Technical Report 1

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Existing Conditions

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University Sciences Building

Northeast USA

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Executive Summary

The purpose of Technical Report 1 is to gain a thorough understanding of the current structure of the University Sciences Building (USB). This is accomplished through descriptions and figures summarizing the foundations, floor systems, framing systems, lateral systems, and roof systems of the USB, lists of the codes used in design and the materials used in construction, and calculation of gravity and lateral loads. Wherever possible, these calculated loads were compared to the loads used in design as given on the structural drawings.

Gravity loads were calculated or verified for the building, including the total weight of the structure. This was further investigated by checking three gravity members: an interior column, a slab panel, and a beam. These were chosen because they were reasonably representative members of the structure. All were found to be adequate, and from comparison with design loads, it was verified that the assumption made regarding 80% solidity of the slab and 90% solidity of the beam was valid.

Lateral load calculations were performed in accordance with ASCE 7-05 procedures. It was found that seismic loads will control over wind by a factor of about 2.0 in the East-West direction and 1.7 in the North-South direction. The design base shear in the North-South direction was calculated to be 938.9 k, and in the East-West direction was calculated to be 1094.5 k. These loads are within 5% of the design base shears listed on the structural drawings. It was also found that exact distribution of these forces to the lateral force resisting elements is difficult at this stage in the analysis process due to the simplifying assumptions required to use the ASCE 7-05 procedures. Further lateral analysis will be performed in Technical Report 3.

Also included in this technical report are appendices. These contain all hand calculations performed on the structure and typical drawings and sections that may be useful to this technical report.

Building Introduction

The University Sciences Building (USB) is a new building located on an urban university campus in the Northeast USA. The site chosen was previously a parking lot serving adjacent campus buildings (See Figure 1). However, the USB provides a much more appealing image on this busy street corner. It is a departure from typical campus architecture in both material usage and architectural style. However, these differences serve as a visible indication of the university's new commitment to building sustainable, functional buildings.

Figure 1 Aerial map from Google.com showing the location of the building site.

While most other campus buildings have brick facades with narrow, strip-like windows, the USB is clad largely in a prefabricated natural stone panel with aluminum-honeycomb back-up, which enables the façade to be very light. Seemingly in homage to the surrounding buildings, the USB also utilizes tall, narrow windows. However, they are of varying widths and placement on the building, which adds interest to the façade (See Figure 2). An additional feature is the 5 story atrium that forms the core of the building. It provides significant focal points such as a sweeping spiral staircase and a four-story "biowall," the first of its kind on a US university campus (See Figure 3). The biowall is used to help mitigate air quality within the building, and it is just one of many features that will help to earn the building a LEED Silver rating upon completion.

Figure 2 Exterior rendering showing the stone façade and variation of windows on the USB.

The USB is a multi-use building, incorporating four large lecture-hall style classrooms, an auditorium, several teaching and research laboratories, and faculty offices. It locates the large classrooms and administrative functions on the ground floor of the building for easy public access, but removes the laboratories and offices to the upper four stories for additional privacy. Including the mechanical penthouse, the building stands 94'-3" above grade with a partial basement. It provides the university with 138,000 square feet of new space, and has a construction cost of approximately $50 million. Construction began in August of 2009, and has an expected completion date of September 2011.

Figure 3 Interior rendering of the atrium.

Structural Overview

The University Sciences Building rests on drilled concrete caissons ranging in diameter from 36" to 58" capped by caisson caps and then grade beams. The lower five floors utilize a voided filigree slab and beam system with cast-in place concrete columns. The mechanical penthouse, however, uses steel columns and floor framing. The lateral system consists of several shear walls spanning from ground to various heights. Masonry infill walls are used between columns on the lower floors to help dampen sound from the surrounding urban environment. These non-structural walls are used solely as back-up walls to support the cladding, and were not a part of this technical report, but their design is an important consideration.

The importance factors for all calculations were based on Occupancy Category III. This was chosen because the USB fits the description of a "college facility with more than 500 person capacity," which requires Occupancy Category III.

Foundations

Geosystems Consultants, Inc. performed several test borings on the proposed site of the USB in October 2007. They found that the subsurface conditions consisted largely of extremely loose brick and rubble fill, followed by alluvium and finally residual soils with relatively low load-bearing capabilities. However, comparatively intact bedrock was encountered approximately 25 feet to 34 feet below the surface of the site.

