# 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.

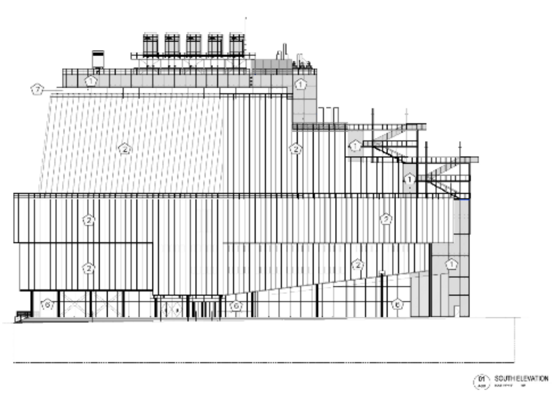


**Figure 3:** South Elevation showing modular façade (A-007)

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.

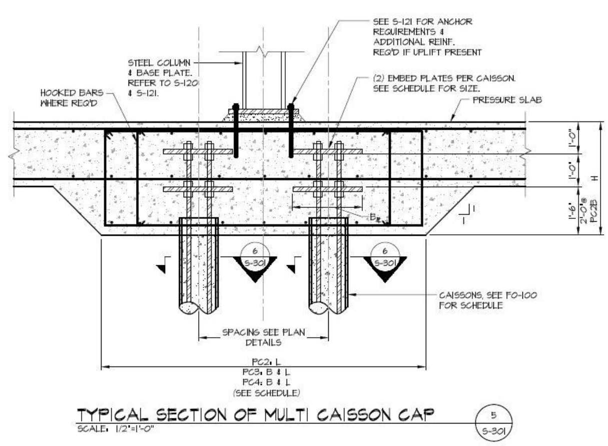



**Figure 4:** Pile cap section (S-301)

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.

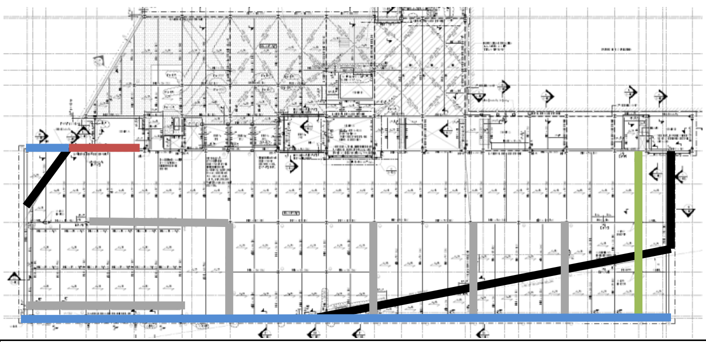


**Figure 5:** Level 5 framing plan showing regular layout against building footprint (S-105)

- 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.

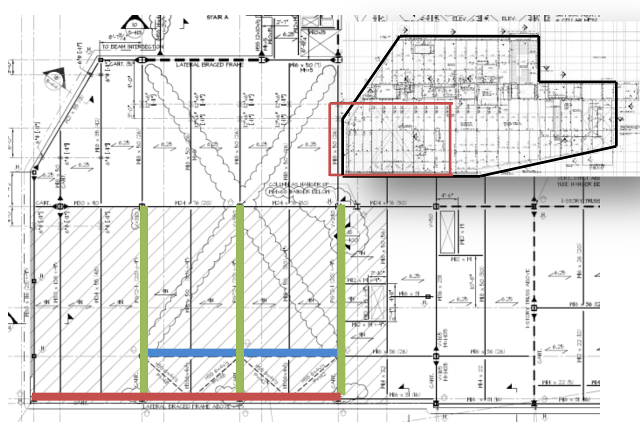


**Figure 6:** Level 3 framing plan showing transfer girders and lateral braced frames (S-103)

- 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.

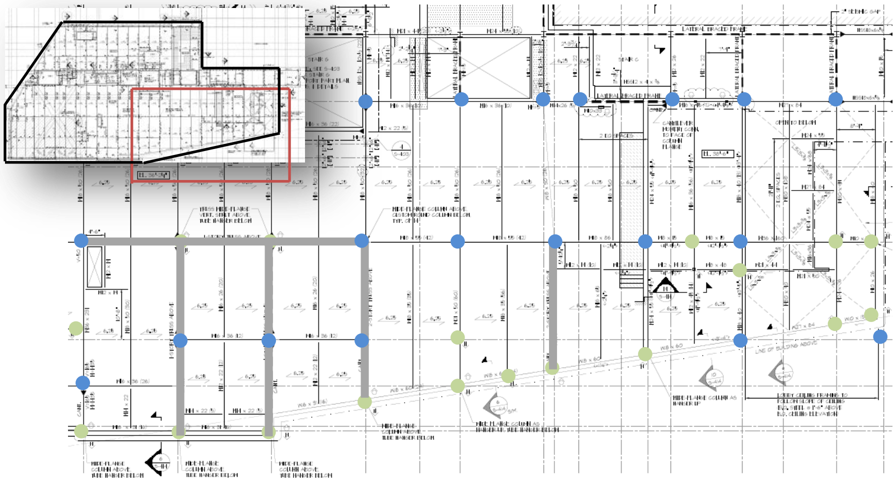


**Figure 7:** Level 3 framing plan showing hangers and outline of hung/cantilevered portion of building (S-103)

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.

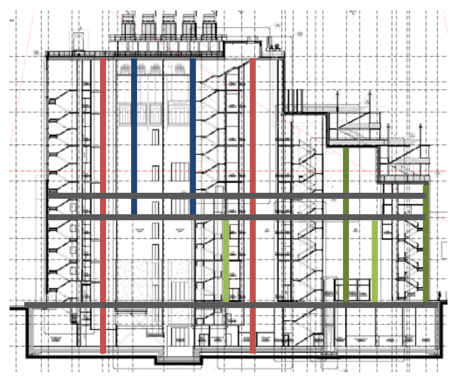


**Figure 8:** Section cut showing N-S braced frames at staggered heights (A-212)

**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

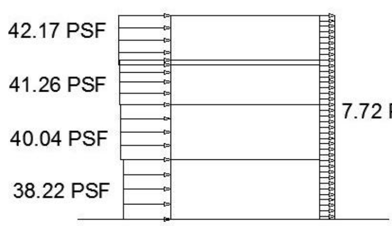
As mentioned above, the wind loads in both directions were found using Analytical Procedure (Method 2) in ASCE 7-05 chapter 6 using assumptions that simplify the geometry and environment of the building. Using the factors in Figure 14 below (calculations in Appendix B), the wind pressures were calculated 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%).

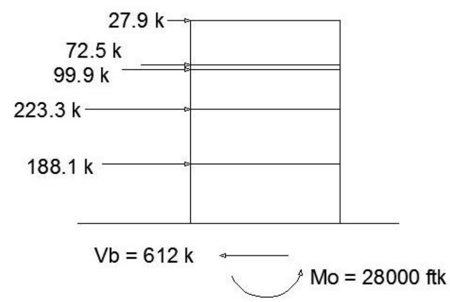
Figure 15 below summarize the revised wind load calculations. The base shears and overturning moments were found for both the North-South (Y) and East-West (X) directions by creating equivalent lateral forces at each story level. More detailed calculations provided in Appendix B show that AAM must resist 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 | - |

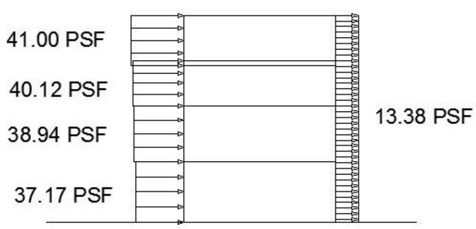
**Figure 14 (Left):** Wind factors for ASCE 7-05 calculations

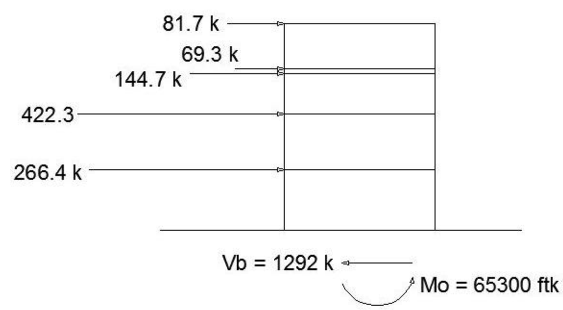
**Figure 15 (Below):** ASCE 7-05 Wind Pressures and equivalent lateral forces




