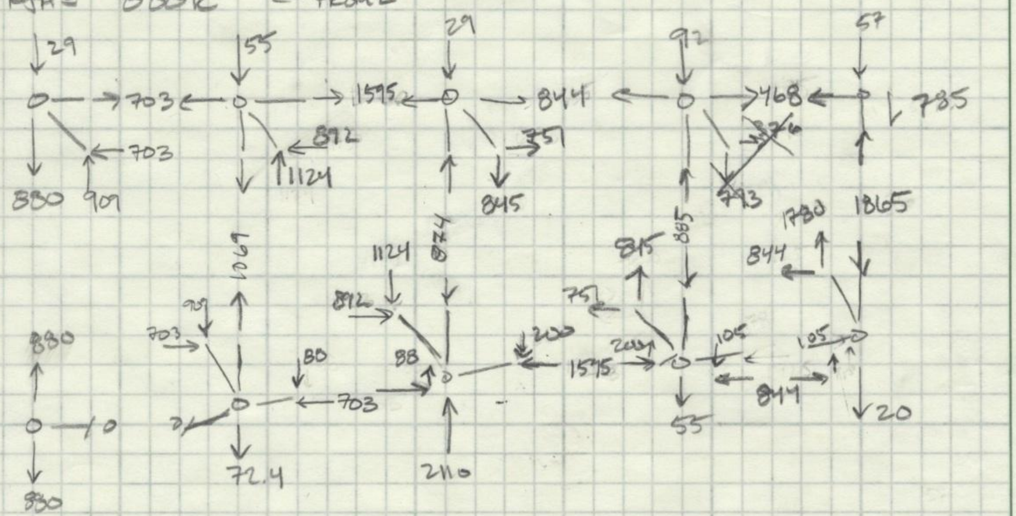
DEAD ITTERATION:

$$R_{LA} = \frac{20(55+72.4) + 40(29) + 60(92+55) + 70(56.8+55) + 81.5(785)}{40}$$

$R_{LA} = 2110\,k$

$R_{JA} = 29+55+72.4+29+92+55+56.8+55+785-2110$

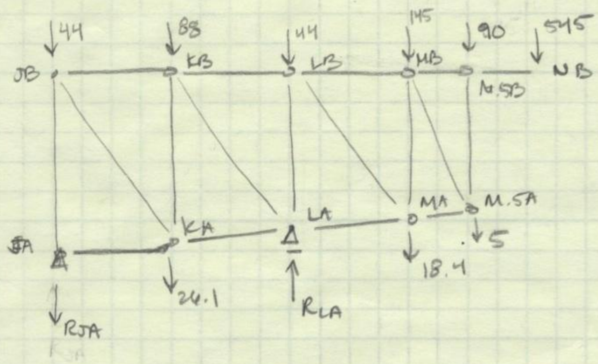
$R_{JA} = -880\,k$ ← FROM L



FROM R

TX 3

LIVE ITERATION



IGNORE PATTERN LOADING FOR TRIAL SIZES.

$\Sigma F_y = 0$

$R_{JA} + 26.1 + 18.4 + 5 + 44 + 88 + 44 + 145 + 90 + 545 = R_{LA}$

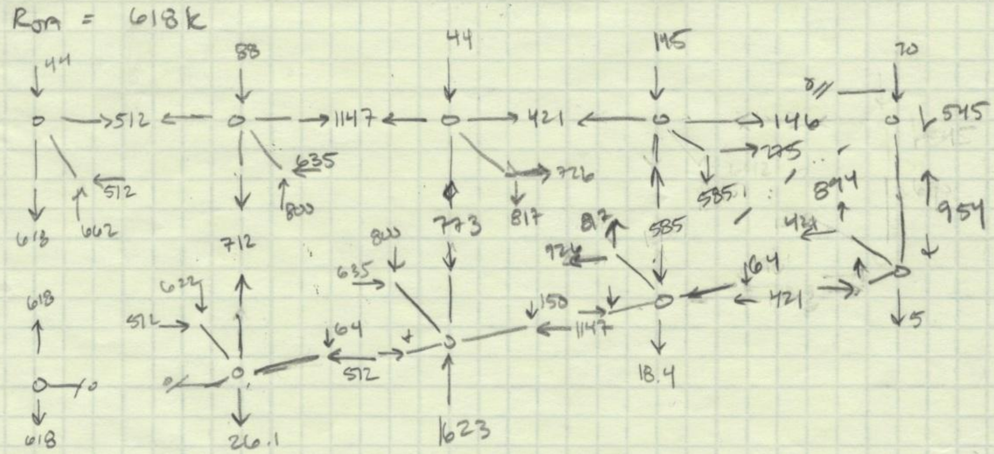
$R_{JA} + 1006 = R_{LA}$

$\Sigma M_{JA} = 0$

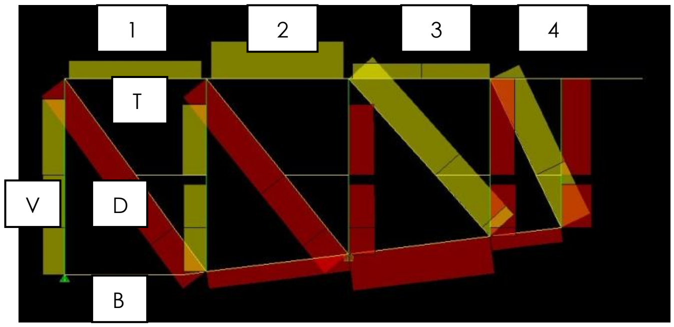
$$R_{LA} = \frac{20(88+26.1) + 40(44) + 60(145+18.4) + 70(95) + 81.5(545)}{40}$$

