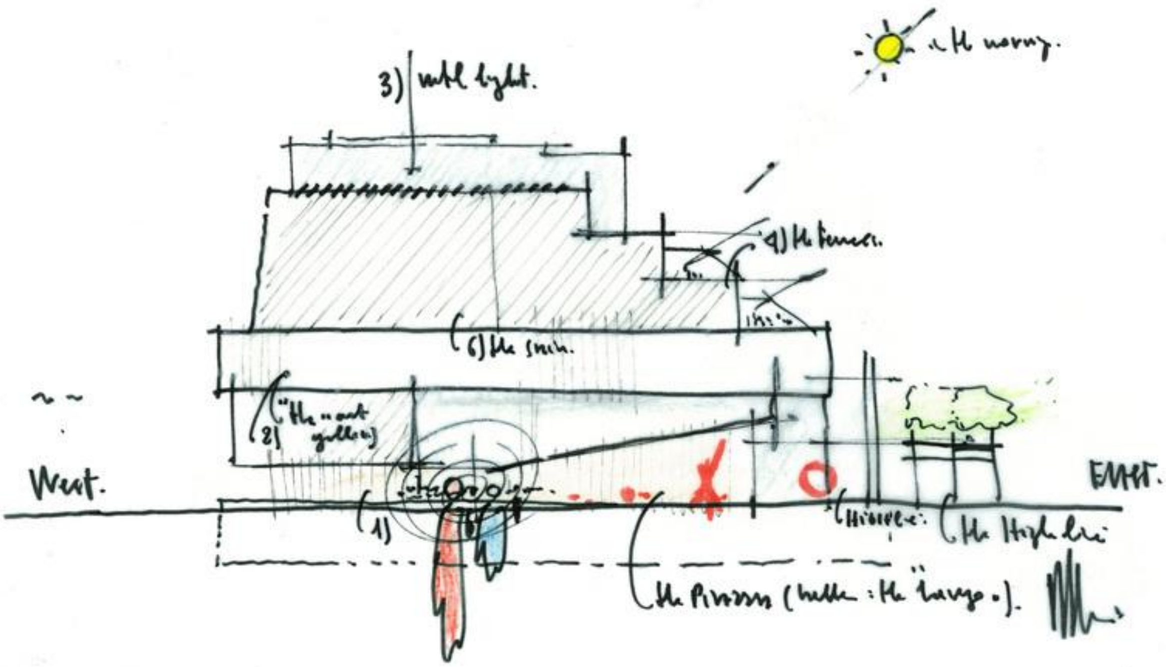
SEAN FELTON | STRUCTURAL



AMERICAN ART MUSEUM | NORTHEAST UNITED STATES

TECHNICAL REPORT 1

EXISTING CONDITIONS

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SEPTEMBER 17, 2012

EXECUTIVE SUMMARY

Technical Report 1 intends to explain the nature of the overall structural system in the American Art Museum (AAM). This understanding is gained through the use of figures and explanations concerning the foundations, floor systems, framing systems, and lateral systems of AAM. Additionally the list of applicable codes, materials, and analysis of the specified gravity and lateral loads are used to further explain the nuances of the system in its current state.

The structural team for AAM was faced with unique challenges related to the geometry of the building affecting both the gravity and lateral load paths. Furthermore, Renzo Piano's architecture typically emphasizes and exposes structural systems, meaning additional design constraints were placed on the structural team for member selection.

Specified gravity loads were verified and investigated in a member check of the decking, a typical joist, and one gravity truss, all supporting level 5. This area was chosen because of its regularized geometry in this highly irregularly shaped building. A model of the building's exterior can be seen in Figure 1. While a few discrepancies were found with regards to design and loading assumptions each checked member proved to be adequate for its loads. Typical decking and joists were verified to within 2.1% and 2 shear studs, respectively. The truss was found significantly over-designed for the gravity load case modeled, and the additional capacity is assumed to be the result of a load case that includes lateral force analysis.

Lateral load calculations were performed in accordance with the local building codes which reference ASCE 7 with minor exceptions listed in the detailed calculations. The 2005 edition of ASCE 7 was used because it was the year and code specified by the design engineers. Assumption simplifying the geometry of the building and approximating the fundamental period of vibration in the seismic analysis caused a 30% discrepancy in the base shear calculations. The carefully calculated building weight, coupled with the design base shear provided, ensured that the actual design fundamental period of vibration could be calculated for the geometrically complex building, rectifying the problems presented by the simplifying assumptions. The simplified wind design parameter assumptions were met according to ASCE 7-05, but because no final wind load design values are provided, this report is unable to offer any further understanding of the actual design wind loads.

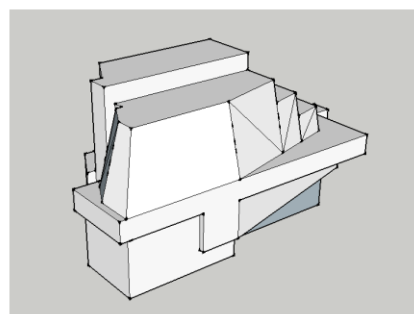

Figure 1: SketchUp Model of Geometry of Building (SW Corner)

The Appendices contain the drawings referenced in this assignment and exhaustive calculations performed for all analyses contained in Technical Report 1.

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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 steel plate, pre-cast concrete, and glazing using a standard module of 3'-4" (about 1m) (shown in Figure 3). While most of the façade components are broken at each story, the long steel plates stretch 60' on the southern wall from levels 2 to 6 and from 6 to 9.

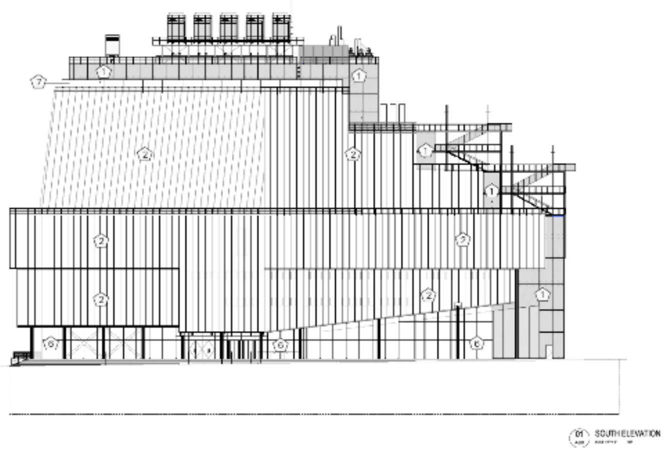


Figure 3: South Elevation showing modular façade (A-007)

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. Mechanical, storage, conservation, offices, and administration are dispersed on the north side at each level. The 220,000 square-foot AAM will stand 148ft tall and cost approximately $266 million. Construction began in May 2011 and is expected to be complete in December 2014.

STRUCTURAL SYSTEMS

OVERVIEW

AAM sits on drilled concrete caissons encased in steel with diameters of either 9.875" or 13.375" capped 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 caissons socketed into bedrock.

The engineers acted on the above suggestions and others. The caissons 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 caissons (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 F.

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.

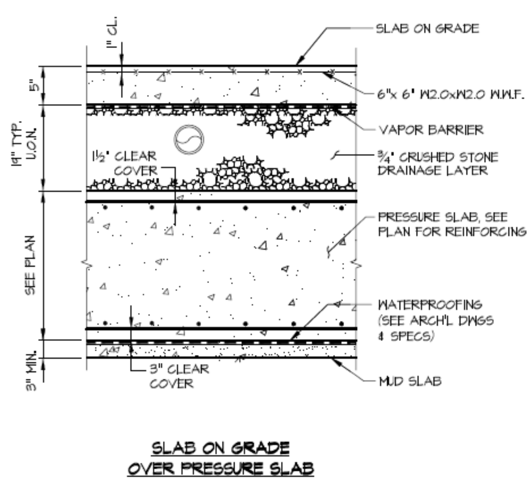


Figure 4: Pressure slab detail (S-201)

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.

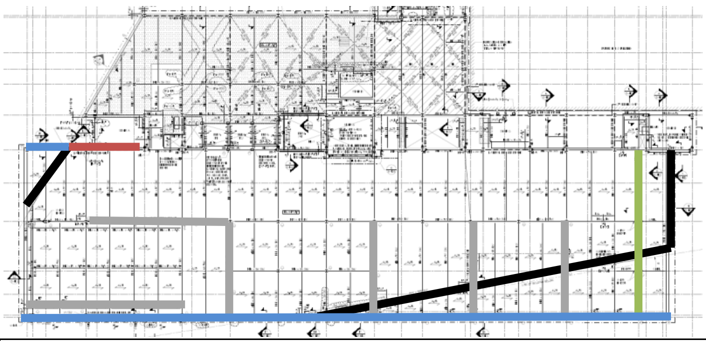


Figure 5: Level 5 framing plan showing regular layout against building footprint (S-105)

- 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.

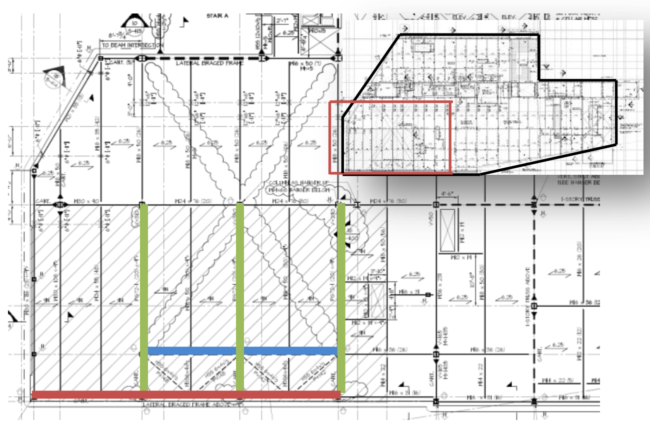


Figure 6: Level 3 framing plan showing transfer girders and lateral braced frames (S-103)

- Lateral Braced Frame (above)
- Lateral Braced Frame (below)
- Plate Girder (d=46")

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.

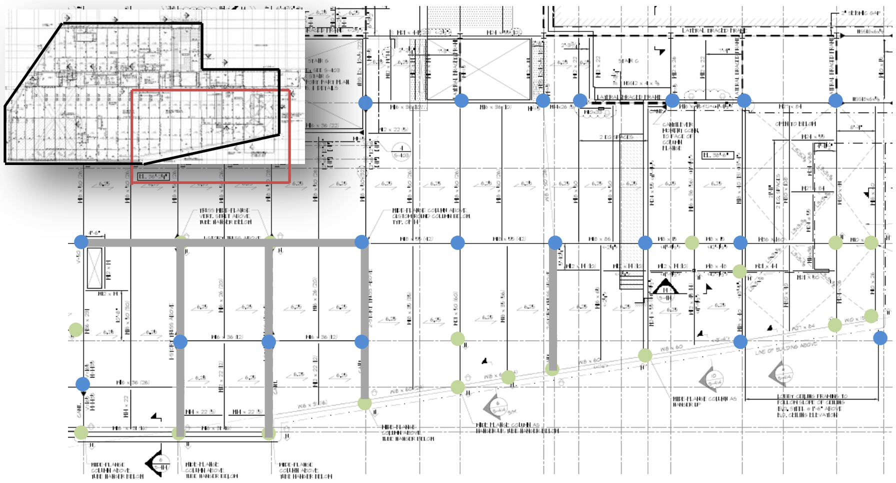


Figure 7: Level 3 framing plan showing hangers and outline of hung/cantilevered portion of building (S-103)

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.