**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

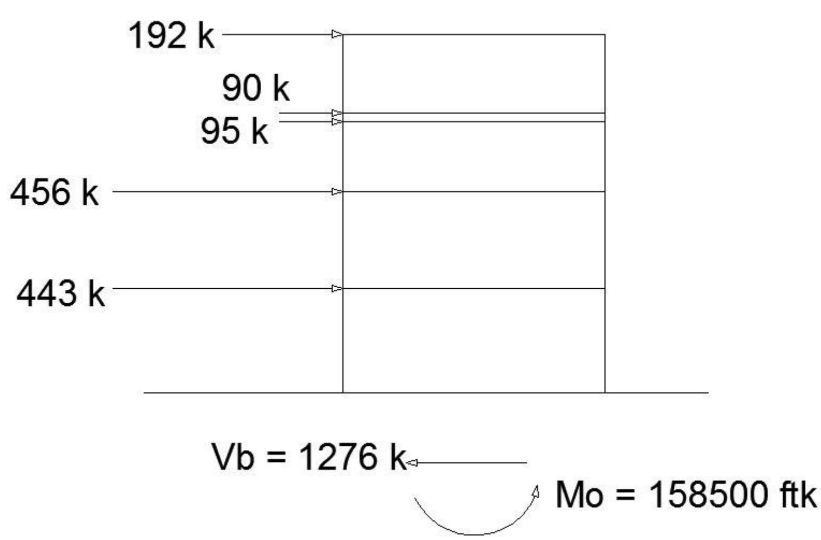



**Figure 17:** Equivalent Lateral Force Procedure Summary

# PROBLEM STATEMENT

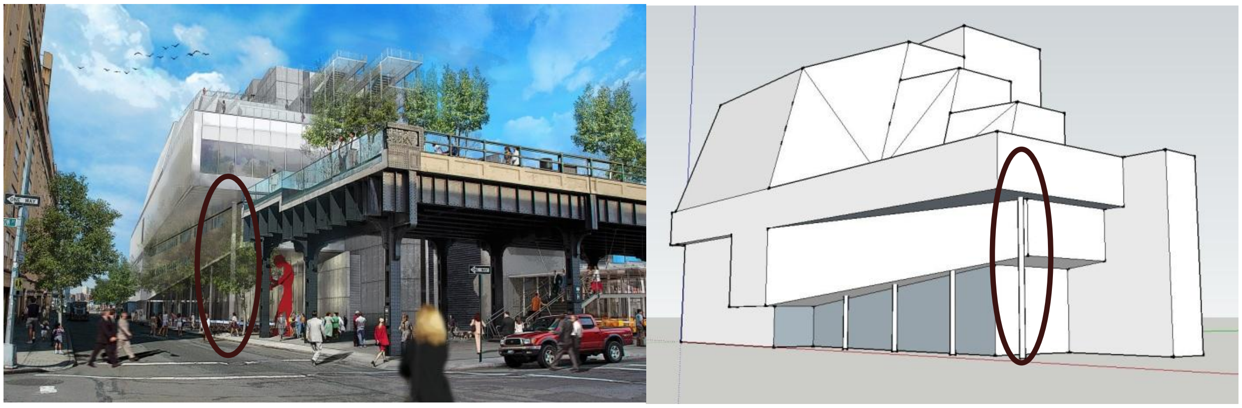


**Figure 18:** Rendering and Sketchup model showing column 3-M.5 from SE corner

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.

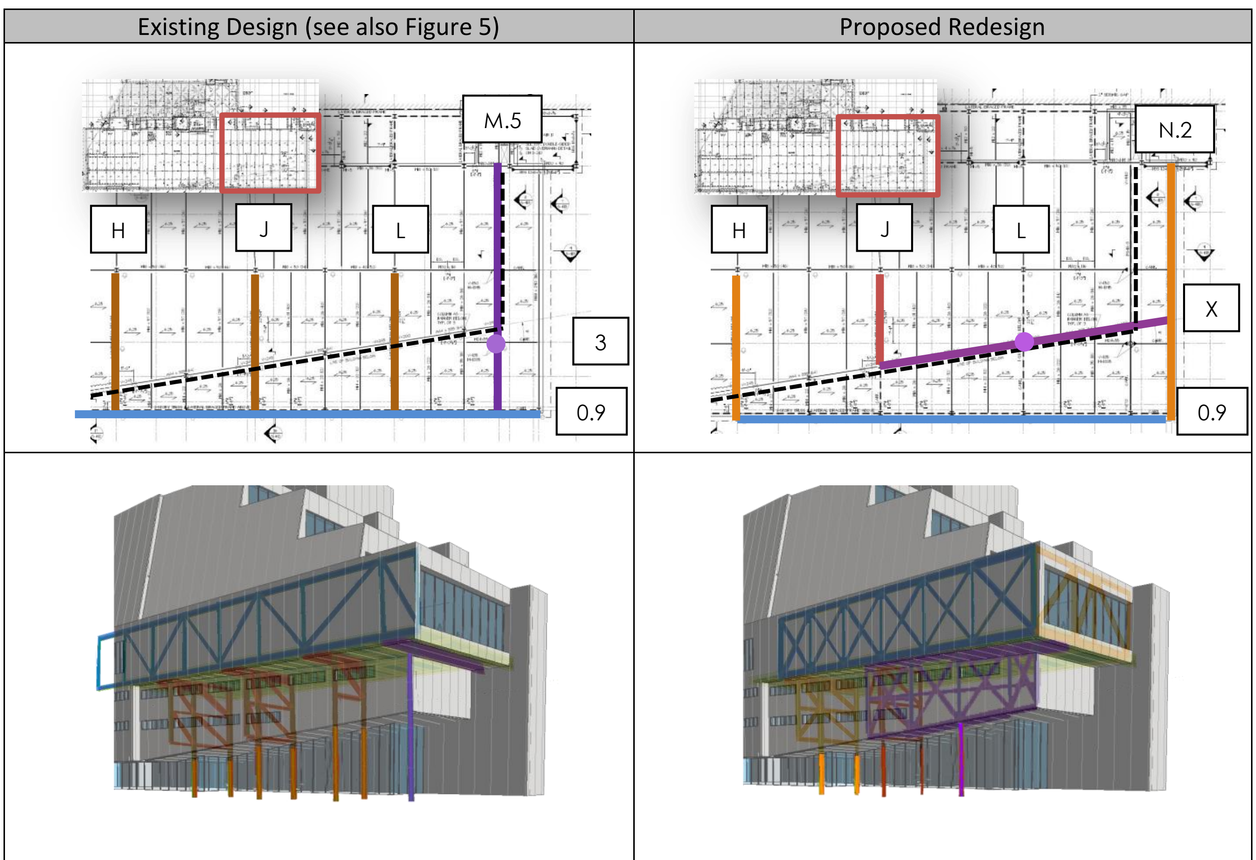



**Figure 19:** Plan and Perspective Comparison of Existing(left) and Proposed (right) Load Paths (S-105)

# 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 |

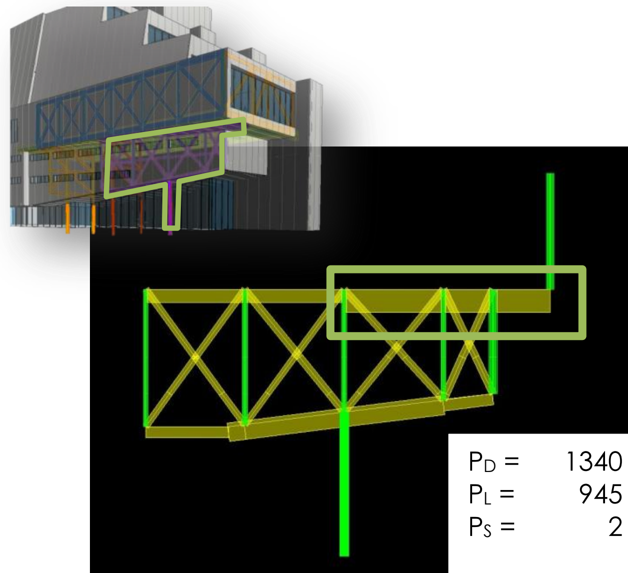
**Figure 21:** HSS Round Column Capacities