$R_{LA} = 1623\text{k}$

$R_{JA} = 618\text{k}$



MEMBER SELECTION TABLE



Truss Loads													Trial Member Selection					
		L	Pdx	Pdy	Plx	Ply	Pd	PL	1.2Pd	1.6Pl	Pu	T/C	p max	L	p	Trial Size		ϕPn
D	1	33.8	703	909	512	662	1149	837	1379	1339	2718	C	0.368	18	0.325	W14x	283	3077
	2	32.2	892	1124	635	800	1435	1021	1722	1634	3356	C	0.298	18	0.266	W14x	342	3759
	3	29.7	751	845	726	817	1130	1093	1357	1749	3105	T	0.322	0	0.271	W14x	342	3690
	4	23.3	844	1780	421	894	1970	988	2364	1581	3945	T	0.253	0	0.204	W14x	455	4902
T	1	20	703		512		703	512	843.6	819.2	1663	T	0.601	0	0.521	W27x	178	1919
	2	20	1595		1147		1595	1147	1914	1835	3749	T	0.267	0	0.172	W27x	539	5814
	3	20	844		421		844	421	1013	673.6	1686	T	0.593	0	0.575	W27x	161	1739
	4	10	468		146		468	146	561.6	233.6	795.2	T	1.258	0	0.575	W27x	161	1739
B	1	20.05	0		0		0	0	0	0	0	C	0.597	22	0.552	W27x	217	1812
	2	20.16	703	88	512	64	708	516	850.2	825.6	1676	C	0.597	22	0.552	W27x	217	1812
	3	20.16	1595	200	1147	150	1607	1157	1929	1851	3780	C	0.265	22	0.205	W27x	539	4878
	4	10.08	844	105	421	64	851	426	1021	681.3	1702	C	0.588	12	0.54	W27x	161	1852
V	1	28	880		618		880	618	1056	988.8	2045	T	0.489	0	0.482	W14x	193	2075
	2	27.3	1069		712		1069	712	1283	1139	2422	T	0.413	0	0.399	W14x	233	2506
	3	25.2	874		773		874	773	1049	1237	2286	C	0.438	14	0.406	W14x	211	2463
	4	22	885		585		885	585	1062	936	1998	C	0.501	14	0.487	W14x	176	2053
	5	21.1	1865		954		1865	954	2238	1526	3764	C	0.266	14	0.247	W14x	342	4049

APPENDIX E: TRUSS DESIGN CALCULATIONS

EXTERIOR WALL ALLOCATION

TRUSS 0.9

Truss 0.9																	
Wt	15	PSF	G		H		I		J		K		L		L.5		
L	Elev.	Ht	W	Wt	W	Wt	W	Wt	W	Wt	W	Wt	W	Wt	W	Wt	
9	151.67	9.1	20	2.73	20	2.73	11.5	1.57									
8	133.5	19.9	20	5.98	20	5.98	20	5.98	17.1	5.11	8.67	2.59					
7	111.83	22.7	20	6.80	20	6.80	20	6.80	20	6.80	20	6.80	14.33	4.87	5	1.70	
6	88.17	23.7	20	7.10	20	7.10	20	7.10	20	7.10	20	7.10	14.33	5.09	5	1.78	
Loads Applied				22.6	k	22.6	k	21.4	k	19.0	k	16.5	k	10.0	k	3.5	k
L	Elev.	Ht.	KLF														
5	64.5	11.8	0.18														

TRUSS X

Truss X								
AREA		H	I	J	K	L	M	M.5
L	Elev.	20	40	60	80	100	120	130
5	64.5	138.3	138.3	138.3	138.3	138.3	172.9	34.6
4	50.67	283.4	283.4	283.4	283.4	283.4	283.4	283.4
3	36.5	245.0	245.0	219.6	165.2	110.8	56.4	29.2
2	24.25	83.4	29.0					
	Bottom	20.08	22.8	25.52	28.24	30.96	33.68	35.04
LOAD		Wt =	15	PSF				
L	Elev.	H	I	J	K	L	M	M.5
5	24.25	2.1	2.1	2.1	2.1	2.1	2.6	0.5
4	50.67	4.3	4.3	4.3	4.3	4.3	4.3	4.3
3	36.5	3.7	3.7	3.3	2.5	1.7	0.8	0.4
2	24.25	1.3	0.4					

TRUSS 0.9

LEVEL 6 LOADS

Z*-G									
Level	Area	DL	DW	DC	LL	SL	D	L	S
9	520	84	2.73	2	50	22	48.4	26.0	11.4
8	590	121	5.98	2	150	0	79.4	88.5	0.0
7	660	121	6.80	2	100	0	88.7	66.0	0.0
6	390	148	7.10	2	130	4.4	66.8	50.7	1.7
						Total =	283	231	13

G.5									
Level	Area	DL	DW	DC	LL	SL	D	L	S
6	390	152.1	0	0	130	4.4	59.3	50.7	1.7

W*-H									
Level	Area	DL	DW	DC	LL	SL	D	L	S
9	470	84	2.73	2	50	22	44.2	23.5	10.3
8	530	121	5.98	2			72.1	0.0	0.0
7	580	121	6.80	2	100		79.0	58.0	0.0
6	390	156	7.10	2	130	4.4	70.0	50.7	1.7
						Total =	265	132	12