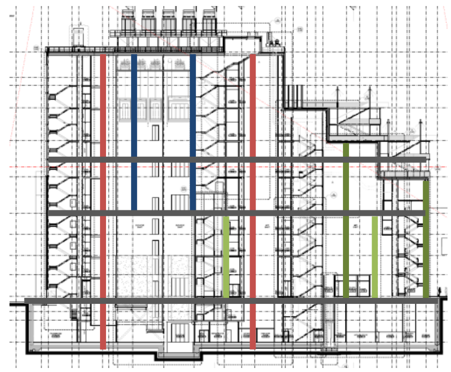


Figure 8: Section cut showing N-S braced frames at staggered heights (A-212)

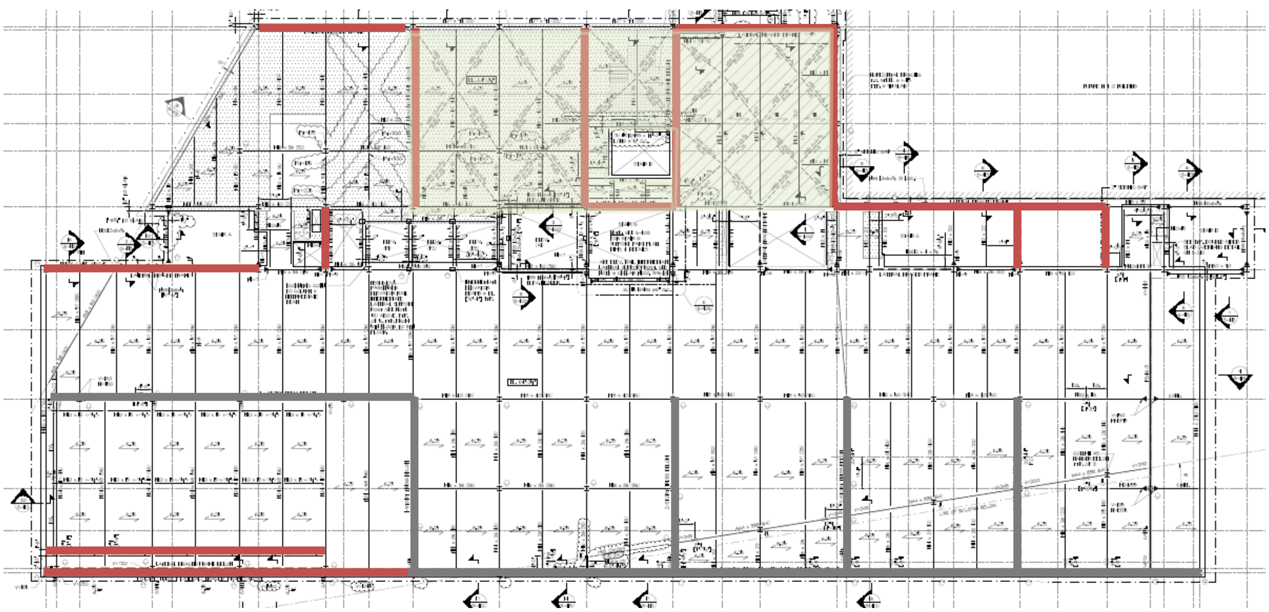


Figure 9: Level 5 Framing Plan Showing Lateral System (S-105)

- 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-200.01

Materials Specifications						
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	A501/A53	-/B	30
NW	Composite Column Alternate	8000	Channels	A36	-	36
NW	Other	5000	Angles	A36	-	36
			Plates	A36	-	36
Gr.	Use	ASTM	Connection Bolts	A325-SC	-	80
70	Reinforcement	A185	(3/4") Anchor Bolts	F1554	36	36
60	Reinforcement (epoxy coated)	A775				
70	Welded Wire Fabric	A185				

Figure 10: Summary of Structural Materials Specifications in AAM

GRAVITY LOAD AND SPOT-CHECKS

OVERVIEW

A spot-check of basic gravity members was performed as part of Technical Report 1. Selected loads and cases analyzed the frame supporting Level 5 and compared to the member selection and load resultants listed on the drawings. Detailed calculations for the gravity checks are located in Appendix A.

LOADS SUMMARY

LIVE LOADS

Typically, one would expect to see Live Loads calculated from ASCE 7 minimums (ASCE 7 Table 4-1). The structural narrative explains that much of AAM does not fit with any ASCE 7 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 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

The snow load calculations are the first of three in Technical Report 1 to be detailed using ASCE 7. 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.

Snow Load Comparison		
Design Parameters		ASCE 7
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

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.

Member Checks

As mentioned in the Gravity Load & Spot-Check overview section, elements supporting level 5 were chosen for their relatively regular geometry and layout. The area, highlighted in blue in Figure 13, is supported by the highest concentration of what this report considers a "typical" decking/slab system, joist size and number of shear studs, and an example of one of the gravity truss supports. All checks were performed using the loading assumptions listed on drawing S-120.00 for live and dead loads.

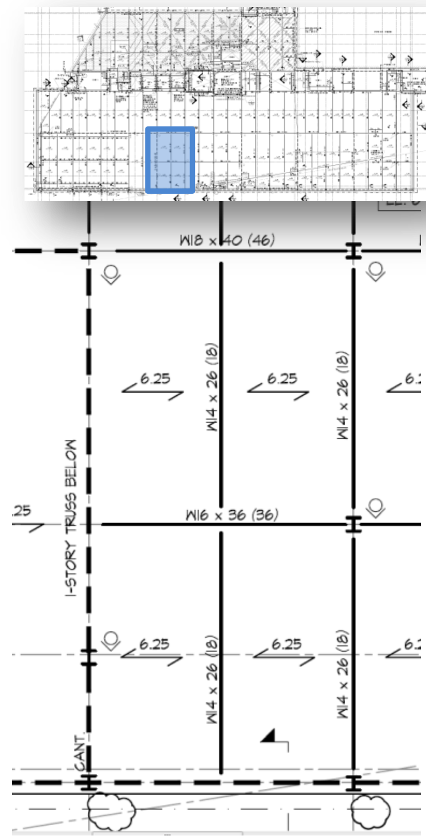


Figure 13: Level 5 Framing Plan Showing Check Area (S-105)

Decking (typical)

Specification 0530002.2.B.3 lists Vulcraft as an acceptable manufacturer of decking for AAM. The capacity information for the decking will therefore be analyzed using their catalog. Section 1.4 of the same specification notes that the decking should be in a 3-span configuration where possible and should remain unshored. The current design has the deck spanning 10'-0" across the W14x26 joists. Using the 3.25"LW topping on 3"-18 gauges composite decking should yield a capacity of 235 PSF for superimposed dead and live loads.

Required loads for this problem are found on the LL and DL schedules on S-200.01. This area in level five is in the gallery and has a LL requirement of 200PSF. The DL Schedule provides an itemized list of the dead loads for this floor type. Only the floor finish (25 PSF) and the MEP/Ceiling (15 PSF) are applicable to find the required loading, a total of 240 PSF.

It is apparent that the 235 PSF superimposed capacity is 2.1% deficient for the 240 PSF requirement specified in the drawings. This report presents three possible ways to reconcile this issue. First, the designer may have performed a more detailed analysis of the ceiling and MEP systems hung by this portion of the slab/decking. Secondly, the designer may have rounded off the capacity to meet his requirement (235 PSF = 240 PSF). Thirdly, a different manufacturer of steel decking systems may list a higher capacity for their system; one that meets the requirements set forth by the drawings. Similar to the snow load inconsistency, this floor is unlikely to ever see a full 200 PSF live load. A more detailed analysis is in Appendix A.

Joist (typical)

Moving down the load path, this report attempts to recreate the original calculations used to select the W14x26 with 18 shear studs (Figure 12). First, the dead load live load from their respective schedules must be factored (1.2D + 1.6L will control on interior floor joist). The selected joist has a span 20'-8" with spacing of 10'-0" on either side. It is simply supported.

Checks were made for positive moment capacity and number of shear studs. This report finds that the size selected is both adequate and the most economical, but specifies 16 shear studs instead of 18 as indicated on the drawings. The designer may have used extra shear studs to ensure that the Plastic Neutral Axis remains above the web portion of the steel member. A full execution of the composite section selection is in Appendix A.

TRUSS AT COLUMN LINE E

A planar STAAD analysis was used in an attempt to replicate the forces in the diagonal bracing between levels 4 and 5 shown in Figure 14. While the rest of the building and the level-1 supports are approximated as fixed, the member properties, sizes, lengths, and connections are all modeled as the drawings indicate. The model and loading are more thoroughly detailed in Appendix A.

The attempts to replicate axial forces of 1510k (braces between CL 0.9 and 2) and 2000k (braces between 2 and 4) were unsuccessful. The cantilevered braces (0.9 to 2) experienced axial loads of 1170 k (compression) and 470k (tension) while the diagonal member (2 to 4) experienced 315 k (tension). This report suggests that the remaining capacity would be used in a lateral load analysis.