## 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.

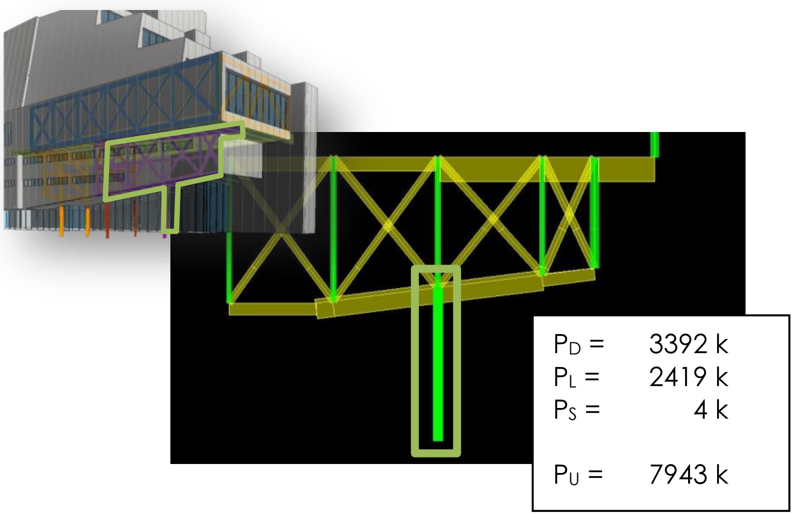
| | Existing Shapes | | PG56-1 | | Capacities | | |
|---|---|---|---|---|---|---|---|
| Lb | n/a | | 20 | ft | ϕMn | 41571 | ft-k |
| D | n/a | | 56 | in | ϕVn | 3402 | k |
| B | 18, 20 | in | 24 | in | ϕTn | 25245 | k |
| tf | 2, 4, 8 | in | 10 | in | ϕPn | 27541 | k |
| tw | 1, 2 | in | 2.25 | in |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

**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.




| | | |
|---|---|---|
| **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

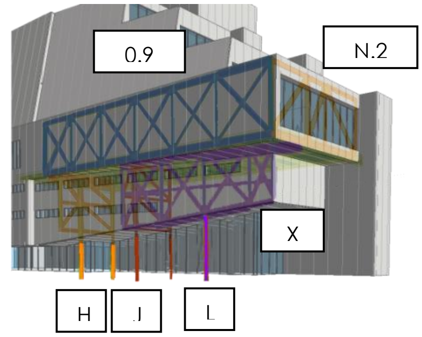



**Figure 25:** Truss Name Summary

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) |
| **$1.2(D + F + T) + 1.6(L + H) + 0.5(L_r$ or $S$ or $R)$** | **(Equation 16-2)** |
| $1.2D + 1.6(L_r$ or $S$ or $R) + (f_1L$ or $0.8W)$ | (Equation 16-3) |
| $1.2D + 1.6W + f_1L + 0.5(L_r$ or $S$ or $R)$ | (Equation 16-4) |
| $1.2D + 1.0E + f_1L + f_2S$ | (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.

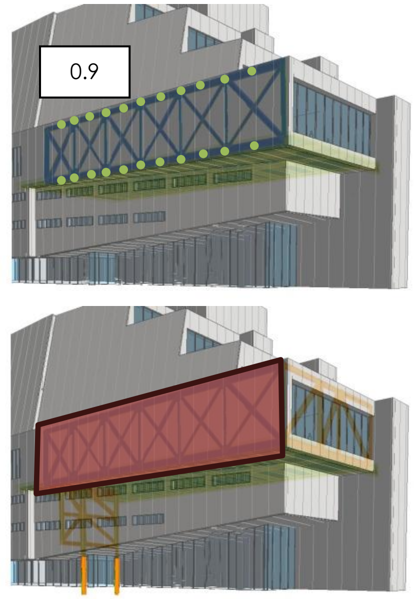



**Figure 26:** Truss 0.9 Location and Load Path Orientation

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.

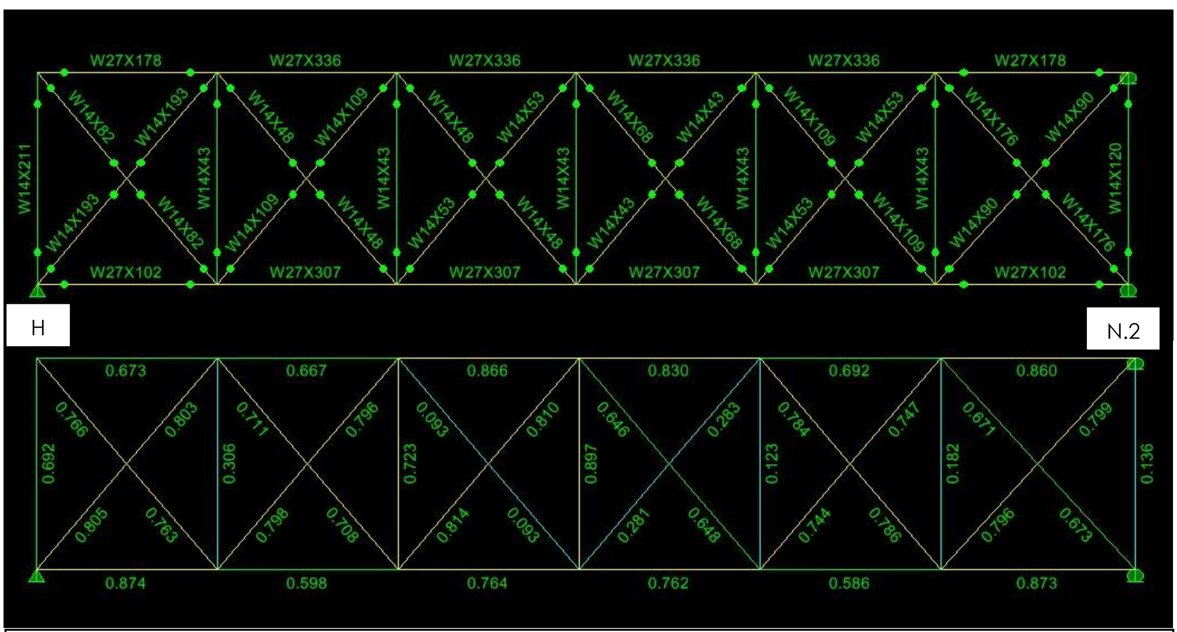



**Figure 27:** Truss 0.9 Modeling input and Design Results

### 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.

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.

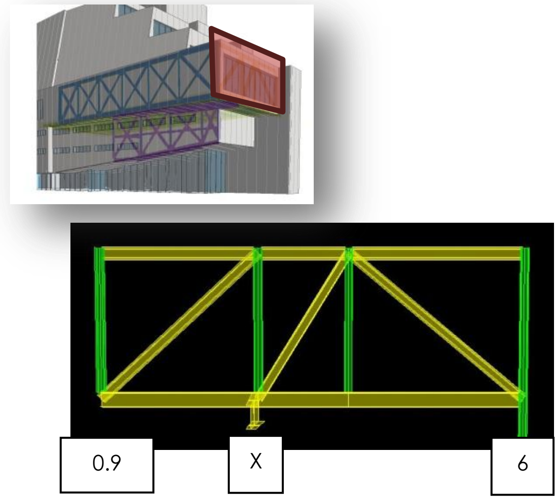



**Figure 28:** Truss N.2 Location and Load Path Orientation

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

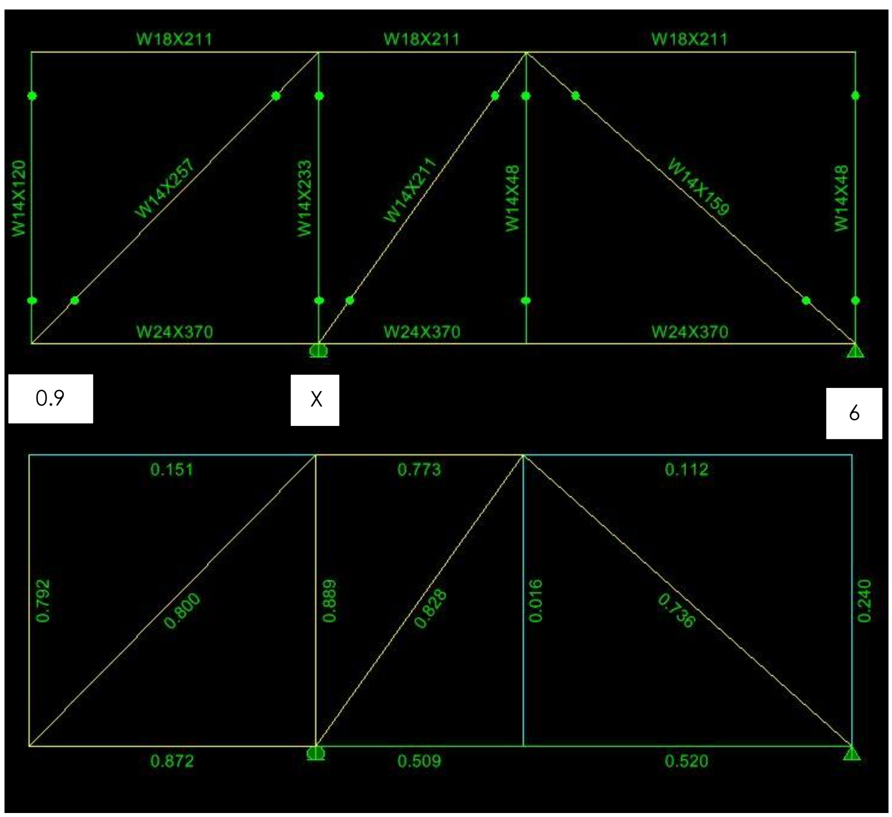



**Figure 29:** Truss N.2 Modeling input and Design Results

### Truss X

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.