H.5									
Level	Area	DL	DW	DC	LL	SL	D	L	S
6	390	157.8	0	0	140	2.2	61.5	54.6	0.9

W*-I									
Level	Area	DL	DW	DC	LL	SL	D	L	S
9	280	84	1.57	2	50	22	27.1	14.0	6.2
8	500	154	5.98	2	125	0	85.0	62.5	0.0
7	550	121	6.80	2	100	0	75.3	55.0	0.0
6	390	159	7.10	2	150	0	71.3	58.5	0.0
						Total =	259	190	6

I.5									
Level	Area	DL	DW	DC	LL	SL	D	L	S
6	390	162.8	0	0	150	0	63.5	58.5	0.0

W*-J									
Level	Area	DL	DW	DC	LL	SL	D	L	S
8	365	187	5.11	2	100	0	75.4	36.5	0.0
7	530	121	6.80	2	100	0	72.9	53.0	0.0
6	390	166	7.10	2	150	0	73.9	58.5	0.0
						Total =	222	148	0

J.5									
Level	Area	DL	DW	DC	LL	SL	D	L	S
6	390	168	0	0	150	0	65.5	58.5	0.0

W*-K									
Level	Area	DL	DW	DC	LL	SL	D	L	S
8	245	187	2.59	2	100	0	50.4	24.5	0.0
7	520	146	6.80	2	100	0	84.8	52.0	0.0
6	390	170	7.10	2	150	0	75.2	58.5	0.0
						Total =	210	135	0

K.5									
Level	Area	DL	DW	DC	LL	SL	D	L	S
6	390	173	0	0	150	0	67.4	58.5	0.0

W*-L									
Level	Area	DL	DW	DC	LL	SL	D	L	S
7	370	183	4.87	2	100	0	74.6	37.0	0.0
6	364	176	5.09	2	150	0	71.2	54.6	0.0
						Total =	146	92	0

W*-L.5									
Level	Area	DL	DW	DC	LL	SL	D	L	S
7	150	183	1.70	2	100	0	31.1	15.0	0.0
6	372	183	1.78	2	150	0	71.9	55.9	0.0
						Total =	103	71	0

W*-M									
Level	Area	DL	DW	DC	LL	SL	D	L	S
6	411	203	0.00	2	150	0	85.5	61.7	0.0

M.5									
Level	Area	DL	DW	DC	LL	SL	D	L	S
6	390	203	33.00	2	150	0	114.2	58.5	0.0

LEVEL 5 LOADS

Load Information		
Wall Load	0.18	KLF
Dead Load	109	PSF
Live Load	200	PSF
Grid Angle	9	deg
	0.157	rad
Trib Angle	0.079	rad

		X Trib			Y Trib				Load Components					Loads	
Grid	X	Start	End	Dx	Base	Start	End	Dy	At	AD	PW	PC	AL	PD	PL
F	0	0	5	5	0.31	0.31	0.70	0.39	2.5	0.3	3.6	2	0.5	5.8	0.5
F.5	10	5	15	10	1.09	0.70	1.49	0.79	10.9	1.2			2.2	1.2	2.2
G	20	15	25	10	1.88	1.49	2.27	0.79	18.8	2.0	3.6	2	3.8	7.6	3.8
G.5	30	25	35	10	2.67	2.27	3.06	0.79	26.7	2.9			5.3	2.9	5.3
H	40	35	45	10	3.45	3.06	3.85	0.79	34.5	3.8	3.6	2	6.9	9.3	6.9
H.5	50	45	55	10	4.24	3.85	4.63	0.79	42.4	4.6			8.5	4.6	8.5
I	60	55	65	10	5.03	4.63	5.42	0.79	50.3	5.5	3.6	2	10.1	11.0	10.1
I.5	70	65	75	10	5.81	5.42	6.21	0.79	58.1	6.3			11.6	6.3	11.6
J	80	75	85	10	6.60	6.21	7.00	0.79	66.0	7.2	3.6	2	13.2	12.7	13.2
J.5	90	85	95	10	7.39	7.00	7.78	0.79	73.9	8.1			14.8	8.1	14.8
K	100	95	105	10	8.18	7.78	8.57	0.79	81.8	8.9	3.6	2	16.4	14.5	16.4
K.5	110	105	115	10	8.96	8.57	9.36	0.79	89.6	9.8			17.9	9.8	17.9
L	120	115	125	10	9.75	9.36	10.14	0.79	97.5	10.6	3.6	2	19.5	16.2	19.5
L.5	130	125	135	10	10.54	10.14	10.93	0.79	105.4	11.5			21.1	11.5	21.1
M	140	135	145	10	11.32	10.93	11.72	0.79	113.2	12.3	3.8	2	22.6	18.2	22.6
M.5	150	145	155.75	10.8	12.11	11.72	12.56	0.85	130.5	14.2			26.1	14.2	26.1
N.2	162	155.8	161.5	5.75	13.02	12.56	13.02	0.45	73.5	8.0	2.0	2	14.7	12.1	14.7

REACTIONS

Reactions (k)		
Level 6	H	N.2
D	0	340
L	0	244
S	0	0.5
Level 5		
D	1112	404
L	747	295
S	18	0.5