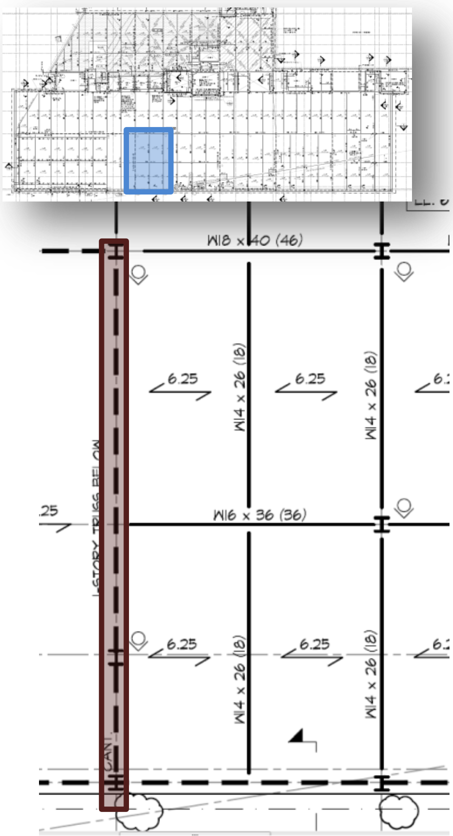
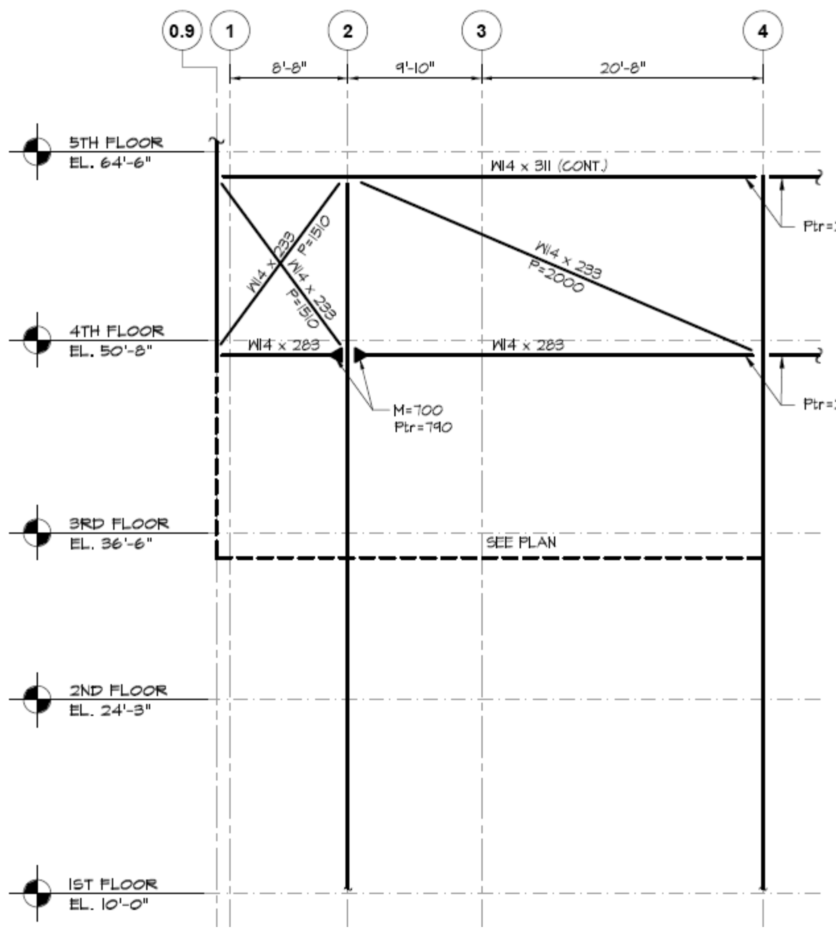


Figure 14: Gravity Truss at Column Line E (S-122)



LATERAL LOADS

OVERVIEW

In order to gain a better understanding of the lateral systems, the wind and seismic lateral loads were calculated for Technical Report 1. Due to the high complexity of AAM's structure, a series of simplifying assumptions had to be used in order to discover the minimum design loads in ASCE 7. The most significant of which was the alteration of the footprint and profile of the building. Future technical reports will use an accurate structural model, but this analysis applies lateral loads to a building of base dimensions 126'x285' with a roof height of 160.' Figure 1 in the Executive Summary shows the true geometry of AAM.

SEISMIC LOADS

The seismic load analysis in this report attempts to replicate the base shear of 946k given in the design parameters on drawing S-200.01 using the Equivalent Lateral Force Procedure detailed in ASCE 7. After an itemized dead load assignment, an estimate of the building's fundamental period of vibration, and Cs factor calculations, the rectangular building's base shear was calculated to be 695k with an overturning moment of 52400 k-ft with the distribution shown in Figure 15.

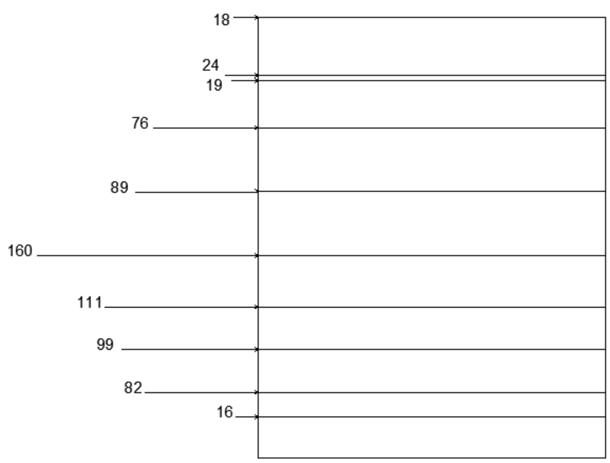


Figure 15: ASCE 7 Estimated Load Distribution for Rectangular Building

V_b = 695 k
M_{ov} = 52400 k-ft

Several values in the calculations for the Cs factor conflict with the provided design parameters for seismic. The seismic design parameters provided for S_{DS} and S_{D1} are not suggested from the geotechnical report, and do not match the ASCE 7 maps. There are no local building code exceptions for seismic design parameters. A request has been made to the design professional to discover the origin of these values.

Additionally, because the fundamental period of vibration, T, was estimated using ASCE 7 as 1.53s in respect to a rectangular building, it is safe to assume that this value does not accurately reflect the fundamental period of the actual building.

Because values I and R were verified, and assuming the weight of the building was calculated correctly, it is possible to use the design parameter base shear to calculate what Cs should be and what the design fundamental period of the building is. Figure 16 shows that procedure.

Calculate Base Shear, Overturning Moment						
ASCE 7-05 Estimation					Design	
			Cs	0.038126	Cs,des	0.052
Level	Ht (ft)	W (k)	V	Mi	Vdes	Mi,des
9RN	160	483	18	2946	25	4012
9RS	142	628	24	3397	33	4626
9	140	500	19	2669	26	3635
8	124	2002	76	9463	104	12887
7	102	2342	89	9106	122	12400
6	78	4188	160	12455	217	16962
5	55	2915	111	6112	151	8323
4	41	2589	99	4048	134	5512
3	24	2155	82	1972	112	2685
2	15	419	16	240	22	326
		18220	695	52409	946	71370
		W(t)	Vb	Mov (kft)	Vb,des	Mov, des

Work Back: Design Cs, T			
Given		Calcs	
V	946	Cs =	Vspec/W
Sds	0.65	=	0.052
Sd1	0.13	* assume T<TL	
I	1.25	Cs,u = Sd1/(T(R/I))	
R	3	T =	(Sd1 I)/(Cs R)
W	18220	=	1.043 s

Figure 16: Summary of the Design Seismic Values and Estimated Seismic Values

As summarized in Figure 16 above, the actual design base shear of 946 K gives an overturning moment of 71400 k-ft with a fundamental period of vibration of 1.043s. These are the values that Technical Report 3 will attempt to reproduce. The complete calculations for the dead loads and seismic loads are in Appendix B and Appendix C respectively.

WIND LOADS

In place of wind tunnel testing required by ASCE 7 for a building of this complex geometry, the rectangular building allows the use of Method 2 for the Main Wind Force Resisting System. Because of the complexity of the actual roof geometry, the calculations in ASCE 7 pertaining to the roof pressure were neglected for the purposes of Technical Report 1. The resultant pressures and loads and their calculations are shown in Figures 17 and 18 below.

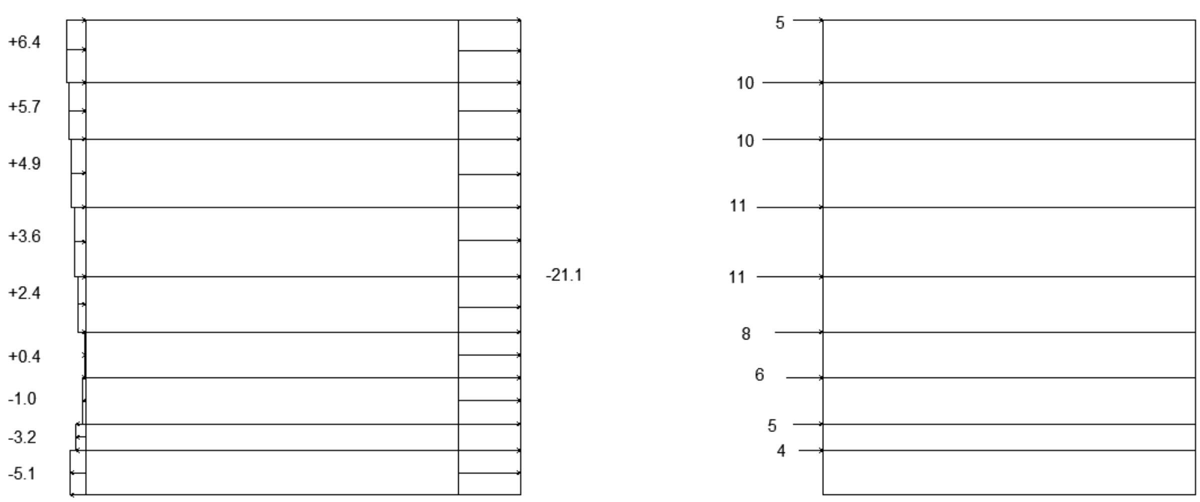


Figure 17: Pressures and Equivalent Lateral Forces

				Wall Pressures							
					WW	LW		vrt dist		Load	
Level	ht	Kz	qz	Cp	0.8	-0.55	Eq. Lat Pressure	per level	PLF hrz	(20ft bay) K	Mi kft
9R	160	1.39	42.09		6.39	-21.07	27.46	10	275	5	879
9	140	1.36	41.18		5.75	-21.07	26.82	18	483	10	1352
8	124	1.32	39.97		4.90	-21.07	25.97	19	493	10	1224
7	102	1.26	38.15		3.63	-21.07	24.70	23	568	11	1159
6	78	1.2	36.33		2.35	-21.07	23.42	23.5	550	11	859
5	55	1.11	33.61		0.44	-21.07	21.51	18.5	398	8	438
4	41	1.04	31.49		-1.05	-21.07	20.02	15.5	310	6	254
3	24	0.94	28.46		-3.17	-21.07	17.90	13	233	5	112
2	15	0.85	25.74		-5.09	-21.07	15.98	12	192	4	58
										Mov =	6333

Figure 18: Equivalent Lateral Pressures, Forces, Overturning Moment Calcs

While the provided design parameters such as the exposure category, importance factor, and basic wind speed match this analysis, there are no final wind design values provided by the structural professional. A comparison between the above values would be impossible because of the differences in geometry and procedure, anyway. Technical Report 1 provides a simplified analysis of the wind design loads for the project, and future technical reports will make adjustments for accuracy. The complete wind loads analysis is in Appendix D.

CONCLUSION

Technical Report 1 analyzed the existing structural conditions of the American Art Museum. The foundations, floor systems, frame systems, and lateral systems were summarized with figures and descriptions to adequately present the current structural design.

Additionally the live, dead, and snow loads were described and calculated to perform gravity member spot-checks of typical members. These values were derived from ASCE 7-05, the latest edition of that code at the time of development, in order to replicate the design provided on the drawings and specifications. The complexity of the building required this technical report to verify the validity of the designers live and dead load assumptions as opposed to compare unique live and dead load selections for given spaces. These provided loads allowed for a transparent and congruent check of the decking/slab selection, typical joist size and shear pin number, and an analysis of one of the gravity trusses. Though a few discrepancies in member selection occurred, each difference can easily be reconciled by obvious simplifying assumptions that do not affect the structural integrity of the building.

An ASCE 7-05 analysis of the wind and seismic loads was also performed, but were limited in relevance due to the complexity of the building's geometry. A comparison between this report's estimate of the seismic design parameters through ASCE 7-05 and the specified design parameters on the drawings provided insight into the true fundamental period of vibration of AAM. The wind load calculations are impossible to comprehensively explain without more information provided by the structural engineers.