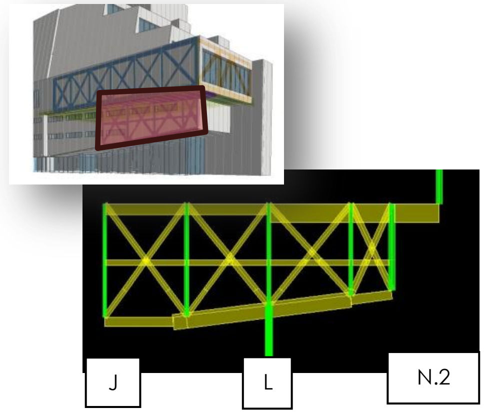



**Figure 30:** Truss X Location and Load Path Orientation

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.

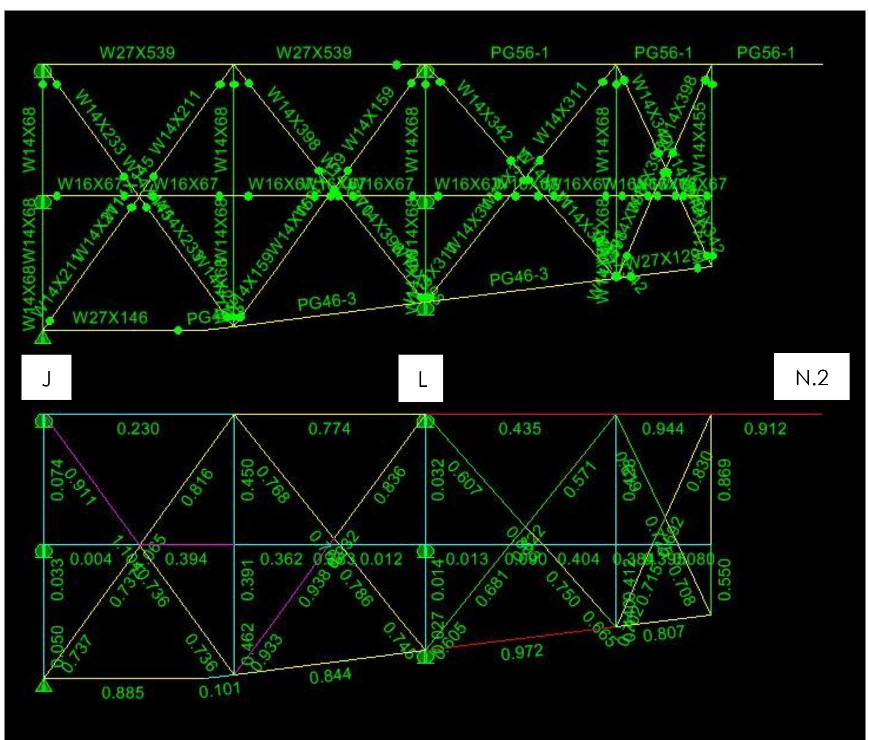



**Figure 31:** Truss X Modeling input and Design Results

## 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.

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.

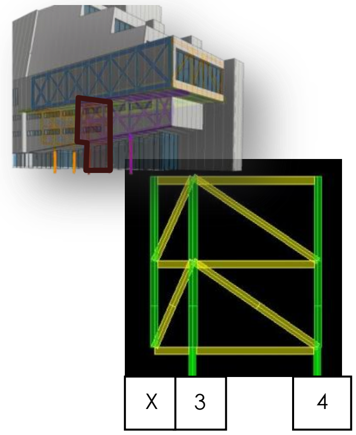



**Figure 32:** Truss J Location and Load Path Orientation

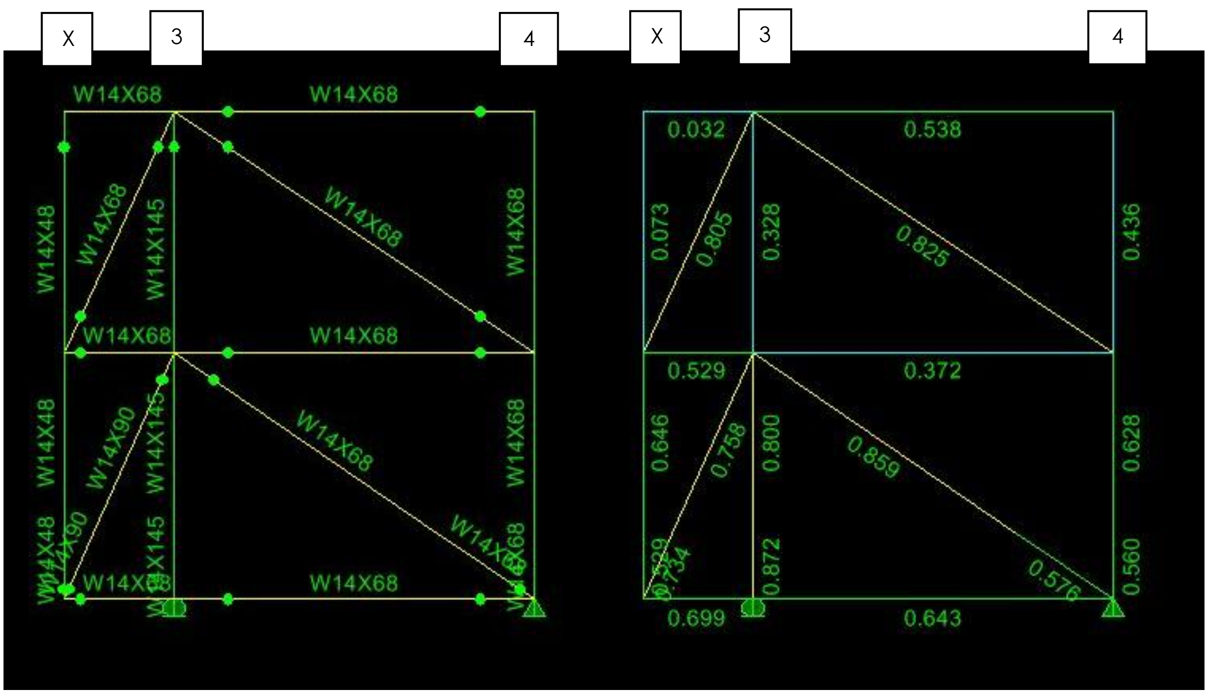



**Figure 33:** Truss J Modeling Input and Design Results

## TRUSS H

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.

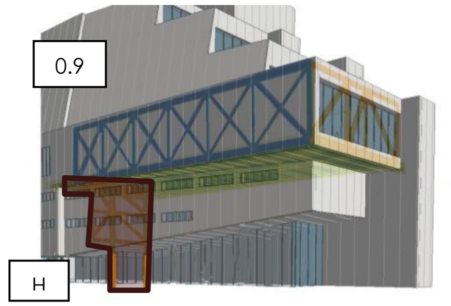



**Figure 34:** Truss H Location and Path

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.