TRUSS N.2

Load Properties			
	Level 6	Level 5	
Trib W	6.5	6.5	ft
DL	203	109	PSF
LL	150	200	PSF
Distributed Loads			
wD	1.3	0.7	klf
wL	1.0	1.3	klf
Point Loads			
Pd	340	404	k
Pl	244	295	k

Reactions		
	N.2	6
D	1340	-350
L	945	-252
S	2	-1

TRUSS X

LOAD SUMMARY

Level		J	J.5	K	K.5	L	L.5	M	M.5	N.2
5	D	2.1	24.0	26.1	24.0	2.1	32.7	50.9	51.6	1340.0
	L		44.0	44.0	44.0		60.0	85.0	90.0	945.0
	S									2.0
4	D	17.5		31.7		22.6		45.9	16.9	
	L	6.8		14.0		10.3		18.4	5.0	
3	D	19.2		34.1		9.1		3.6	1.5	
	L	6.8		12.1		9.0		0.0	0.0	

Load	Reaction (k)	
	J	L
D	-1072	3392
L	-787	2419
S	-2	4
Pu	-2547	7943

AREA TAKEOFF

X-H							
Level	Area	DL	DW	DC	LL	D	L
2	170	20	1.3	0.5	0	5.2	0.0

X-H.5							
Level	Area	DL	DW	DC	LL	D	L
5	220	109	0	0	200	24.0	44.0
4	170	98	0	0	50	16.7	8.5
3	200	118	0	0	50	23.6	10.0
2	80	20	0.0	0.5	0	2.1	0.0

X-I							
Level	Area	DL	DW	DC	LL	D	L
5	220	109	2.1	0	200	26.1	44.0
4	160	98	4.3	2	50	21.9	8.0
3	215	118	3.7	2	50	33.3	10.8
3(2)	160	20	0	0	0	3.2	0.0
2	66	20	0.4	0.5	0	2.3	0.0

X-I.5							
Level	Area	DL	DW	DC	LL	D	L
5	220	109	0	0	200	24.0	44.0
4	150	98	0	0	50	14.7	7.5
3	145	118	0	0	50	17.1	7.3
2	50	20	0	0.5	0	1.5	0.0

X-J.5							
Level	Area	DL	DW	DC	LL	D	L
5	220	109	0	0	200	24.0	44.0
4	120	98	0	0	50	11.8	6.0
3	125	118	0	0	50	14.8	6.3

X-K							
Level	Area	DL	DW	DC	LL	D	L
5	220	109	2.1	0	200	26.1	44.0
4	130	98	4.3	2	50	19.0	6.5
3	135	115	2.5	2	100	20.0	13.5

X-K.5							
Level	Area	DL	DW	DC	LL	D	L
5	220	109	0	0	200	24.0	44.0
4	115	98	0	0	50	13.7	8.9
4(3)	85	112	0	0	150	9.5	12.8
3	120	112	0	0	150	13.4	18.0

X-L.5							
Level	Area	DL	DW	DC	LL	D	L
5	300	109	0	0	200	32.7	60.0
4	235	98	0	0	50	23.0	11.8
3	95	15	0	0	0	1.4	0.0

X-M							
Level	Area	DL	DW	DC	LL	D	L
5	425	109	2.6	2	200	50.9	85.0
4	250	98	4.3	2	50	34.4	12.5
4(3)	350	15	0.0	0	0	5.3	0.0
3	100	15	0.8	0.5	0	2.8	0.0

X-M.5							
Level	Area	DL	DW	DC	LL	D	L
5	450	109	0.5	2	200	51.6	90.0
4	100	98	4.3	2	50	16.9	5.0
4(3)	85	15	0	0	0	1.3	0.0
3	40	15	0.4	0.5	0	1.5	0.0

TRUSS J

Area Takeoff							
Dead Loads				4			0.9
	w	D	wD (klf)	A	PD	PC	PD
5	10	109	1.1	445	48.5	0.0	1072
4	10	98	1.0	445	43.6	2.0	
3	10	118	1.2	445	52.5	2.0	
Live Loads				4			0.9
	w	D	wD (klf)	A	PL	PC	PL
5	10	200	2.0	445	89.0		787
4	10	50	0.5	445	22.3		
3	10	50	0.5	445	22.3		
Level 5					Reactions (k)		
	w	0.9	X	4		3	4
D	1.1	1072	-18.7	48.5	D	-973	429
L	2.0	787	-6.8	89.0	L	-22	179
S			-4		S	-6	2
Level 4							
D	1.0		-361.7	45.6			
L	0.5		283.6	22.3			
Level 3							
D	1.2		-419.7	54.5			
L	0.5		-333.9	22.3			

TRUSS H

Level 5				
	0.9	X	4	w
D	1112	-19	35	2.2
L	747	-7	64	4
S	18			
Level 4				
	0.9	X	4	w
D		780	31	2.0
L		568	19	1
Level 3				
	0.9	X	4	w
D		-640	38	2.36
L		-464	32	1

	Reactions (k)	
Load	3	4
D	2479	-906
L	1661	-572
S	34	-16
Pu	5646	-2008.8
Colun	24R-1	15A

APPENDIX F: FOUNDATION IMPACT

4-H								
Loads In			Exterior			Point Loads		
Pd	-898	k	Area	56	sft	Pd1	70	k
Pl	-572	k	DL	198	PSF	Pl1	58	k
Ps	-16	k	LL	200	PSF	Ps1	0	k
Self	8	k	Interior			Design Loads		
			Area	470	sft	Pd2	-820	k
			DL	126	PSF	Pl2	-514	k
Pu =	-1814	k	LL	100	PSF	Ps2	-16	k