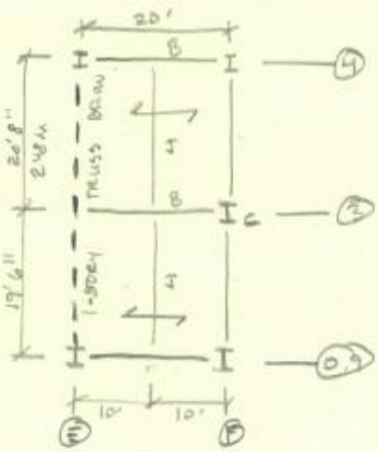
APPENDICES

APPENDIX A: GRAVITY SYSTEM CHECK CALCULATIONS

1/11

GRAVITY SPOT-CHECK

TYP DECK, JOIST, COLUMN, TRUSS SYSTEM AT LEVEL 5 FLOOR FRAMING



A: DECKING

SUPERIMPOSED DL
MECH / CEILING : 15 PSF
FLOOR FINISH : 25 PSF
SUPERIMPOSED LL : 200 PSF
240 PSF

* LOAD ASSUMPTIONS TAKEN FROM PAGE S-200.01 LL PLANS, DL SCHEDULE

* DECKING CALLOUT ON P. S-105 NOTES 3 1/4" CONCRETE (L.W.) ON 3" - 18 GA METAL DECK

* USING 3-SPAN WHERE POSSIBLE, UNSHORED (SPEC 053000 1.4 B.D)

* VULCRAFT 1 OF 4 POSSIBLE MFG (SPEC 053000 2.2.B.3)

DECK SPANS 10'-0"
– USE VULCRAFT DECK CATALOG

L.W. COMPOSITE 3VLI :
t = 3.25
3VLI 18:

UNSHORED 1 SPN	2 SPN	3 SPN	SPAN 10'-0"
12'-9"	15'-0"	15'-0"	235 PSF
OK	OK	OK	

235 PSF ≯ 240 PSF NG

$\frac{5 \text{ PSF}}{240 \text{ PSF}}$ RESULTS IN A 2.1% DEFICIENCY

1.) POSSIBLY ROUNDING (i.e. 235 → 240 = 240 OK)
OR
2.) DIFFERENT MFG COMPANY MAY HAVE HIGHER CAPACITY (242 ≥ 240 ∴ OK)

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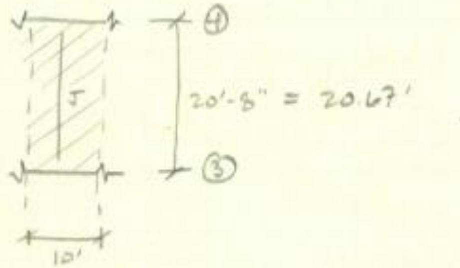
B: JOIST 3-4 : 20'-8"

DL : FLOOR TYPE 1 = 109 PSF (S-200.01)
DECK/SLAB = 49 PSF
FLOOR FINISH = 25 PSF
CEILING/MECH = 15 PSF
STL SELF WT = 20 PSF

LL = 200 PSF



FIND beff

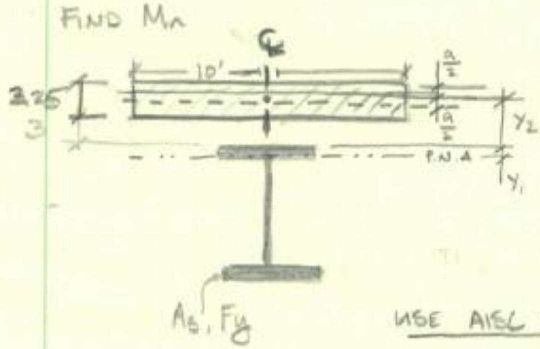
$$b_{eff} = \max \begin{cases} S/2 = 10' \\ \ell/8 = 2.6' \end{cases}$$

FIND Mu

$$w_J = 10\left[(1.2 \cdot 109) + (1.6 \cdot 200)\right] / 1000 = \underline{4.5 \text{ klf}}$$

$$M_u = \frac{w_u \ell^2}{8} = \frac{4.5 \cdot (20.67^2)}{8} = \underline{240 \text{ k}\cdot\text{ft}}$$

FIND Mn



f'c = 4000 psi (S-200.01)
Fy = 50 ksi (S-200.01)

Assume continuous bracing
Assume a ≈ 1" ∴ $Y_2 = 6.25 - 0.5$ in
$Y_2 = 5.75$ in, use $\underline{Y_2 = 5.5 \text{ in}}$

USE AISC XIV TABLE 3-19 TO SELECT:

TRY W14x SECTION FOR CONSISTENCY ($Y_2 = 5.5$; Y_1 w/in Flange)
ASSUME 3/4" SHEAR STUDS: 1/RIB, WEAK LOCATION PER TABLE 3-21.

W14x22 : $\phi M_n = 248$ k·ft @ $\Sigma Q_n = 199$ k
$n = \frac{199\text{k}}{17.2\text{k}}$ (3/4" wk, ⊥ DECK, LWf'c=4ksi) = 12 → 24 studs XNG
(> 1 stud per rib ∴ NG)

W14x26 : $\phi M_n = 250$ k·ft @ $\Sigma Q_n = 135$ k
$n = \frac{135}{17.2} = 8$ → 16 studs TW = 20.67×26 + 160 = 697 lb

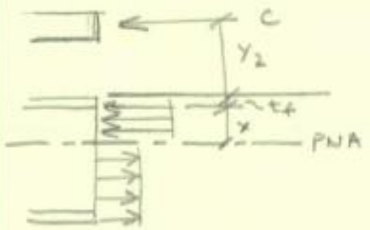
W14x30 : $\phi M_n = 263$ k·ft @ $\Sigma Q_n = 111$ k
$n = \frac{111}{17.2} = 7$ → 14 studs TW = 20.67×30 + 140 = 760 lb

3/11

USE W14 x 26 w/ 16 SMDS

CHECK $\phi M_n \geq M_u$

$b_{eff} = 10'$ (per gravity calcs p.2)

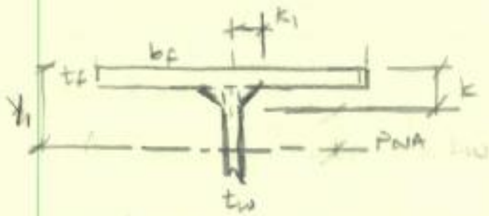


$\Sigma Q_n = 17.2 \cdot 8 = 137.6\ k$

$\Sigma Q_n = 0.85 f'_c\, b a$

$a = \dfrac{\Sigma Q_n}{0.85 f'_c\, b} = \dfrac{137.6}{0.85(4)(10' \times 12)}$

$a = 0.337\ in$



$Y_1 = ?$ $\qquad k_1 = 0.75\ in$

$b_f = 5.03\ in$ $\qquad k = 0.82\ in$

$t_f = 0.42\ in$

$t_w = 0.255\ in$

FIND AREA OF STEEL IN COMPRESSION; A_{sc} ASSUME WEB

$$A_{sc} = b_f t_f + (Y_1 - t_f) t_w + (k - t_f) k_1$$
$$= (5.03)(0.42) + [(Y_1 - 0.42) 0.255] + [(0.82 - 0.42) 0.75]$$
$$= 2.113 + 0.255 Y_1 - 0.107 + 0.3$$
$$= 2.306 + 0.255 Y_1$$

FIND Y_1: $T = C$

$$A_s F_y = 0.85 f'_c\, b_{eff}\, a + 2 A_{sc} F_y$$

$$\frac{A_s F_y - 0.85 f'_c\, b_{eff}\, a}{2 F_y} = A_{sc}$$

$$\underbrace{\frac{7.69}{2}}_{=3.845} - \frac{\overbrace{0.85(4)(120)(0.337)}^{=137.5}}{\underbrace{2(50)}_{=100}} = 2.306 + 0.255 Y_1$$

$$\frac{2.470 - 2.306}{0.255} = Y_1$$

$Y_1 = 0.643\ in \gg 0.42\ in \therefore$ PNA IN WEB

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FIND A_{sc} CENTROID

$$\bar{y} = \frac{\Sigma A_i y_i}{\Sigma A_i} = \frac{2.113\left(\frac{0.42}{2}\right) + (0.255 \cdot 0.643)\left(\frac{0.643-0.42}{2}\right) + 0.3\frac{(0.82-0.42)}{2}}{[2.306 + 0.255(0.443) = 2.47]}$$

$\bar{y} = 0.211$ in FROM TOP OF STEEL

FIND M_n FROM CONCRETE COMPRESSION LOCATION:

$$y_2 = 6.25 - \frac{0.337}{2} = 6.082$$

$$A_s f_y\left(y_2 + \frac{d}{2}\right) - 2 A_{sc} F_y (y_2 + \bar{y}) = M_n$$

$$7.69(50)\left(6.082 + \frac{13.9}{2}\right) - 2(2.47)(50)(6.082 + 0.211) = M_n$$