In light of these conditions, traditional shallow spread footings would not be acceptable. Both driven steel H-piles and drilled caissons were considered as options for deep foundations, but H-piles were rejected due to vibration concerns within the subway station adjacent to the site, as well as noise concerns for the surrounding academic buildings. Instead, drilled caissons ranging in diameter from 36" to 58" were chosen to carry the loads from grade beams to the bedrock below. It was also recommended that the fill under the slab on grade (SOG) comprising the majority of the first floor be removed to a level of approximately 4 feet below the surface, followed by heavy compaction of subsurface materials, and then backfilled with structural fill to minimize settlement of the SOG due to the extremely poor load-bearing capacity of the brick/rubble fill.

Lastly, groundwater observation wells were installed, and groundwater was found to be present approximately 13 feet to 18 feet below the surface of the site. This is a potential concern, because some of the basement walls are 14 feet underground, and could encounter some loading due to hydrostatic pressure, particularly in seasons where the groundwater table rises due to rain. This was not evaluated in this technical report, but is a consideration for future design.

Floor Systems

Although it may not appear so upon first glance at the very irregular shape of the building, the bay sizes are relatively consistent throughout the USB. It simply rotates the bays as necessary to accommodate the different rotations of the wings of the building. Figure 4 shows a typical floor plan with the different bay sizes highlighted with different colors. The legend lists the bay sizes with the span required for the slab first, and then the span required for the girder (if one is present).

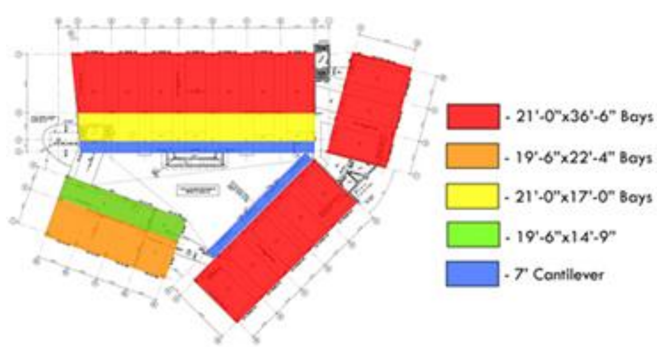


Figure 4 Floor plan from Sheet S203 showing typical bay sizes.

All of the elevated floors of the USB are a voided filigree system. This is a hybrid of precast, prestressed concrete and cast-in-place concrete. In essence, it consists of 2 ¼" of precast, prestressed concrete that functions as leave-in formwork. This is assembled and shored on site, followed by the placement of top and additional bottom reinforcing (if required, placed on rebar chairs on the bottom of the precast), and then further concrete is cast in place to unite the system. To help reduce the weight of the structure, polystyrene voids are incorporated where the concrete is not required for structural strength. Wire joists referred to as "filigree trusses" are used to transfer horizontal shear over the cold joint between precast and cast-in-place concrete.

Three separate systems were used, depending on the required spans and uses. For areas that include a span above 36 feet (typically laboratories), an 8" voided filigree slab (V.F.S.) was used to span between 18" deep voided filigree beams (V.F.B.). A schematic layout of this type of system, used in the majority of the building, is shown in Figure 5. In the Office Wing (shown in Figure 4 in green and orange), where shorter spans were allowed, the beams were removed from the system and the slab was thickened to 10 inches total depth. However, the cross section of this slab remains similar to the condition shown in the "Section 3" within Figure 5. Lastly, in the two "links" (shown in Figure 6), this flat plate is thickened to 12 inches total depth, again with a similar condition to "Section 3" in Figure 5. These links are the uniting elements in the building, and had to be cast last on every floor. These are united to the building with rebar across the cold joint rather than an official expansion joint.