4-J								
Loads In			Exterior			Point Loads		
Pd	429	k	Area	112	sft	Pd1	74	k
Pl	179	k	DL	198	PSF	Pl1	64	k
Ps	2	k	LL	200	PSF	Ps1	0	k
Self	8	k	Interior			Design Loads		
			Area	414	sft	Pd2	511	k
			DL	126	PSF	Pl2	243	k
Pu =	1003	k	LL	100	PSF	Ps2	2	k

6-N.2								
Loads In			Exterior			Point Loads		
Pd	-350	k	Area	250	sft	Pd1	50	k
Pl	-252	k	DL	198	PSF	Pl1	50	k
Ps	-1	k	LL	200	PSF	Ps1	0	k
Self	8	k	Interior			Design Loads		
			Area	0	sft	Pd2	-293	k
			DL	126	PSF	Pl2	-202	k
Pu =	-675	k	LL	100	PSF	Ps2	-1	k

3-H								
Loads In			Exterior			Point Loads		
Pd	2479	k	Area	470	sft	Pd1	100	k
Pl	1661	k	DL	198	PSF	Pl1	100	k
Ps	34	k	LL	200	PSF	Ps1	0	k
Self	8	k	Interior			Design Loads		
			Area	56	sft	Pd2	2587	k
			DL	126	PSF	Pl2	1761	k
Pu =	5938	k	LL	100	PSF	Ps2	34	k

3-J								
Loads In			Exterior			Point Loads		
Pd	-973	k	Area	526	sft	Pd1	104	k
Pl	-179	k	DL	198	PSF	Pl1	105	k
Ps	-6	k	LL	200	PSF	Ps1	0	k
Self	8	k	Interior			Design Loads		
			Area	0	sft	Pd2	-861	k
			DL	126	PSF	Pl2	-74	k
Pu =	-1154	k	LL	100	PSF	Ps2	-6	k

3-L								
Loads In			Exterior			Point Loads		
Pd	3392	k	Area	526	sft	Pd1	104	k
Pl	2419	k	DL	198	PSF	Pl1	105	k
Ps	4	k	LL	200	PSF	Ps1	0	k
Self	20	k	Interior			Design Loads		
			Area	0	sft	Pd2	3516	k
			DL	126	PSF	Pl2	2524	k
Pu =	8260	k	LL	100	PSF	Ps2	4	k

APPENDIX G: DEFLECTION CALCULATIONS

Truss Information								Itemized			Live Load Deflection				Total Load Deflection			
		Lsimple		Ls	Lcant		Lc	Deflections (in)			Support		Mid/		Support		Mid/	
I.D.	Condition	ft	in	in	ft	in	in	D	L	T	1	2	End	Max	1	2	End	Max
J	Cantilever	0	0	0	6	4	76	-0.46	-0.03	-0.49	0.00	0.00	0.03	0.03	0	0	0.49	0.49
X	Cantilever	40	6	486	42	1	505	2.12	1.50	3.62	0.03	0.00	1.50	1.53	0.49	0	3.62	4.13
N.2	Cantilever	43	11	527	26	1	313	0.58	0.41	0.99	0.00	1.53	0.41	2.85	0	4.129	0.99	7.57
H	Cantilever	0	0	0	19	6	234	1.13	0.76	1.89	0.00	0.00	0.76	0.76	0	0	1.89	1.89
0.9	Simple	121	6	1458	0	0	0	1.21	0.875	2.09	0.76	2.85	0.88	2.68	1.89	7.572	2.085	6.82
X:0.9	Simple	13	0	156	0	0	0	1.05	0.75	1.80	0.03	1.80	-	1.83	0.49	6.82	-	7.31

Allowable Deflection at Cantilever							
	X	Y	dX	dY	L	L/180	L/120
3-L	2664	222	498	-234	550.2	3.06	4.59
0.9-N.2	3162	-12					

Allowable Deflection for X:0.9							
	X	Y	dX	dY	L	L/360	L/240
3-J	2184	222	0	-234	234	0.65	0.98
0.9-J	2184	-12					

APPENDIX H: COST AND SCHEDULE INFORMATION

RS MEANS CITY DATA

	[illegible]	71.7	
on mountain	97.2	86.4	
aint Paul	100.4	119.9	10
inneapolis	101.4	122.0	11
uluth	99.7	110.3	10
ochester	100.1	106.1	10
ankato	97.3	103.2	9
ndom	96.0	92.8	9
illmar	95.6	103.9	9
. Cloud	97.2	119.4	10
ainerd	97.1	103.7	10
troit Lakes	99.1	97.9	9
midji	98.4	99.8	9
ef River Falls	98.0	94.5	9
rksdale	96.1	54.9	77
eenville	99.9	64.3	84
elo	97.7	57.3	79
enwood	97.3	54.4	78
kson	97.4	63.6	82
idian	96.5	65.3	82
rel	97.5	58.1	80
xi	98.1	59.5	81
omb	96.1	54.0	77
mbus	97.4	56.2	79
Louis	99.9	106.3	102.
ling Green	98.2	94.5	96.
ibal	97.1	91.8	94.
ville	100.3	89.4	95.
River	99.1	95.2	97.
Girardeau	99.5	89.5	95.
ston	97.5	89.9	94.
r Bluff	97.0	89.7	93.
as City	100.7	108.2	104.
oseph	99.9	94.3	97.
othe	96.6	95.2	96.
sonville	96.2	102.9	99.
	98.6	81.7	91.2
son City	96.6	91.7	94.4
bia	97.7	93.2	95.7
a	96.7	92.4	94.8
	95.1	95.7	95.3
field	98.6	82.8	91.7
s	103.3	77.5	91.9
oint	102.5	75.4	90.6
City	100.3	75.1	89.2
Falls	104.5	76.8	92.3
	101.4	75.0	89.7
	102.7	75.0	90.5
la	103.1	77.5	91.8
ll	100.6	77.0	90.2
	99.3	76.3	89.1
	100.4	79.5	91.2
	98.6	73.0	87.3
us	96.8	70.6	85.3
	98.6	74.0	87.8
sland	98.7	77.8	89.5
s	97.8	78.1	89.1
	97.6	69.6	85.3
latte	98.1	78.3	89.4
e	100.0	66.6	85.3
	100.0	64.8	84.5
as	100.2	109.0	104.1
	98.7	99.3	99.0
	99.0	92.0	95.9
City	98.9	91.8	95.8
	97.6	[illegible]	[illegible]