$$M_n = 3456 \text{ k-in} = 288 \text{ k-ft}$$

$$\phi M_n \geq M_u$$

$$0.9 \cdot 288 \geq 240$$

$\boxed{259 \text{ k-ft} \geq 240 \text{ k-ft} \therefore \text{OK STRENGTH}}$

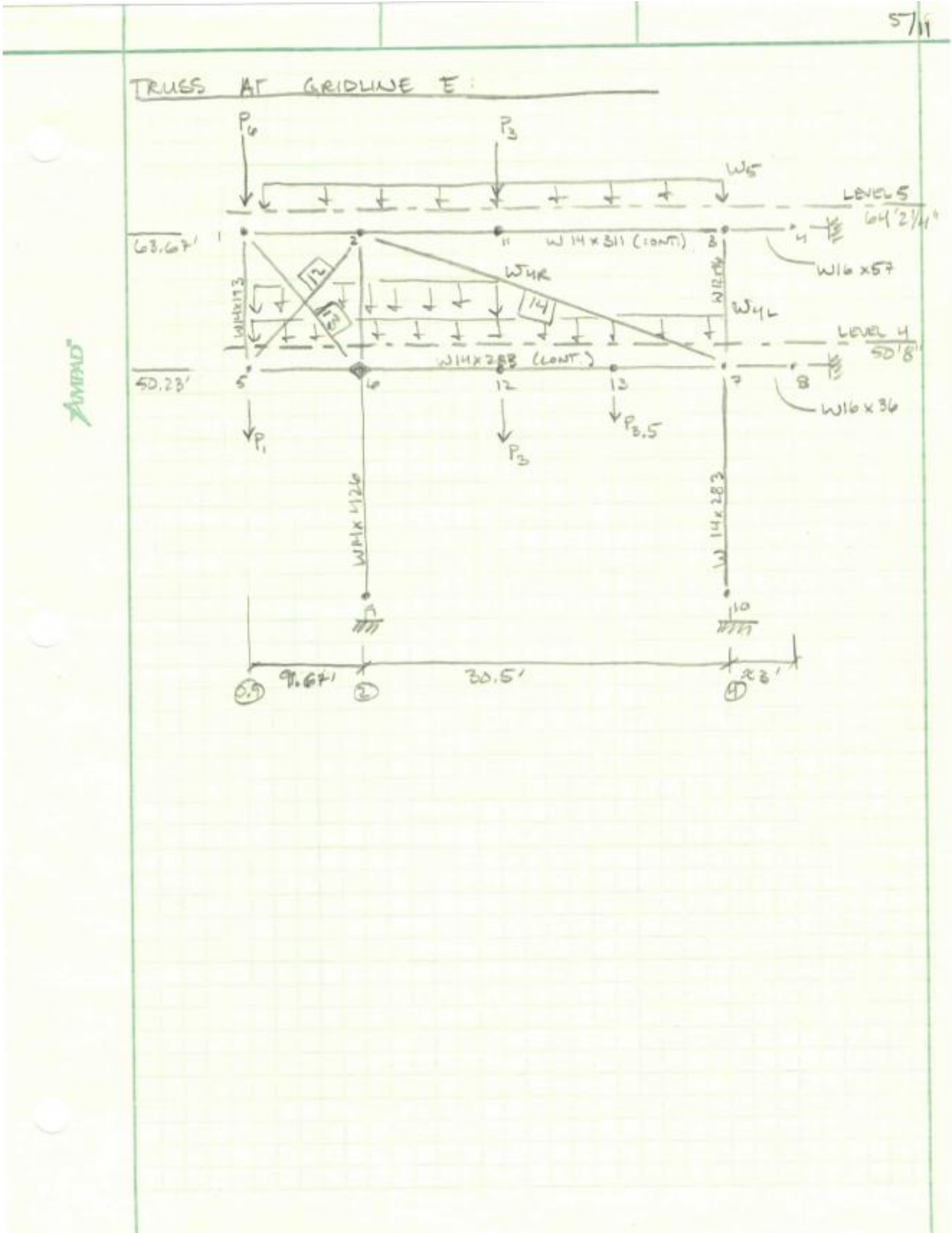
* NOTE: DRAWINGS SELECT W14x26 WITH 18 SHEAR STUDS, WHICH IS ONE (1) ADDITIONAL ON EACH END.

$$\Sigma Q_n (9) = 17.2 \cdot 9 = 155 \text{ k}$$

FIND Y_1 WHEN $\Sigma Q_n = 155$ k. $\quad a = \frac{155}{0.85 \cdot 4 \cdot 120} = 0.38$ in

$$\frac{3.815 - 1.5504 - 2.306}{0.255} = Y_1$$

$Y_1 = -0.0447$; PNA IS NOT IN WEB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TRUSS AT GRIDLINE E:
P6
P3
W5
LEVEL 5
64'2 1/4"
63.67'
W 14 x 311 (CONT.)
W16 x 57
W4R
W4L
W14x193
LEVEL 4
50'8"
50.23'
W14x283 (CONT.)
W16 x 36
P1
P3
P3.5
W14x426
W 14x283
9.67'
30.5'
≈3'

6/11

THIS TRUSS ANALYSIS IS DESIGNED TO CHECK THE CAPACITY OF THE DIAGONAL BRACING ON MEMBERS 1-6, 2-7, AND 5-2

DRAWING S-122 INDICATES FOLLOWING LOADS CARRIED IN MEMBERS:

1-6 : W14x233 : P = 1510 k
5-2 : W14x233 : P = 1510 k
2-7 : W14x233 : P = 2000 k

LOADING: LOADING WILL BE GIVEN IN ACCORDANCE WITH THE INFORMATION PROVIDED ON THE DRAWINGS

NODE 1 : $P_6 = 1475\text{k}$ (S-120.01)

LEVEL 5 LOADS:

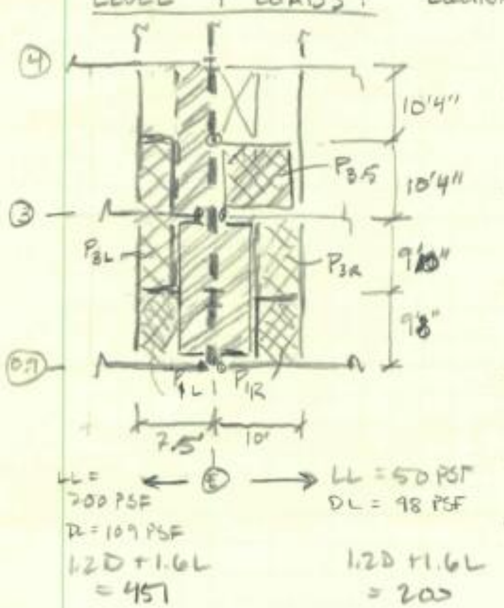
DISTRIBUTED LOAD FROM (L); PT LOAD AT NODE 2 FROM (R)

$w_{T5} = \frac{1}{2} w_J$ (FROM GRAVITY P. 2) $= \frac{4.5}{2} = 2.25$ klf

SINCE w_J INCLUDES SELF-WT, P_B AT NODE 2 IS EQUIVALENT TO $\frac{1}{2}$ OF 2x THE TOTAL JOIST LOAD

$P_B = \frac{1}{2} \cdot 2 w_J \ell_J = 4.5 \cdot 20.67' = 93\text{k}$

LEVEL 4 LOADS: EQUIVALENT TRIB AREA



TRIB AREA AS PT. LOAD (P_x)

TRIB AREA AS DISTR. LOAD (w_x)

DEAD & LIVE LOADS FROM S-200.01

$P_1 = P_{1L} + P_{1R} = (5 + 3.75')9.75' \cdot 451$

$\underline{P_1 = 38.5\text{ k}}$

$P_3 = [3.75(9.75 + 10.33)]451 + [5 \cdot 9.75]200$

$\underline{P_3 = 43.7\text{ k}}$

$P_{3.5} = 200 \cdot 10.33 \cdot 10 = \underline{20.7\text{ k}}$

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LEVEL 4 LOADS (CONT'D)

$w_{4L} = 457 \cdot 3.75 = 1.7$ klf

$w_{4R} = 200 \cdot 5 = 1$ klf

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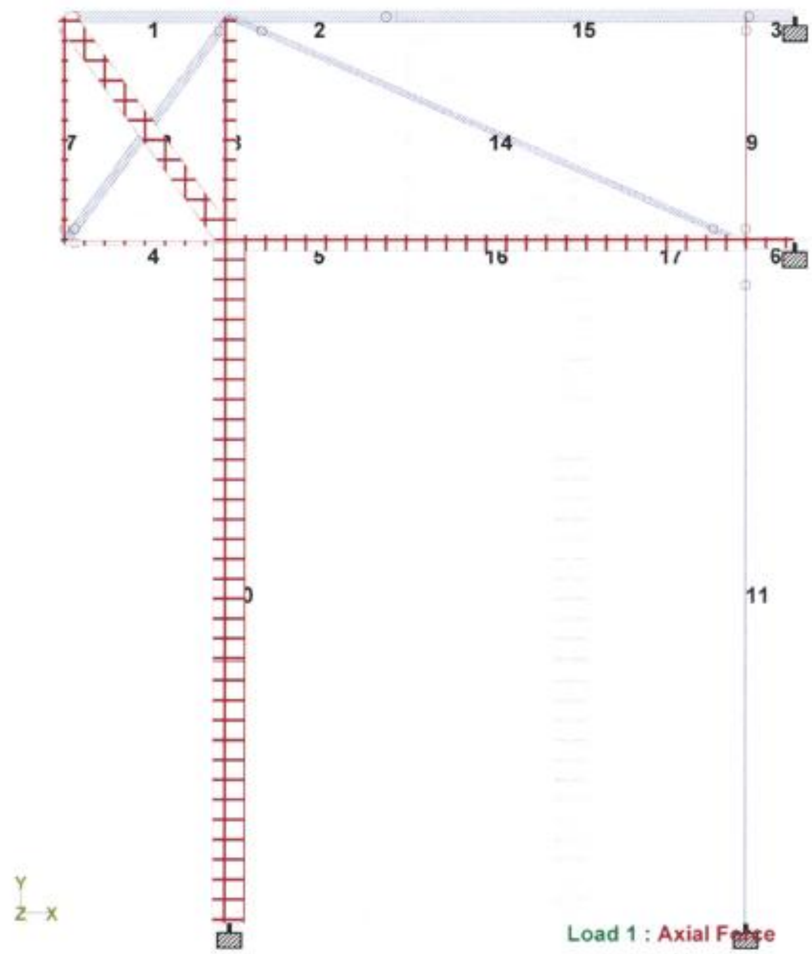
By	Date 16-Sep-12	Chd
File felton_thesis_truss E.std	Date/Time 17-Sep-2012 05:09	

Job Title

Client



Whole Structure Fx 2000kip:1ft 1 1.2D+1.6L

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Job Title	Part		
	Ref		
	By	Date 16-Sep-12	Chd
Client	File felton_thesis_truss E.std	Date/Time 17-Sep-2012 05:09	

Beam Force Detail Cont...

			Axial	Shear		Torsion	Bending	
Beam	L/C	d (ft)	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip·in)	My (kip·in)	Mz (kip·in)
9	1:1.2D+1.6L	0.000	32.868	0.000	0.000	0.000	0.000	0.000
		1.344	32.868	0.000	0.000	0.000	0.000	-0.000
		2.688	32.868	0.000	0.000	0.000	0.000	-0.000
		4.032	32.868	0.000	0.000	0.000	0.000	-0.000
		5.376	32.868	0.000	0.000	0.000	0.000	-0.000
		6.720	32.868	0.000	0.000	0.000	0.000	-0.000
		8.064	32.868	0.000	0.000	0.000	0.000	-0.000
		9.408	32.868	0.000	0.000	0.000	0.000	-0.000
		10.752	32.868	0.000	0.000	0.000	0.000	-0.000
		12.096	32.868	0.000	0.000	0.000	0.000	-0.000
		13.440	32.868	0.000	-0.000	-0.000	-0.000	-0.000
10	1:1.2D+1.6L	0.000	1.92E+3	1.115	0.000	0.000	0.000	538.107
		4.023	1.92E+3	1.115	0.000	0.000	0.000	484.296
		8.046	1.92E+3	1.115	0.000	0.000	0.000	430.485
		12.069	1.92E+3	1.115	0.000	0.000	0.000	376.675
		16.092	1.92E+3	1.115	0.000	0.000	0.000	322.864
		20.115	1.92E+3	1.115	0.000	0.000	0.000	269.053
		24.138	1.92E+3	1.115	0.000	0.000	0.000	215.243
		28.161	1.92E+3	1.115	0.000	0.000	0.000	161.432
		32.184	1.92E+3	1.115	0.000	0.000	0.000	107.621
		36.207	1.92E+3	1.115	0.000	0.000	0.000	53.811
		40.230	1.92E+3	1.115	-0.000	-0.000	-0.000	-0.000
11	1:1.2D+1.6L	0.000	-61.150	0.219	0.000	0.000	0.000	105.649
		4.023	-61.150	0.219	0.000	0.000	0.000	95.084
		8.046	-61.150	0.219	0.000	0.000	0.000	84.519
		12.069	-61.150	0.219	0.000	0.000	0.000	73.954
		16.092	-61.150	0.219	0.000	0.000	0.000	63.389
		20.115	-61.150	0.219	0.000	0.000	0.000	52.824
		24.138	-61.150	0.219	0.000	0.000	0.000	42.260
		28.161	-61.150	0.219	0.000	0.000	0.000	31.695
		32.184	-61.150	0.219	0.000	0.000	0.000	21.130
		36.207	-61.150	0.219	0.000	0.000	0.000	10.565
		40.230	-61.150	0.219	-0.000	-0.000	-0.000	-0.000
12	1:1.2D+1.6L	0.000	1.17E+3	-0.000	0.000	0.000	0.000	0.000
		1.656	1.17E+3	-0.000	0.000	0.000	0.000	0.000
		3.311	1.17E+3	-0.000	0.000	0.000	0.000	0.000
		4.967	1.17E+3	-0.000	0.000	0.000	0.000	0.000
		6.623	1.17E+3	-0.000	0.000	0.000	0.000	0.000
		8.279	1.17E+3	-0.000	0.000	0.000	0.000	0.000
		9.934	1.17E+3	-0.000	0.000	0.000	0.000	0.000
		11.590	1.17E+3	-0.000	0.000	0.000	0.000	0.000
		13.246	1.17E+3	-0.000	0.000	0.000	0.000	0.000
		14.902	1.17E+3	-0.000	0.000	0.000	0.000	0.000
		16.557	1.17E+3	-0.000	-0.000	-0.000	-0.000	-0.000
13	1:1.2D+1.6L	0.000	-469.992	-0.000	0.000	0.000	0.000	0.000