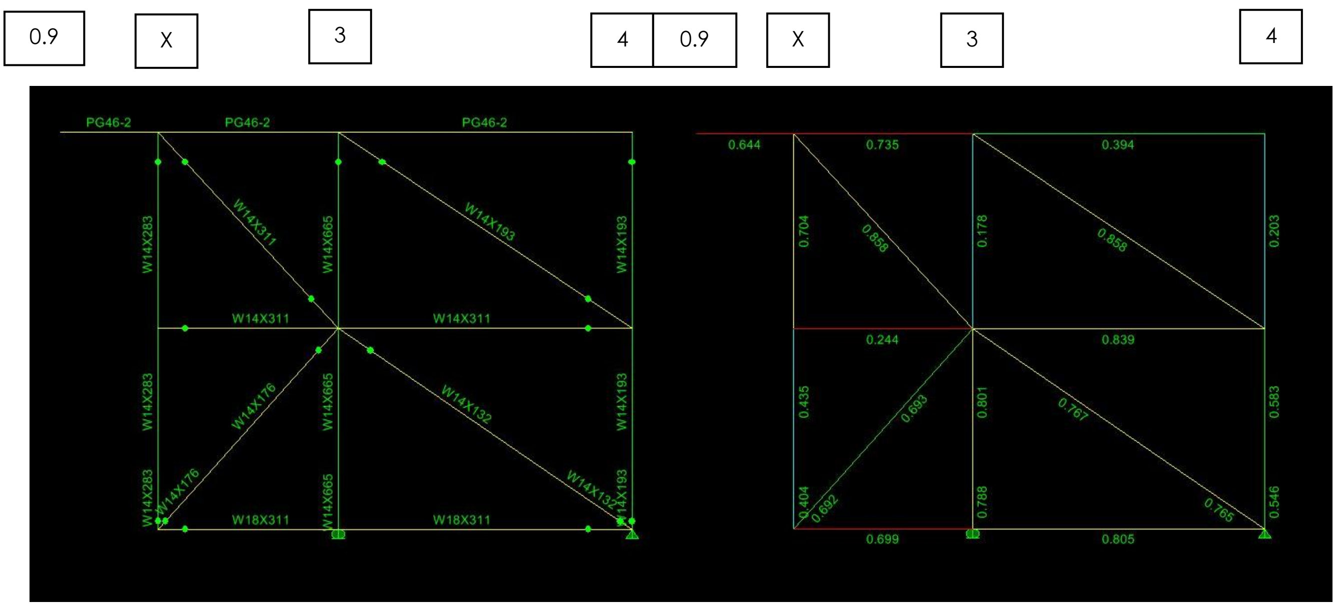



**Figure 35:** Truss H Modeling Input and Design Results

# ETABS Verification

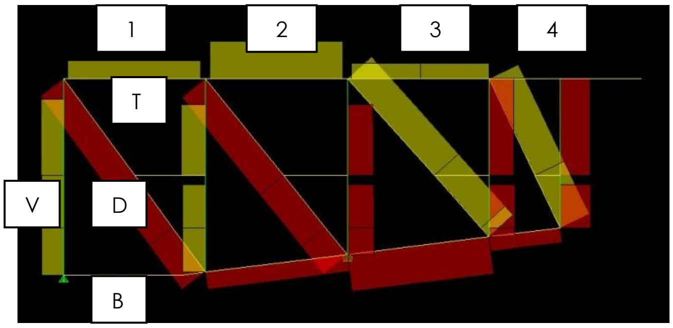



**Figure 36:** Nomenclature for Truss X Variation

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).

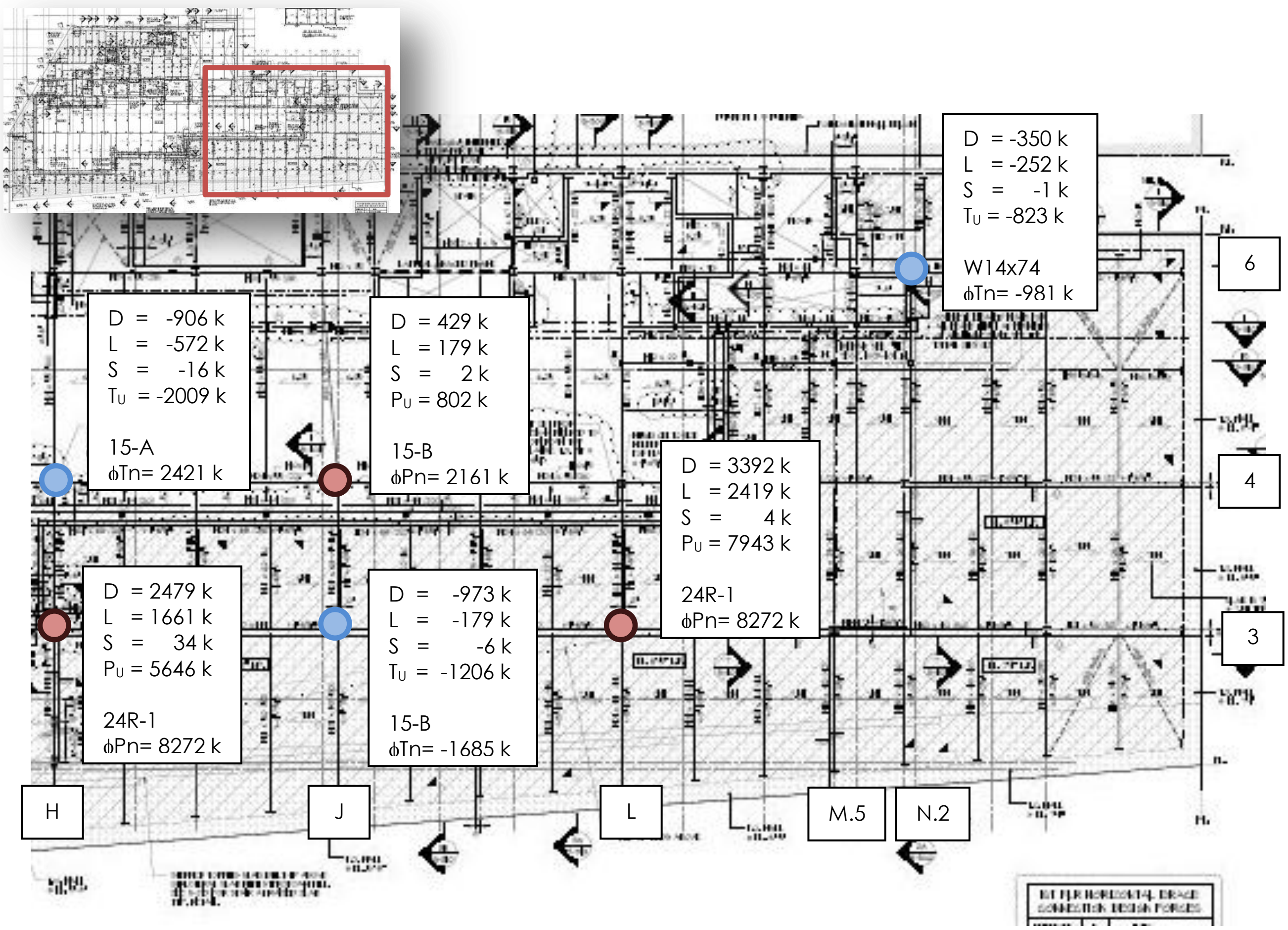



**Figure 38:** Truss Support Columns

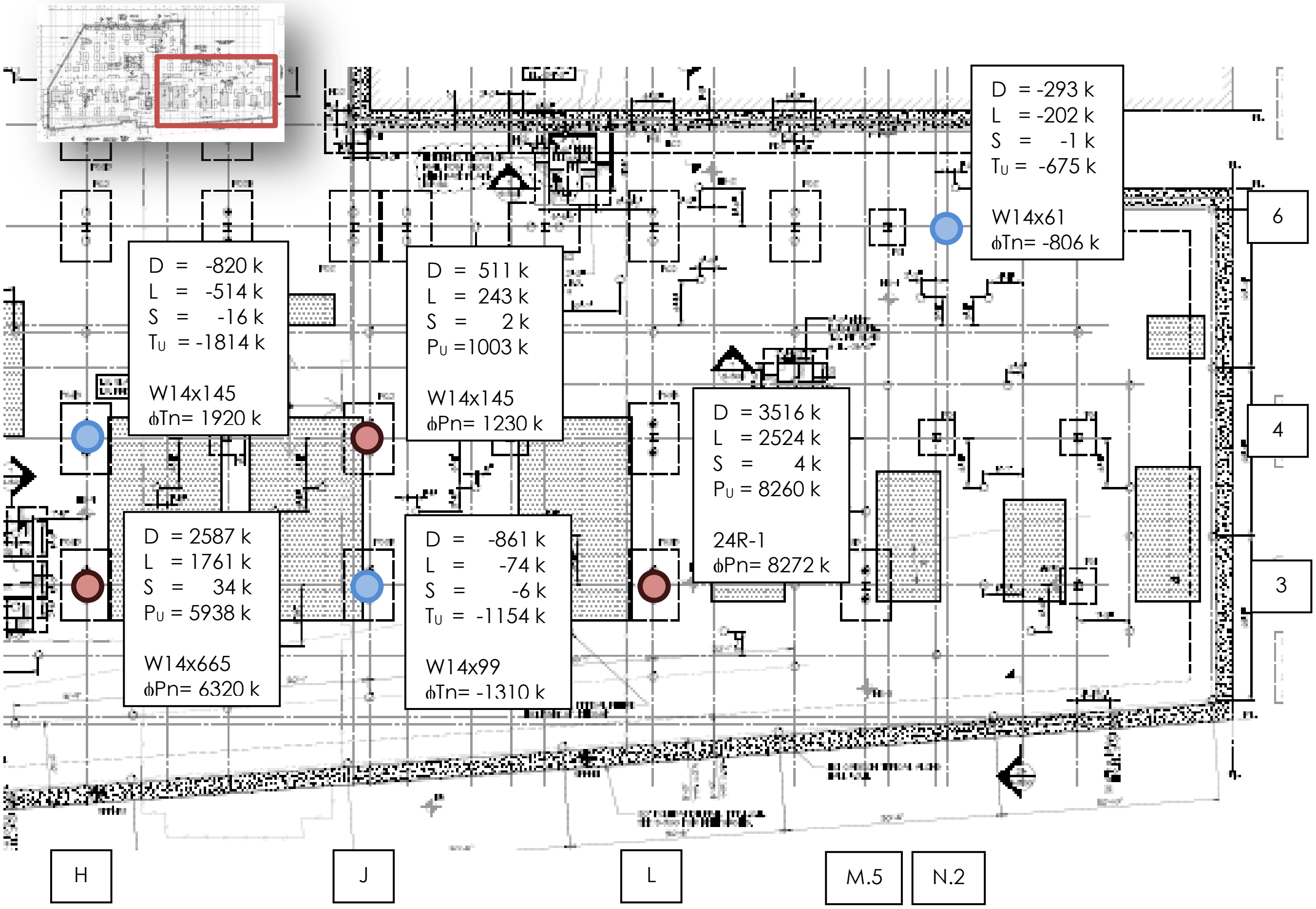



**Figure 39:** Substructure for Truss Support Columns
Tension Support
Compression Support

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.