Elizabeth City

	98.4	92.8	95.9
	96.4	74.6	86.8
	96.0	77.7	88.0
	95.8	70.6	84.7
	94.9	70.7	84.2
	97.3	94.5	96.1
	102.7	124.8	112.4
	100.1	124.8	111.0
	99.0	124.7	110.3
	100.9	124.9	111.5
	98.9	124.9	110.4
	98.4	122.3	108.9
	99.2	124.9	110.5
	99.2	124.8	110.5
	97.1	123.4	108.7
	99.0	122.8	109.5
	97.6	123.3	108.9
	99.5	122.5	109.6
	98.9	121.6	108.9
	99.4	123.9	110.2
	99.1	73.5	87.9
	99.0	73.5	87.8
	99.7	73.5	88.2
	100.4	73.5	88.5
	97.6	73.5	87.0
	97.1	73.5	86.7
es	97.1	70.2	85.2
	96.2	69.4	84.4
	97.7	73.4	87.0
	99.6	73.5	88.1
	99.7	73.5	88.2
	98.4	73.4	87.4
	104.2	167.0	131.9
	100.0	164.8	128.6
	98.3	164.8	127.6
	98.0	133.0	113.4
	98.3	133.0	113.6
	102.9	133.1	116.2
	98.3	133.0	113.6
	98.1	121.5	108.4
	101.1	164.8	129.1
	102.5	164.8	130.0
	103.0	164.8	130.2
	102.4	164.8	129.9
	100.7	164.8	129.0
	101.0	145.4	120.5
	102.6	164.8	130.0
	101.7	145.1	120.9
	97.0	99.8	98.2
	97.6	99.0	98.2
	100.4	118.5	108.4
	99.6	128.6	112.4
	99.0	117.6	107.2
	92.2	94.9	93.4
	96.8	88.6	93.2
	98.7	96.2	97.6
	96.7	93.3	95.2
	98.2	93.1	95.9
	98.2	100.1	99.0
	100.0	104.1	101.8
	97.5	105.5	101.0
	100.1	96.0	98.3
	96.5	87.9	92.7
	96.3	92.7	94.8
	99.0	56.2	80.2
	98.8	57.5	80.6
	99.0	55.1	79.7
	100.3	53.9	79.9
	96.1	53.9	77.5
	97.0	[illegible]	[illegible]

SUPERSTRUCTURE

Truss 0.9											
		Diagonal		Vertical		Horizontal					
Member Size		Li	n	Li	n	Li	n	Lin ft	Weight (k)	Weight (t)	
W14x	43	31	1	24	5			151	6.5	3.2	
	48	31	2					62	3.0	1.5	
	53	31	2					62	3.3	1.6	
	68	31	1					31	2.1	1.1	
	82	31	1					31	2.5	1.3	
	90	32	1					32	2.9	1.4	
	109	31	2					62	6.8	3.4	
	176	32	1					32	5.6	2.8	
	193	31	1					31	6.0	3.0	
	211			24	1			24	5.1	2.5	
W27x	102					41.5	1	41.5	4.2	2.1	
	178					41.5	1	41.5	7.4	3.7	
	307					20	4	80	24.6	12.3	
	336					20	4	80	26.9	13.4	
										53.4	tons

Truss H											
		Diagonal		Vertical		Horizontal					
Member Size		Li	n	Li	n	Li	n	Lin ft	Weight (k)	Weight (t)	
W14x	132	25	1					25	3.3	1.7	
	193	25	1	28	1			53	10.2	5.1	
	176	19.5	1					19.5	3.4	1.7	
	283			28	1			28	7.9	4.0	
	311	37.77	1			33.33	1	71.1	22.1	11.1	
	665			28	1			28	18.6	9.3	
W18x	311					33.33	1	33.33	10.4	5.2	
PG46-2	1291					40.16	1	40.16	51.8	25.9	
										63.9	tons

Truss J											
		Diagonal		Vertical		Horizontal					
Member Size		Li	n	Li	n	Li	n	Lin ft	Weight (k)	Weight (t)	
W14x	48			28	1			28	1.3	0.7	
	68	65	1	28	1	27	3	174	11.8	5.9	
	90	15.5	1					15.5	1.4	0.7	
	145			28	1			28	4.1	2.0	
										9.3	tons