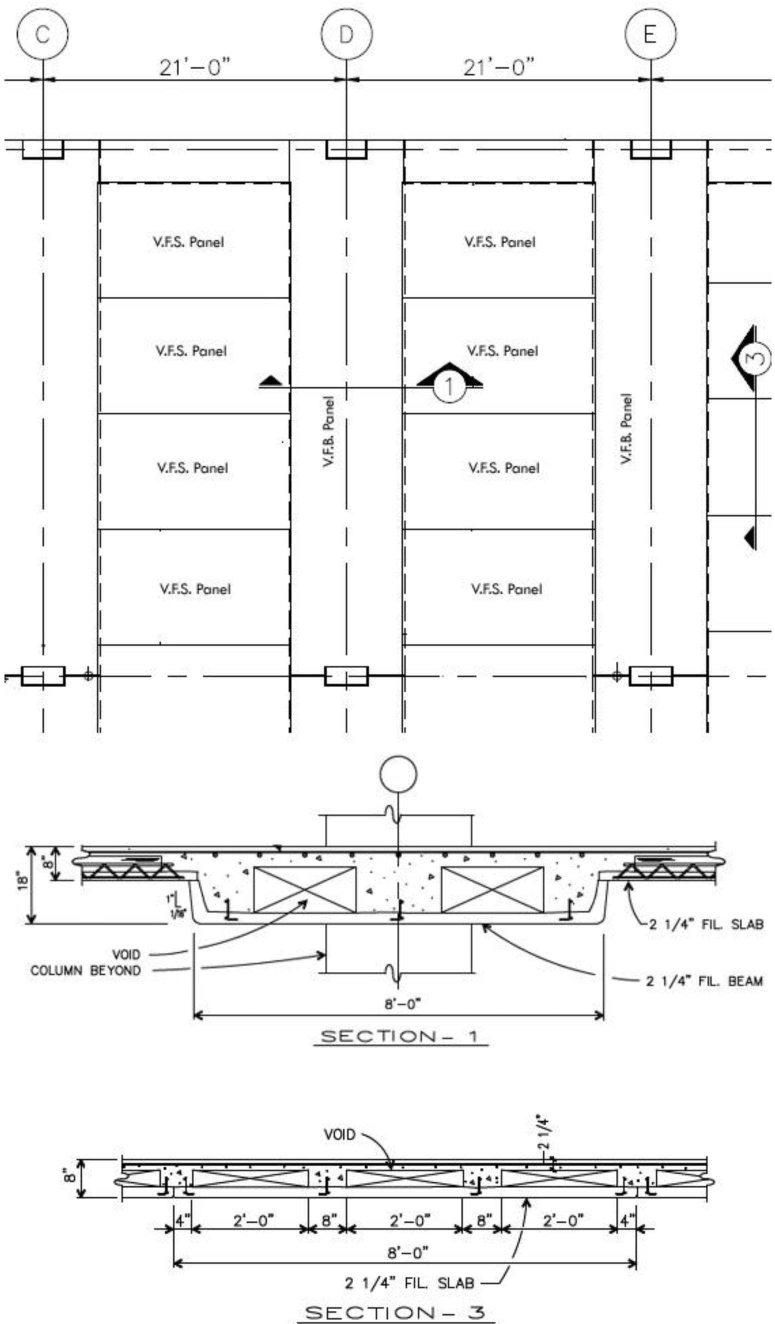


Figure 5 Typical bay with section cuts showing the condition within the beam and the slab. Modified from the filigree slab shop drawings and not to scale (NTS).

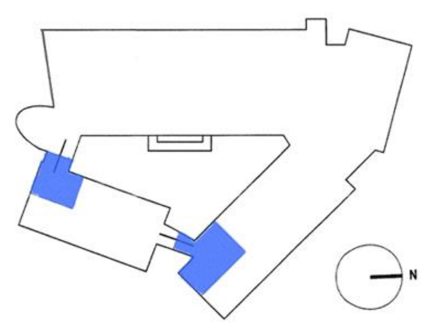


Figure 6 Modified keyplan from Sheet S202 showing the "link" areas in blue.

Framing System

The columns in the lower five stories of the USB are all cast-in-place concrete. The columns closest to the atrium on the ground floor are round columns 2 feet in diameter. Most are changed at the second level to 36"x16" rectangular columns. All other columns are 36"x16" columns, rotated as required to fit into walls. At the penthouse level, the columns change to A572 steel W-shapes. These columns range in size from W8x40 to W8x67.

Lateral System

Shear walls are the main lateral force resisting system in the USB. They are scattered throughout the building to best resist the lateral forces in the building (See Figure 7). All of these walls are 12" thick cast-in-place concrete. Most span from ground level to the roof, but since roof heights vary, they are not necessarily the same height (See Appendix C for detailed shear wall data). They are anchored at the base by grade beams that run the full length of the walls. This is a potential overturning concern due to the large forces that can occur on a shear wall. This concern was not investigated in depth in this technical report. However, Sheet S310 contains the structural engineer of record's calculations with regard to uplift on the caissons (this has been included as Figure 8 in Appendix D). Another issue not investigated for this technical report, but that will be of concern later, are the checks for force transfer at the link elements to ensure that the lateral forces are able to reach the shear walls.