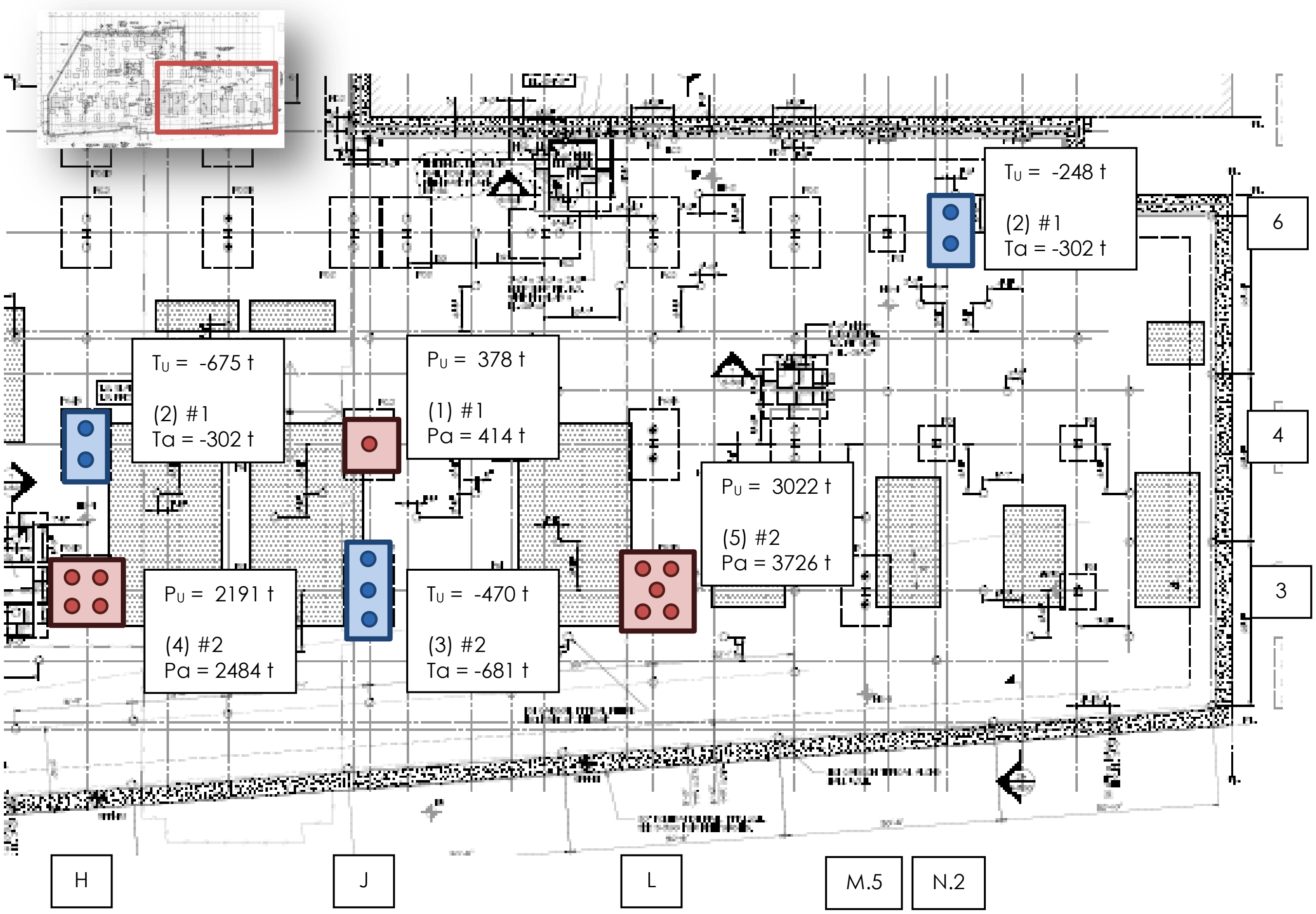
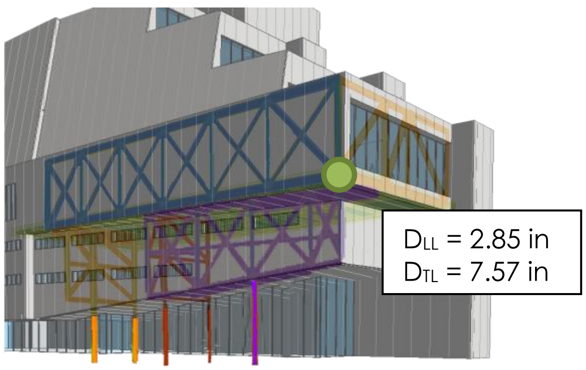



**Figure 41:** Pile Requirement Summary
Tension Support
Compression Support

# 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.

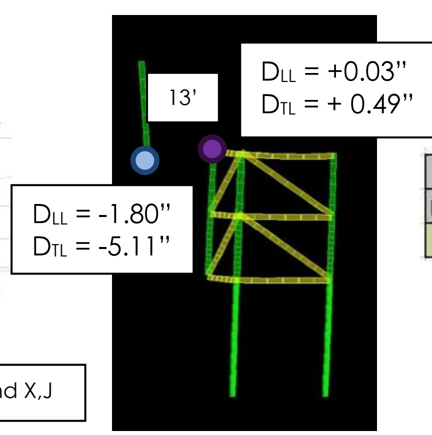



| Cantilever | | | |
|---|---|---|---|
| LL | Dalw | TL | Dalw |
| 2.85 | 3.06 | 7.57 | 4.59 |

**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

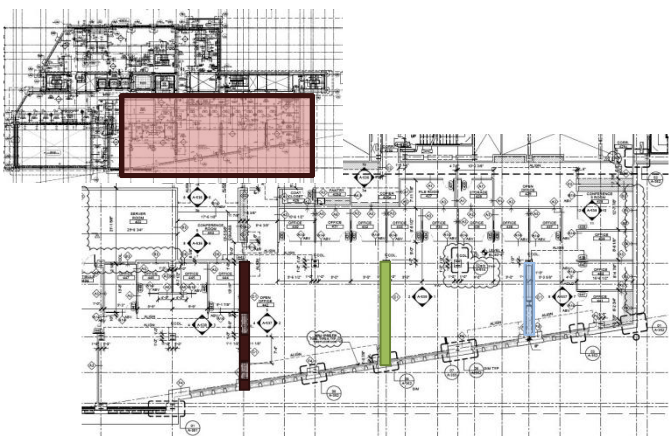
# ARCHITECTURE CONSIDERATIONS

## OVERVIEW

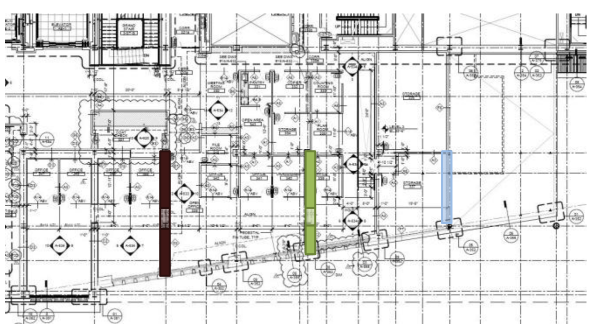
Respect for the current architectural scheme was a crucial consideration in the redesign of AAM. The office spaces on levels 3 and 4 are connected by passages through the existing truss systems. Entire systems and components such as section Truss N.2 and section PG56-1 were designed specifically to mitigate 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

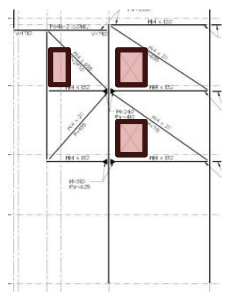
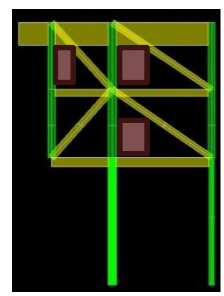
As mentioned above, the open office spaces on levels 3 and 4 are broken by gravity trusses which support the upper floors. In order to allow movement between these spaces, the web openings in the trusses were utilized by the architect. Figure 44 below displays how these openings were maintained in Trusses 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

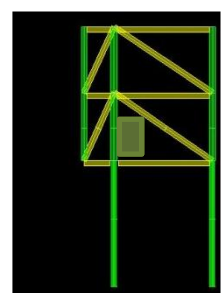


Level 3

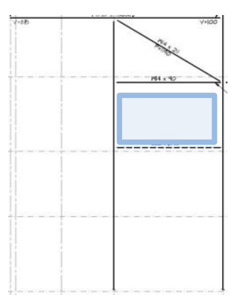
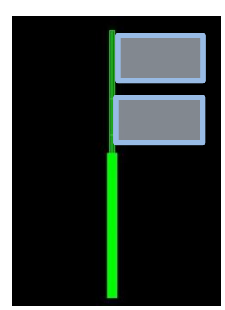
Truss H

Existing



Proposed

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.