Truss L											
		Diagonal		Vertical		Horizontal					
Member Size		Li	n	Li	n	Li	n	Lin ft	Weight (k)	Weight (t)	
W14x	257			25	1			25	6.4	3.2	
										3.2	tons

Truss 0.9											
		Diagonal		Vertical		Horizontal					
Member Size		Li	n	Li	n	Li	n	Lin ft	Weight (k)	Weight (t)	
W12x	96	127	1					126.5	12.1	6.1	
W14x	61			24	3			72	4.4	2.2	
	90			24	1			24	2.2	1.1	
	109			24	1			24	2.6	1.3	
	120			24	1			24	2.9	1.4	
	145			24	1			24	3.5	1.7	
	257	31	1					31	8.0	4.0	
	233	31	1					31	7.2	3.6	
W24x	94					41.5	1	41.5	3.9	2.0	
	131					20	2	40	5.2	2.6	
	176					20	2	40	7.0	3.5	
	192					121.5	1	121.5	23.3	11.7	
										41.2	tons

Truss H											
		Diagonal		Vertical		Horizontal					
Member Size		Li	n	Li	n	Li	n	Lin ft	Weight (k)	Weight (t)	
W14x	68			28	1			28	1.9	1.0	
	81			28	1			28	2.3	1.1	
	120					21	1	21	2.5	1.3	
	132					33.33	2	66.66	8.8	4.4	
	145			28	1			28	4.1	2.0	
	211	69	1					69	14.6	7.3	
	655	18.8	1					18.8	12.3	6.2	
PG46-2	1291					19.5	1	19.5	25.2	12.6	
										35.8	tons

Truss J											
		Diagonal		Vertical		Horizontal					
Member Size		Li	n	Li	n	Li	n	Lin ft	Weight (k)	Weight (t)	
W14x	90			28	1			28	2.5	1.3	
	120			28	1			28	3.4	1.7	
	132					27	1	27	3.6	1.8	
	176			28	1			28	4.9	2.5	
	193	65	1					65	12.5	6.3	
	370	15.5	1					15.5	5.7	2.9	
W24x	192					27	1	27	5.2	2.6	
PG46-2	1291					40.5	1	40.5	52.3	26.1	
										45.1	tons

Truss L											
		Diagonal		Vertical		Horizontal					
Member Size		Li	n	Li	n	Li	n	Lin ft	Weight (k)	Weight (t)	
W14x	82			14	1			14	1.1	0.6	
	90					21	1	21	1.9	0.9	
	211	25	1					25	5.3	2.7	
	233			14	1			14	3.3	1.6	
PG46-2	1291					40.5	1	40.5	52.3	26.1	
										32.0	tons

Truss N.2

Member	Size	Diagonal Li	Diagonal n	Vertical Li	Vertical n	Horizontal Li	Horizontal n	Lin ft	Weight (k)	Weight (t)
W14x	48			24	2			48	2.3	1.2
	120			24	1			24	2.9	1.4
	159	36	1					36	5.7	2.9
	211	29	1					29	6.1	3.1
	233			24	1			24	5.6	2.8
	257	34	1					34	8.7	4.4
W18x	211					70	1	70	14.8	7.4
W24x	370					70	1	70	25.9	13.0
										36.0 tons

Truss X

Member	Size	Diagonal Li	Diagonal n	Vertical Li	Vertical n	Horizontal Li	Horizontal n	Lin ft	Weight (k)	Weight (t)
W14x	68			25	2			50	3.4	1.7
	159	33.82	1					33.82	5.4	2.7
	211	34.5	1					34.5	7.3	3.6
	233	34.5	1					34.5	8.0	4.0
	311	32.2	1					32.2	10.0	5.0
	342	54	1					54	18.5	9.2
	398	57	1					57	22.7	11.3
	455	21.25	1					21.25	9.7	4.8
W27x	129					10.2	1	10.2	1.3	0.7
	146					17	1	17	2.5	1.2
	539					20.25	2	40.5	21.8	10.9
PG46-3	748					44.18	1	44.18	33.0	16.5
PG56-1	2369					42	1	42	99.5	49.7
										121.6 tons

Truss N.2

Member	Size	Diagonal Li	Diagonal n	Vertical Li	Vertical n	Horizontal Li	Horizontal n	Lin ft	Weight (k)	Weight (t)
W38x	201					70	1	70	14.1	7.0
PG46-3	748					70	1	70	52.4	26.2
										26.2 tons

Truss X

Member	Size	Diagonal Li	Diagonal n	Vertical Li	Vertical n	Horizontal Li	Horizontal n	Lin ft	Weight (k)	Weight (t)
W10x	15					20.25	0.5	10.13	0.2	0.1
W14x	61			28	1			28	1.7	0.9
	90			28	1			28	2.5	1.3
W16x	31					20.25	1	20.25	0.6	0.3
	84					20.25	1	20.25	1.7	0.9
W18x	60					20.25	2	40.5	2.4	1.2
W27x	84					20.25	1	20.25	1.7	0.9
W36x	135					20.25	1.5	30.38	4.1	2.1
W44x	335					20.25	3.5	70.88	23.7	11.9
										19.3 tons

FOUNDATIONS

REINFORCEMENT

DYWIDAG Prestressing Steel Threadbar System

DYWIDAG Prestressing Steel Threadbar is a high tensile alloy steel bar which features a coarse right-hand thread over its full length. The system is proven worldwide and offers versatility in a range of applications.