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Beam Force Detail Cont...

			Axial	Shear		Torsion	Bending	
Beam	L/C	d (ft)	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip`in)	My (kip`in)	Mz (kip`in)
		1.656	-469.992	-0.000	0.000	0.000	0.000	0.000
		3.311	-469.992	-0.000	0.000	0.000	0.000	0.000
		4.967	-469.992	-0.000	0.000	0.000	0.000	0.000
		6.622	-469.992	-0.000	0.000	0.000	0.000	0.000
		8.278	-469.992	-0.000	0.000	0.000	0.000	0.000
		9.933	-469.992	-0.000	0.000	0.000	0.000	0.000
		11.589	-469.992	-0.000	0.000	0.000	0.000	0.000
		13.244	-469.992	-0.000	0.000	0.000	0.000	0.000
		14.900	-469.992	-0.000	0.000	0.000	0.000	0.000
		16.556	-469.992	-0.000	-0.000	-0.000	-0.000	-0.000
14	1:1.2D+1.6L	0.000	-314.462	-0.000	0.000	0.000	0.000	0.000
		3.333	-314.462	-0.000	0.000	0.000	0.000	0.000
		6.666	-314.462	-0.000	0.000	0.000	0.000	0.000
		9.999	-314.462	-0.000	0.000	0.000	0.000	0.000
		13.332	-314.462	-0.000	0.000	0.000	0.000	0.000
		16.665	-314.462	-0.000	0.000	0.000	0.000	0.000
		19.998	-314.462	-0.000	0.000	0.000	0.000	0.000
		23.331	-314.462	-0.000	0.000	0.000	0.000	0.000
		26.664	-314.462	-0.000	0.000	0.000	0.000	0.000
		29.997	-314.462	-0.000	0.000	0.000	0.000	0.000
		33.330	-314.462	-0.000	-0.000	-0.000	-0.000	-0.000
15	1:1.2D+1.6L	0.000	-668.495	23.254	0.000	0.000	0.000	0.000
		2.067	-668.495	18.603	0.000	0.000	0.000	-512.699
		4.134	-668.495	13.952	0.000	0.000	0.000	-913.245
		6.201	-668.495	9.301	0.000	0.000	0.000	-1.2E+3
		8.268	-668.495	4.651	0.000	0.000	0.000	-1.38E+3
		10.335	-668.495	0.000	0.000	0.000	0.000	-1.44E+3
		12.402	-668.495	-4.651	0.000	0.000	0.000	-1.38E+3
		14.469	-668.495	-9.302	0.000	0.000	0.000	-1.2E+3
		16.536	-668.495	-13.952	0.000	0.000	0.000	-913.244
		18.603	-668.495	-18.603	0.000	0.000	0.000	-512.699
		20.670	-668.495	-23.254	-0.000	-0.000	-0.000	-0.000
16	1:1.2D+1.6L	0.000	955.143	20.690	0.000	0.000	0.000	0.000
		1.033	955.143	19.657	0.000	0.000	0.000	-249.359
		2.066	955.143	18.624	0.000	0.000	0.000	-486.269
		3.099	955.143	17.591	0.000	0.000	0.000	-710.730
		4.132	955.143	16.558	0.000	0.000	0.000	-922.741
		5.165	955.143	15.525	0.000	0.000	0.000	-1.12E+3
		6.198	955.143	14.492	0.000	0.000	0.000	-1.31E+3
		7.231	955.143	13.459	0.000	0.000	0.000	-1.48E+3
		8.264	955.143	12.426	0.000	0.000	0.000	-1.64E+3
		9.297	955.143	11.393	0.000	0.000	0.000	-1.79E+3
		10.330	955.143	10.360	-0.000	-0.000	-0.000	-1.92E+3
17	1:1.2D+1.6L	0.000	955.143	-10.340	0.000	0.000	0.000	-1.92E+3
		1.034	955.143	-11.374	0.000	0.000	0.000	-1.79E+3

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C:\TEMP\felton_thesis_truss E.std 09/17/12 05:09:06
STAAD PLANE
START JOB INFORMATION
ENGINEER DATE 16-Sep-12
END JOB INFORMATION
INPUT WIDTH 79
UNIT FEET KIP
JOINT COORDINATES
1 0 63.67 0; 2 9.67 63.67 0; 3 40.17 63.67 0; 4 43 63.67 0; 5 0 50.232 0;
6 9.67 50.23 0; 7 40.17 50.23 0; 8 43 50.23 0; 9 9.67 10 0; 10 40.17 10 0;
11 19.5 63.67 0; 12 19.5 50.23 0; 13 29.83 50.23 0;
MEMBER INCIDENCES
1 1 2; 2 2 11; 3 3 4; 4 5 6; 5 6 12; 6 7 8; 7 5 1; 8 6 2; 9 7 3; 10 9 6;
11 10 7; 12 1 6; 13 5 2; 14 2 7; 15 11 3; 16 12 13; 17 13 7;
DEFINE MATERIAL START
ISOTROPIC STEEL
E 4.176e+006
POISSON 0.3
DENSITY 0.489024
ALPHA 6.5e-006
DAMP 0.03
TYPE STEEL
STRENGTH FY 5184 FU 8352 RY 1.5 RT 1.2
G 1.60615e+006
ISOTROPIC STEEL_50
E 29000
TYPE S
STRENGTH FY 50 FU 82 RY 1 RT 1
POISSON 0.3
DENSITY 0.49
DAMP 0.03
G 1.6e+006
END DEFINE MATERIAL
MEMBER PROPERTY AMERICAN
9 TABLE ST W12X96
7 TABLE ST W14X193
12 TO 14 TABLE ST W14X233
8 TABLE ST W14X257
11 TABLE ST W14X283
10 TABLE ST W14X426
3 TABLE ST W16X57
6 TABLE ST W16X36
1 2 15 TABLE ST W14X311
4 5 16 17 TABLE ST W14X283
CONSTANTS
BETA 0 MEMB 1
MATERIAL STEEL_50 ALL
SUPPORTS
4 8 TO 10 FIXED
MEMBER RELEASE
1 3 4 6 TO 9 12 TO 14 START MZ
2 5 7 TO 14 END MZ
LOAD 1 LOADTYPE Gravity  TITLE 1.2D+1.6L
JOINT LOAD
1 FY -1475
5 FY -38.5
12 FY -43.7
13 FY -20.7
11 FY -93
MEMBER LOAD
4 5 16 17 UNI GY -1
1 2 15 UNI GY -2.25
4 5 UNI GY -1.7
PERFORM ANALYSIS
FINISH