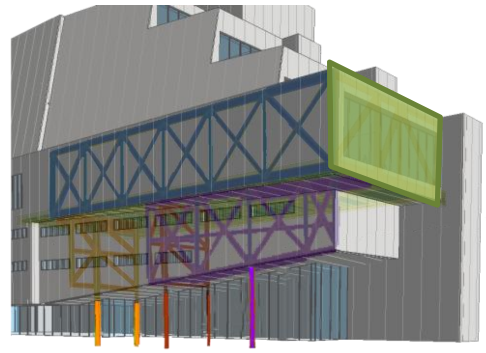


**Figure 45:** Location of Truss N.2

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.

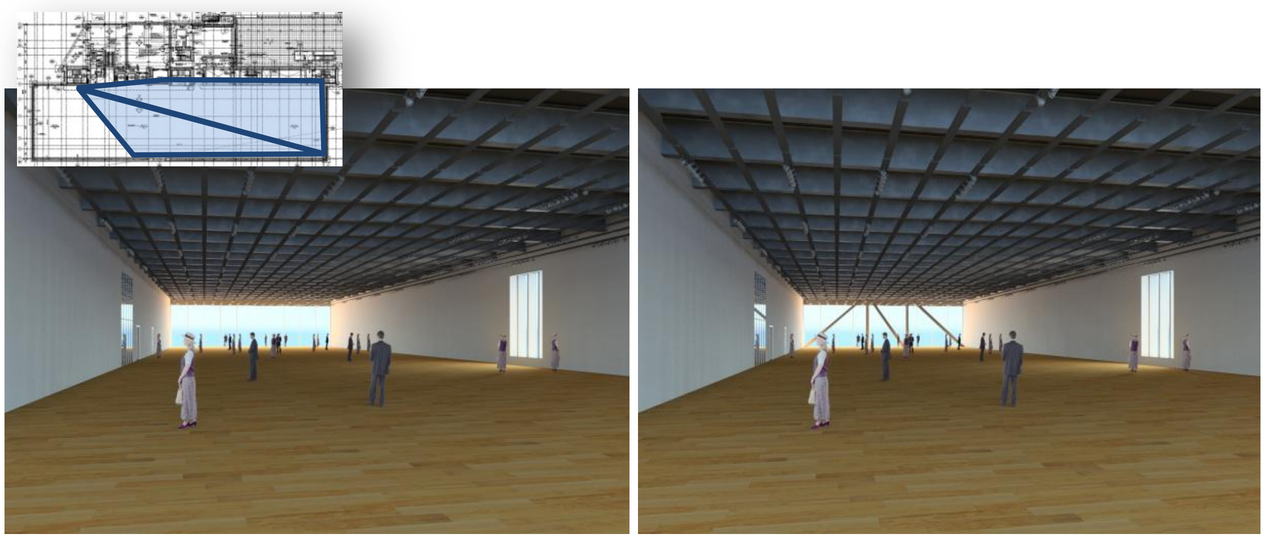


**Figure 46:** Interior Renderings of Level 5 Gallery Space with Current (top) and Proposed (bottom) Designs (A-105)

## 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.

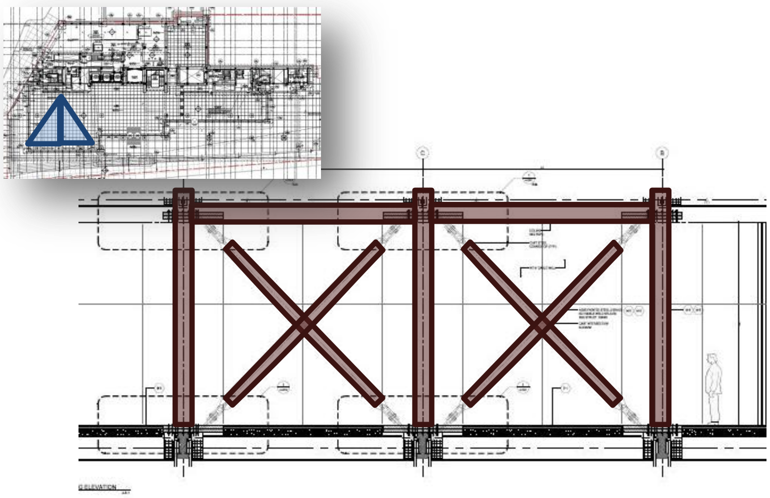


**Figure 47 (Left):** AESS in AAM lobby (A-399)

**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.

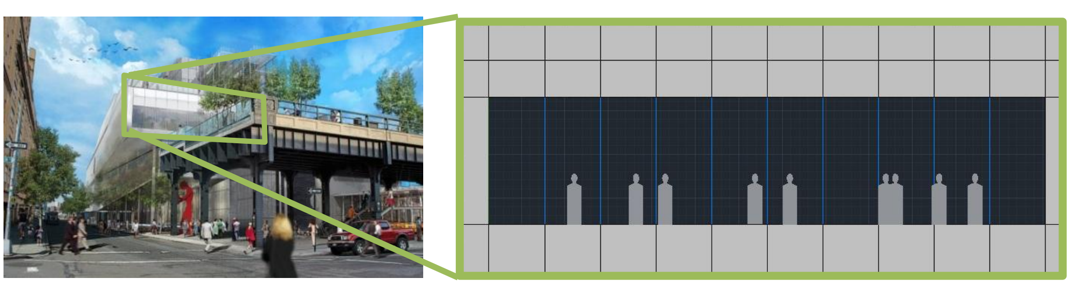
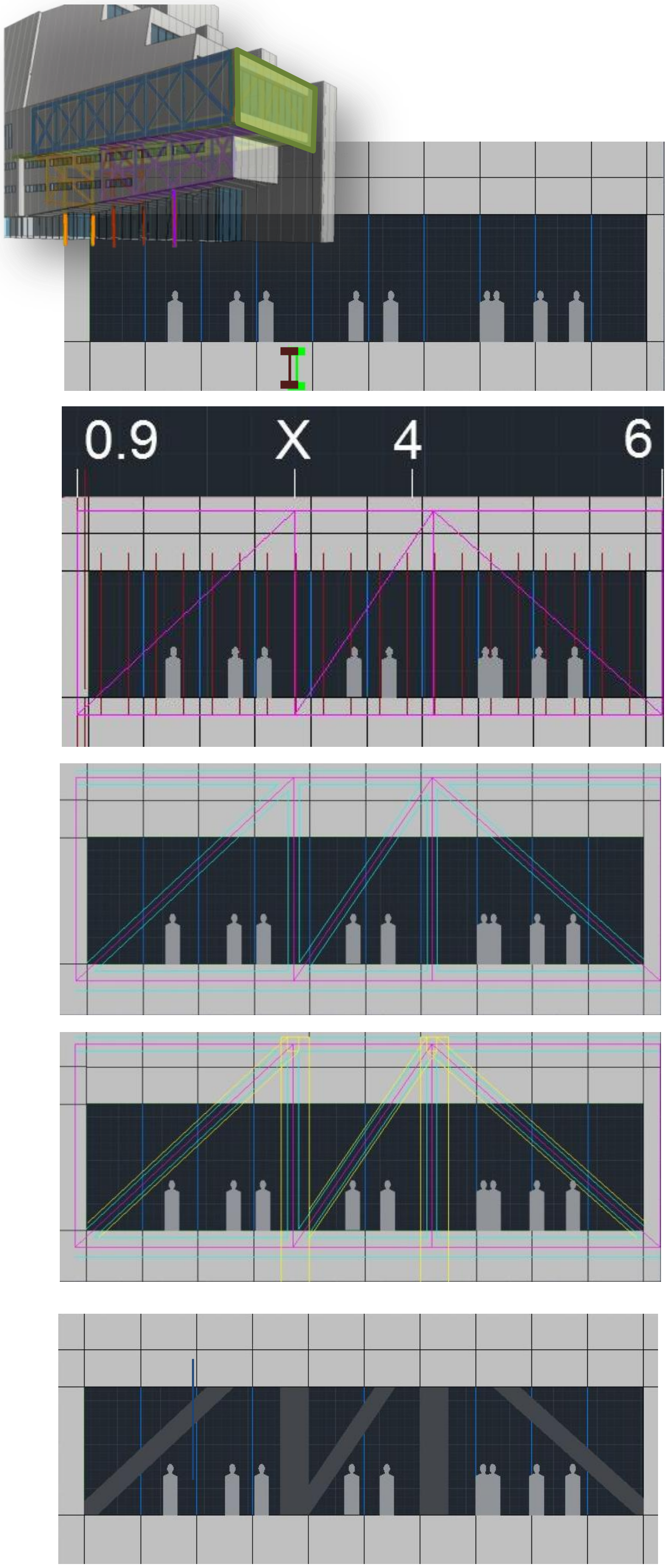