Manufactured in accordance with the German Certificate of Approval (Deutsches Institut für Bautechnik), the system also offers general conformance with BS 4486 : High Tensile Steel Bars for Prestressing of Concrete. During the steel making process, the threadbars are hot rolled, quenched and tempered, followed by cold working and further tempering, to achieve the necessary performance.

DYWIDAG Prestressing Steel Threadbars, 15mm - 75mmØ are suitable for all static loading applications. Additionally, for post-tensioning and dynamic applications, DYWIDAG Prestressing Steel Threadbars 26.5mm - 40mmØ, see note (c) below, offer a fatigue resistance in excess of 2 million load cycles over a tensile range of 630 - 682N/mm² as specified in the European Technical Approval No. ETA - 05/0123 and ETAG 013. Stress relaxation when loaded to 70% *f*pu is less than 3.5% over a 1000 hour period in accordance with BS4486.

Key features of the system are:
- Fully threaded bar – can be cut and coupled at any point.
- Coarse pitch threadform (d/2), right-hand, with two faces ensuring the thread is self cleaning. Ideal for construction site use.
- Low relaxation steel – minimum relaxation during service life.
- Prestressing grade steel – high strength offers weight savings and reduced working diameters.

Technical Data for Prestressing Steel Threadbar

Nominal Diameter	Steel Grade	Ultimate Strength *f*pu	0.1% (a) Proof Strength	70% (b) Ultimate Strength	50% Ultimate Strength	Cross Sectional Area	Diameter Over Threads	Thread Pitch	Bar Weight
mm	N/mm²	kN	kN	kN	kN	mm²	mm	mm	kg/m
15	900/1100	195	159	136	98	177	17	10	1.44
20	900/1100	345	283	241	173	314	23	10	2.56
26.5	950/1050	579	523	405	290	551	30	13	4.48
32	950/1050	844	764	591	422	804	36	16	6.53
36	950/1050	1069	967	748	535	1018	40	18	8.27
40	950/1050	1320	1194	924	660	1257	45	20	10.21
47	950/1050	1822	1648	1275	911	1735	52	21	14.10
57	835/1035	2671	2155	1870	1335	2581	64	21	20.95
65	835/1035	3447	2771	2413	1724	3318	71	23	27.10
75	835/1035	4572	3645	3200	2286	4418	82	24	35.90

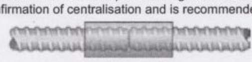
(a) 0.1% Proof Stress also referred to, in general terms, as Yield Strength - *fy*. (b) For geotechnical applications 75% *f*pu may be used for proof testing.
(c) Approval Standards: Ø 26.5 - 47mm (grade 950/1050N/mm²) ETA 05/0123 and ETAG 013. Øs 15 & 20mm (grade 900/1100N/mm²) formtie approvals. Øs 57 - 75mm (grade 855/1035 N/mm²) system approval.

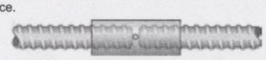
Modulus of Elasticity: E = 205,000 N/mm² +/- 5%.
Stock Lengths: 15mm - 20mmØ bars, 6.0m; 26.5mm - 75mmØ bars, 12.0m. Tolerances +/- 50mm.
All bar diameters can be cut to length to suit customer requirements.

Couplers for Threadbars

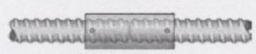
Couplers enable prestressing steel threadbars to be coupled or extended, reliably and efficiently. Coupler strength (for bar Øs 26.5 - 47mm) = 1.27 x Yield Strength, which equates to 1.15 x Ultimate Strength, in accordance with German Approval Certificates. Coupler strengths for other prestressing steel bar grades (bar Øs 15 & 20mm, and 57 - 75mm) exceed the published Ultimate Bar Strengths and are covered by separate approvals (see note C, Technical Data).

Precautions should be taken to ensure that the coupler remains centrally located. This can be achieved through the use of grub screws and/or a centre pin. Marking the two bars with paint or similar at half a coupler length prior to engagement provides visual confirmation of centralisation and is recommended as good working practice.


standard coupling


centre pin - optional


paint - visual indicator


grub screws - optional

DSI Prestressing Steel Threadbar

DRIVEN PILES

Depth	68	ft	Type	Do (in)	t (in)	Di	Astl	plf	Embd.(ft)	L	
fy	80	ksi	1	13.375	0.5	12.375	20.2	68.8	11	79	ft
f'c	5000	psi	2	13.375	0.5	12.375	20.2	68.8	16	84	ft
dstl	490	PCF									
	0.284	pci									

Diameter		Area	
75	mm	4418	mm2
2.95	in	6.848	in2

RS Means p 290					
Dia	Mat'l	Sales	Labor	Equip	O&P
12	32.00	5%	6.80	4.41	15%
13.375	33.72	5%	7.59	4.92	15%
14	34.5	5%	7.95	5.15	15%
Thick Wall Addition					
+	0.93	5%	0	0	15%
Reinforcement					
+	0.83	5%	0.34	0	15%
Final Cost Figures					
Dia	Mat'l	Sales	Labor	Equip	O&P
13.38	154.84	5%	30.99	4.92	15%
Loc	1.042		1.670	1.319	
Time					1.01
			Total	$	238.75

Current	Type	n	Cost
	1	2	37722.01
	2	10	200547.39
	Total	$	238269.40
Proposed	Type	n	Cost
	1	5	94305.02
	2	12	240656.87
	Total	$	334961.90
Difference		$	96692.49