Page: 1
```

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APPENDIX B: TOTAL 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		

ASCE 7-05
SEISMIC LOADS 1/3

SEISMIC DESIGN CRITERIA 11.4.1

S_S = 0.35 (FIG 22-1)
S_1 = 0.06 (FIG 22-2)

THE GEOTECHNICAL REPORT LISTS SITE CLASSIFICATIONS D, E ARE BOTH APPLICABLE (11.4.2)

SITE CLASS:	D	E	
F_a	1.5	2.1	(TABLE 11.4-1)
F_v	2.4	3.5	(TABLE 11.4-2)

$S_{MS} = F_a S_S$
$S_{M1} = F_v S_1$

	D	E	
S_{MS}	0.525	0.735	(EQN 11.4-1)
S_{M1}	0.144	0.21	(EQN 11.4-2)

$S_{DS} = \frac{2}{3} S_{MS}$
$S_{D1} = \frac{2}{3} S_{D1}$

	D	E	
* S_{DS}	0.35	0.49	(EQN 11.4-3)
* S_{D1}	0.096	0.14	(EQN 11.4-4)

SEISMIC DESIGN CATEGORY "D" (TABLE 11.6-2)
OCCUPANCY III ; $0.2 \le S_{D1}$ IN BOTH CASES

NOTE: BUILDING IS HIGHLY IRREGULAR IN GEOMETRY & TORSIONAL RESISTANCE. ASSUME SIMPLIFIED GEOMETRY FOR EQ. LAT. FORCE PROCEDURE.

$I = 1.25$ (11.5.1)

$R = 3$ (TABLE 12.2-1) LATERAL SYSTEM IS NOT SPECIFICALLY DETAILED FOR SEISMIC LOADING.

$$0.01 \le C_s \le \min \begin{cases} \dfrac{S_{D1}}{T(R/I)} & T \le T_L \\[2ex] \dfrac{S_{D1} T_L}{T^2 (R/I)} & T > T_L \end{cases}$$

$T_L = 6$ (FIG 22-15)

FIND T : $\le C_u T_a$

$T_a = C_t h_n^x$

$C_t = 0.02$ (PER TABLE 12.8-2)
$x = 0.75$

$T_a = \underline{0.90}$

* S_{DS} & S_{D1} ARE IN THE NOTES AS 0.65g AND 0.13g RESPECTIVELY. THIS IS BASED ON A LOCAL STUDY DATED 9 FEB, 2009

ASCE 7-05
SEISMIC LOADS

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$C_u = 1.7$ $(S_{D1} \geq 0.1$ TABLE 12.8-1)

$T \leq C_u T_a = 1.7 \cdot 0.90 = \underline{1.53s}$

$\underset{T}{1.53s} \leq \underset{T_L}{6s}$

	D	E
C_s upper $= \dfrac{S_{D1}}{T(R/I)} =$	$\dfrac{0.56}{1.26(3/1.25)}$	$\dfrac{0.82}{1.26(3/1.25)}$
$C_{s\,up} =$	0.185	0.270

$C_s = \dfrac{S_{DS}}{\frac{R}{I}} = \dfrac{0.35}{3/1.25}$ (D) $\dfrac{0.49}{3/1.25}$ (E)

MAX CONTROLS

$C_s = \underline{0.146} \leq 0.185$ ✓ok ✓$\underline{0.204} \leq 0.270$

$V = C_s W$

WEIGHT CALCULATIONS DETERMINED IN EXCEL

TOTAL BASE SHEAR (K)

$V = W \cdot C_s$

Click to add header

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Calculate Cs									
ASCE 7-05 Estimation									
Ss	0.35	Fa	1.5	Sms	0.525		Sds	0.35	D
S1	0.06	Fv	2.4	Sm1	0.144		Sd1	0.096	
		Fa	2.1	Sms	0.735		Sds	0.49	E
		Fv	3.5	Sm1	0.21		Sd1	0.14	
I	1.25								
R	3								
		T<TL	Cs,u	=Sd1/(T*(R/I))		=	Cs,u		Cs
Ta	0.9	Y					0.0261	D	0.14583
Cu	1.7						0.0381	E	0.20417
T	1.53								
TL	6						Cs =	0.03813	

Work Back: Design Cs, T				
Given		Calcs		
V	946	Cs =	Vspec/W	
Sds	0.65	=	0.052	
Sd1	0.13	* assume T<TL		
I	1.25	Cs,u = Sd1/(T(R/I))		
R	3	T =	(Sd1 I)/(Cs R)	
W	18220	=	1.043	s

Calculate Base Shear, Overturning Moment						
ASCE 7-05 Estimation					Design	
			Cs	0.0381	Cs,des	0.052
Level	Ht (ft)	W (k)	V	Mi	Vdes	Mides
9RN	160	483	18	2946	25	4012
9RS	142	628	24	3397	33	4626
9	140	500	19	2669	26	3635
8	124	2002	76	9463	104	12887
7	102	2342	89	9106	122	12400
6	78	4188	160	12455	217	16962
5	55	2915	111	6112	151	8323
4	41	2589	99	4048	134	5512
3	24	2155	82	1972	112	2685
2	15	419	16	240	22	326
		18220	695	52409	946	71370
		W(t)	Vb	Mov (kft)	Vb,des	Mov, des

WIND LOADS 1/4

USE LOCAL CITY BUILDING CODE, 2008.

NOTE: DUE TO COMPLEX GEOMETRY OF BUILDING, THIS ANALYSIS ASSUMES RECTANGULAR DIMENSIONS WITH FLAT ROOF AT HEIGHT = 160 ft. ALSO THE PROJECT MUST BE ASSUMED TO BE AT A DISTANCE ≥ 2000 ft TO THE RIVER (BC 1609.6.1)

V = 98 MPH (BC 1609.3) (verified by structural notes)

EXPOSURE CATEGORY C : OPEN TERRAIN (verified by structural notes)
* REFFERS TO RIVER ON W SIDE (BC 1609.4)

I = 1.15 FOR WIND (BC TABLE 1604.5) (verified by structural drawings)

WIND LOADS PER BC 1609.6.5.1

HT	PSF
0 - 100 ft	20
100 - 200 ft	25

AS THIS DOES NOT MAKE SENSE, WIND LOADS CAN BE DETERMINED PER ASCE 7-05 CH. 6 (BC 1609.1.1)

CONSIDER ABOVE SIMPLIFYING ASSUMPTIONS
RISK III IS CONSISTENT
$V = 98$ MPH (BC 1609.3)
$K_d = 0.85$ (TABLE 6-4)
$I = 1.15$ (TABLE 6-1)

BC 2008 EXP CATEGORY "C" IS SURFACE ROUGHNESS "C" IN ASCE 7

$K_z = 1.39$ (TABLE 6-3)

TOPOGRAPHICAL FACTOR, $K_{zt} = 1.0$ (6.5.7.2)

GUST EFFECT FACTOR: BUILDING IS NOT RIGID PER SEISMIC CALCS

$$G_f = 0.925\left(\frac{1 + 1.7 I_{\bar{z}}\sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1 + 1.7 g_v I_{\bar{z}}}\right) \quad \text{(EQN 6-8)}$$