**Figure 49:** Current Façade Design




(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

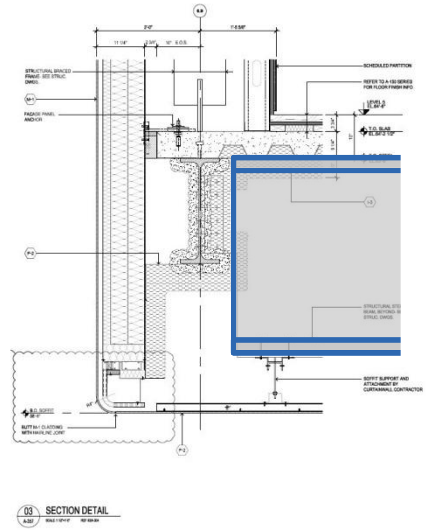



**Figure 52:** Section showing envelope dimensions at cantilever (3: A-357)

Plate Girder Outline

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.

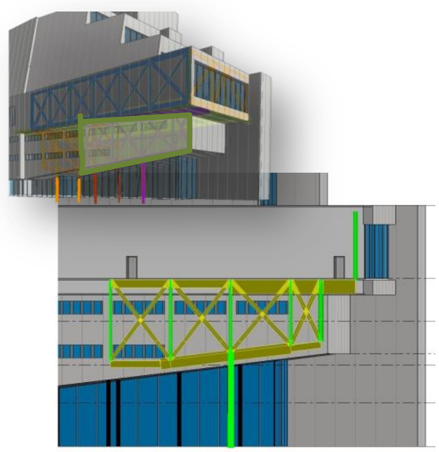
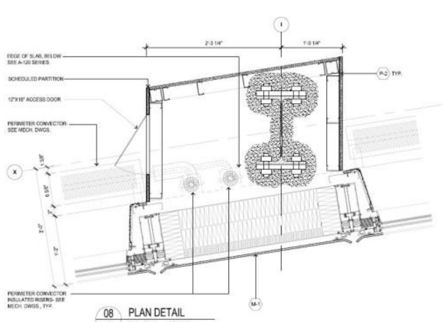


**Figure 53 (Above):** Structural/architectural conflict on levels 3 and 4.

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




## 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.

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

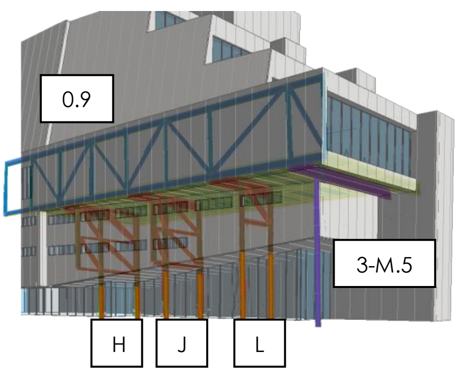
## OVERVIEW

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.

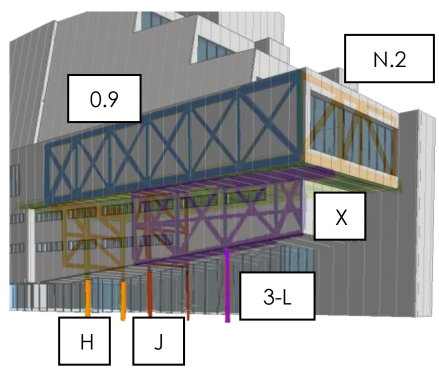
## COST

### 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.

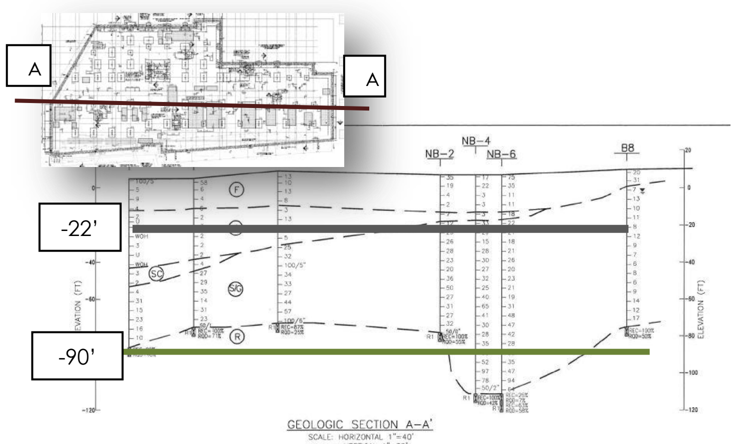



**Figure 57:** Geotechnical Section A-A

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

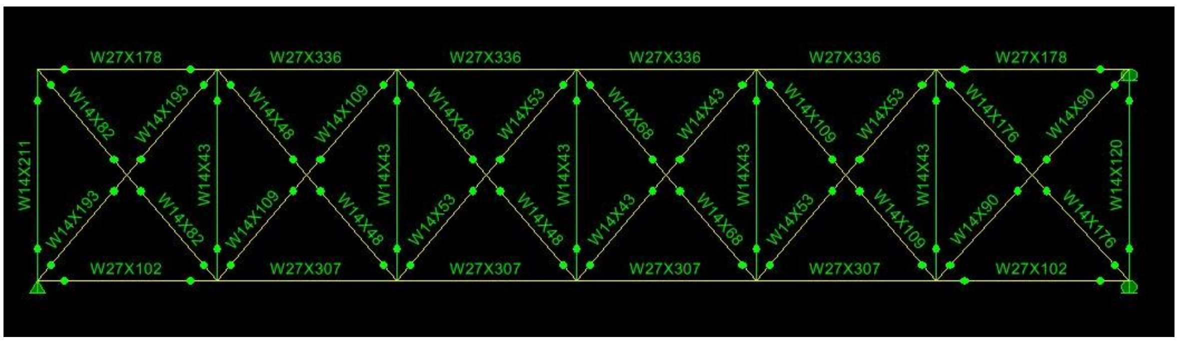



**Figure 59:** Truss 0.9 Constructability Concerns

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

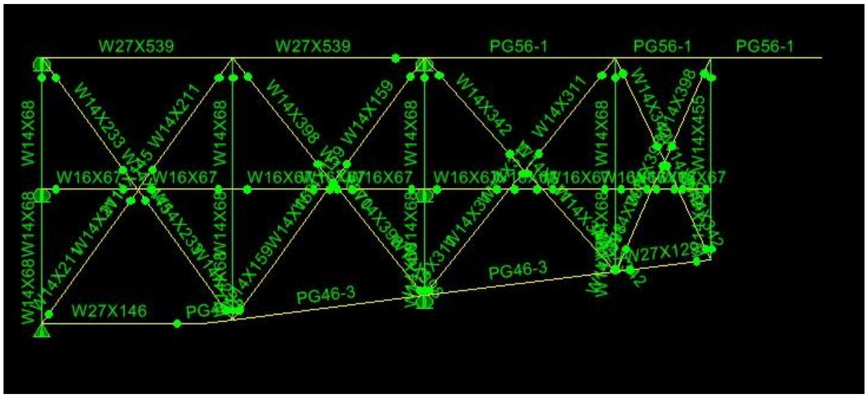



**Figure 60:** Truss X Constructability Concerns

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" |

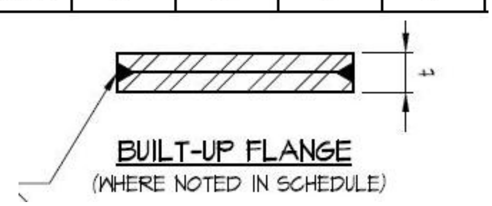



**Figure 61:** PG56-1 Precedence and Comparison (S-210)

## 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