ASSUMPTIONS LISTED: CALCS CARRIED OUT IN EXCEL

$n_1 = 0.83$ Hz (SEISMIC CALCS P. 2/2)

$$g_R = \sqrt{2\ln(3600 n_1)} + \frac{0.577}{\sqrt{2\ln(3600 n_1)}} \quad \text{(EQN 6-2)}$$

$g_Q = g_v = 3.4$

$$Q = \left(\frac{1}{1 + 0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}\right) \quad \text{(EQN 6-6)}$$

	EW	NS
B	126 ft	285 ft
h	170 ft	170 ft

$\bar{z} = 170$
$\ell = 650$ (TABLE 6-2)
$\bar{\epsilon} = 1/8$ (TABLE 6-2)

$$L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}} \quad \text{(EQN 6-7)}$$

2/4

WIND LOADS

$$R = \sqrt{\frac{1}{\beta} R_n R_h R_B (0.53 + 0.47 R_L)}$$ (EQN 6-10)

$\beta = 0.01$ (ASSUME)

$$R_n = \frac{7.47 N_1}{(1 + 10.3 N_1)^{5/3}}$$ (EQN 6-11)

$$N_1 = \frac{n_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}}$$ (EQN 6-12)

$$\bar{V}_{\bar{z}} = \bar{b} \left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}} V$$

$\bar{b} = 0.8$ (TABLE 6-2)
$\bar{\alpha} = 1/4$ (TABLE 6-2)

$$R_\ell = \frac{1}{\eta_\ell} - \frac{1}{2\eta_\ell^2}(1 - e^{-2\eta_\ell}) \quad \eta > 0 \quad ; \quad R_\ell = 1, \eta = 0$$ (EQN 6-13)

$$\eta_h = 4.6 n_1 h / \bar{V}_{\bar{z}}$$

$$\eta_B = 4.6 n_1 B / \bar{V}_{\bar{z}}$$

$$\eta_L = 15.4 n_1 L / \bar{V}_{\bar{z}}$$

$$I_{\bar{z}} = c \left(\frac{33}{\bar{z}}\right)^{1/6}$$ (EQN 6-5)

$c = 0.15$ (TABLE 6-2)

EQUATIONS ABOVE ARE CARRIED THROUGH IN EXCEL (6.5.8)

FIND $q_z = 0.00256\, K_z K_{zt} K_d V^2 I$ (EQN 6-15)

$= 0.00256 \cdot 1.391 \cdot 1.0 \cdot 0.85 \cdot 110^2 \cdot 1.15$

$q_z = 50$ PSF (6.5.10) - ROOF

FIND INTERNAL PRESSURE COEFFICIENT (6.5.11.1)

ENCLOSURE CLASSIFICATION (6.5.9) → PARTIALLY ENCLOSED

$GC_{pi} = \pm 0.55$ (FIGURE 6-5)

WIND LOADS 3/4

EXTERNAL PRESSURE COEFFICIENT 6.5.11.2

- SIMPLIFYING ASSUMPTIONS MAKE THIS A MONOSLOPE ROOF, (FLAT)
  h = 160' (TABLE 6-6)

WALL PRESSURES

	E/W	N/S
L	285	126
B	126	285
h	160	160
L/B	2.25	0.44

WALL	C_p E/W	C_p N/S	
WW	0.8	0.8	(q_z)
LW	-0.3	-0.5 ← CONTROLS	(q_h)
SIDE	-0.7	-0.7	(q_h)

- REVISIT K_z : DETERMINED IN EXCEL
- FIND PRESSURES AT EACH LEVEL

$$p = q_z G C_p - q_h G C_{pi} \qquad (6\text{-}19)$$

THE EQUIVALENT LATERAL PRESSURES WERE CALCULATED USING EXCEL.

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G_f Calculations

Constant Input Data

Parameters		E - W Wind		Table 6-2	
h	160	B	126	α bar	0.25
V =	110	L	285	b bar	0.8
n_1	0.833	N - S Wind		c	0.15
		B	285	ℓ	650
		L	126	ε bar	0.125

Misc. Factors

g_R	4.1457
I_{zbar}	0.1153

Q Factor

	E - W	N - S
Q	0.7509	0.6953
$Lbar_z$	791.8	

R Factor

	E - W	N - S
R	0.362	0.262732
β	0.01	
R_n	0.05	
N_1		5.0510
$Vbar_z$		130.5823
R_h	0.19	
nu_h		4.6950
	E - W	N - S
R_B	0.234	0.112425
R_L	0.112	0.233912

	E - W	N - S
nu_B	3.6973	8.3630
nu_L	8.3630	3.6973

Factors for Pressure Calculations

	E - W	N - S	
G_f =	0.88	0.84	(take max)
GC pi =	0.55		
Kd =	0.85		
Kzt =	1.0		
I =	1.15		

Wall Pressures

Level	ht	Kz	qz	Cp	WW	LW	Eq. Lat Pressure	vrt dist per level	PLF hrz	Load (20ft bay) K	Mi kft
					0.8	-0.55					
9R	160	1.39	42.09		6.39	-21.07	27.46	10	275	5	879
9	140	1.36	41.18		5.75	-21.07	26.82	18	483	10	1352
8	124	1.32	39.97		4.90	-21.07	25.97	19	493	10	1224
7	102	1.26	38.15		3.63	-21.07	24.70	23	568	11	1159
6	78	1.2	36.33		2.35	-21.07	23.42	23.5	550	11	859
5	55	1.11	33.61		0.44	-21.07	21.51	18.5	398	8	438
4	41	1.04	31.49		-1.05	-21.07	20.02	15.5	310	6	254
3	24	0.94	28.46		-3.17	-21.07	17.90	13	233	5	112
2	15	0.85	25.74		-5.09	-21.07	15.98	12	192	4	58
										Mov =	6333

APPENDIX E: SNOW LOAD CALCULATIONS

ASCE 7-05
SNOW LOADS
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GROUND SNOW LOAD 7.2

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

FLAT ROOF SNOW LOADS 7.3

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

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

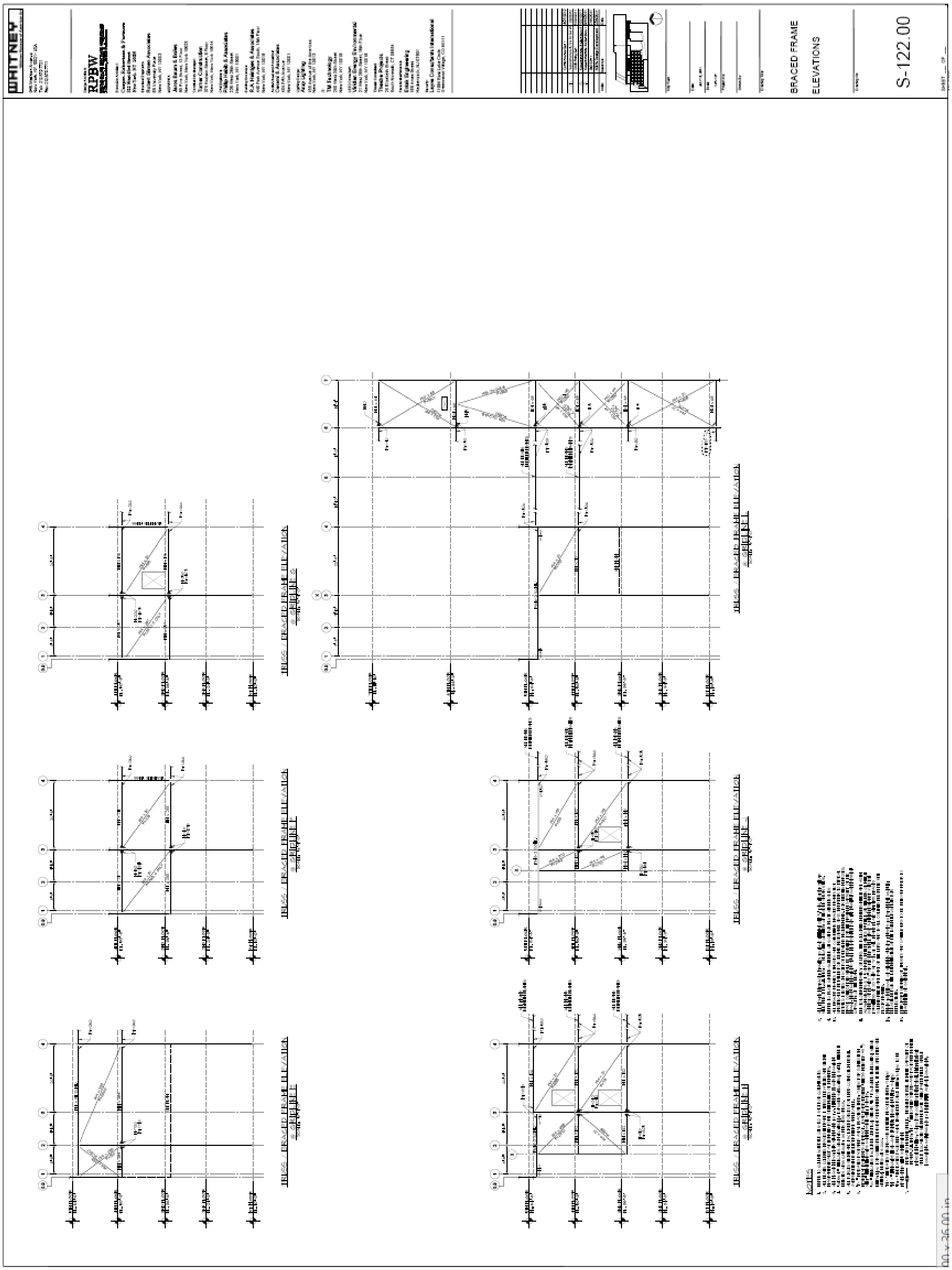
IMPORTANCE FACTOR 7.3.3
OCCUPANCY III
$I = 1.15$ - verified by structural notes

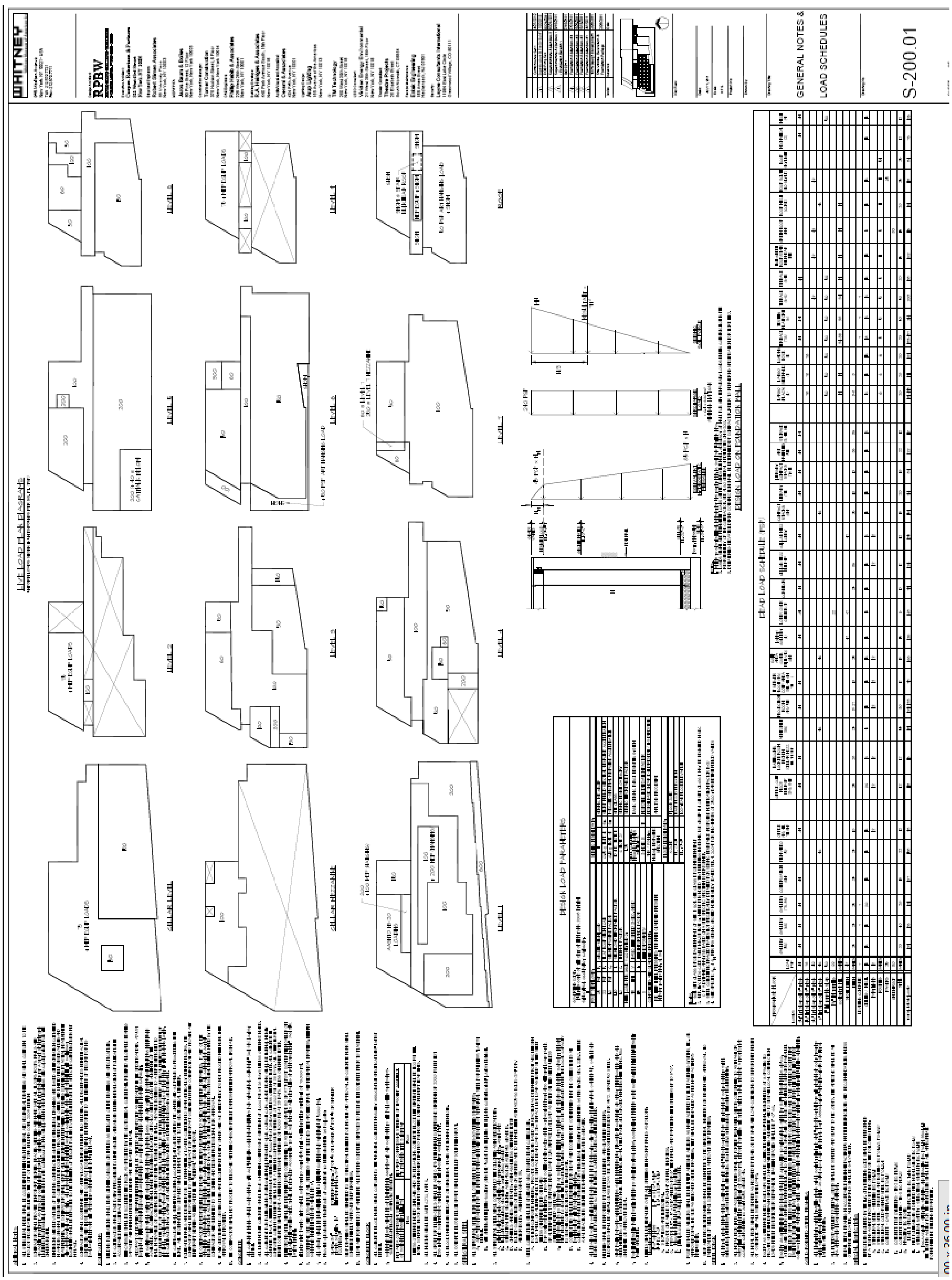
$p_f = 0.7 \cdot 1.0 \cdot 1.0 \cdot 1.15 \cdot 25 = 21$ PSF

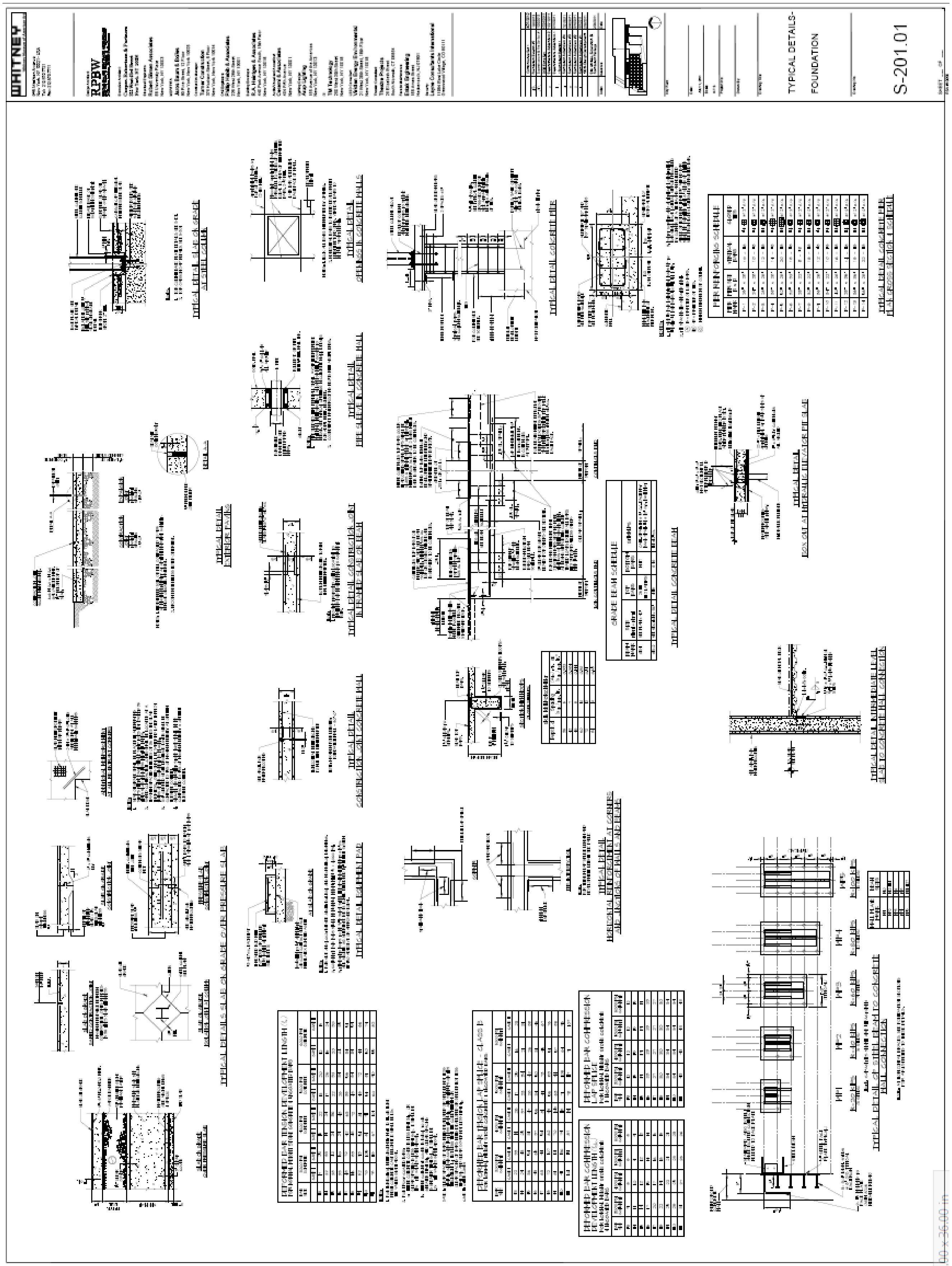
$p_{f,\,min} = 20\, I = 20 \cdot 1.15 = 23$ PSF * CONTROLS

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

APPENDIX F: TYPICAL PLANS




WHITNEY
RPBW
GENERAL NOTES &
LOAD SCHEDULES
S-200.01


WHITNEY
RPBW
TYPICAL DETAILS-
FOUNDATION
S-201.01