Figure 7 Typical floor plan taken from Sheet S203. Shear walls are indicated in blue.

Roof Systems

There are six different roofs on the USB, due mostly to architectural reasons. Figure 8 shows these roofs and their heights above the ground reference elevation of 0'-0". The Office roof (shown in red) is at the same elevation as the fifth floor. Its structure is a 10" flat plate filigree slab system, similar to the office floors below it. The "Ledge" roof (shown in orange) is at the same level as the Penthouse floor, and is a continuation of the 10" V.F.S./24" V.F.B. system used in the adjacent AHU Mechanical Room. The atrium roof, 5th Level Mechanical Room roof, and AHU Mechanical Room roof (shown in yellow, green, and purple, respectively) are all 3" P2404 Canam roof deck on steel W-shape framing. The Chiller Mechanical Room roof (shown in blue) is 3" of cast-in-place concrete topping on 3" P2432 Canam composite deck (6" total depth) supported by W-shape framing. This heavier structure is necessary because this roof supports two large cooling towers and a diesel generator. This roof is also the only one with a parapet, which serves as a screen to hide the mechanical equipment and stretches from this roof level to 94'-3".

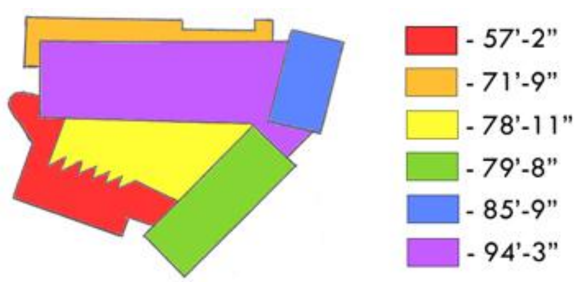


Figure 8 Modified keyplan image from Sheets S205, & S206 showing different roof heights in relation to 0'-0"

Regardless of the underlying structure, all roofs receive the same finish. This consists of sloped rigid insulation under Thermoplastic-Polyolefin (TPO) single-ply membrane.

Design Codes

According to Sheet S001, the original building was designed to comply with:

- 2006 International Building Code (IBC 2006) with Local Amendments
- 2006 International Mechanical Code (IMC 2006) with Local Amendments
- 2006 International Electrical Code (IEC 2006) with Local Amendments
- 2006 International Fuel Gas Code (IFGC 2006) with Local Amendments
- Local Fire Code based on the 2006 International Fire Code (IFC 2006) with Local Amendments.
- Minimum Design Loads for Buildings and Other Structures (ASCE 7-05)
- Building Code Requirements for Structural Concrete (ACI 318-08)
- Masonry Construction for Buildings (ACI 530)
- AISC Manual of Steel Construction, Load Resistance Factor Design (LRFD)

These are also the codes that were used to complete the analyses contained in this technical report, with heavy emphasis on the use of ACI 318-08 and ASCE 7-05.

Materials Used

Due to the variety of structural types on this project, there are also many different kinds of materials. These are listed in Figure 9 below. All information was derived from Sheet S001.

Concrete		
Usage	Weight	Strength (psi)
Caissons	Normal	3000
Caisson Caps	Normal	3500
Footings	Normal	3500
Foundation Walls	Normal	4500
Shear Walls	Normal	4500
Slab-on-Grade	Normal	3500
Columns	Normal	5000
Structural Slabs/Beams	Normal	4500
Precast	Normal	5000
Housekeeping Pads	Normal	3500
Concrete on Steel Deck	Normal	3000

Steel		
Type	Standard	Grade
W-Shaped Structural Steel	ASTM A572	50
Hollow Structural Sections (HSS)	ASTM A500	C
Anchor Rods	ASTM F1554	N/A
Bolts, Washers, and Nuts	ASTM A325	N/A
3/4"x4 1/2" Long Welded Shear Studs	ASTM A496	N/A
Steel Deck	ASTM A653	A or B
Deformed Reinforcement Bars	ASTM A615	60
Welded Wire Fabric	ASTM A185	N/A

Masonry		
Type	Standard	Strength (psi)
Concrete Masonry Units	ACI 530	2175
Mortar	ASTM C270	N/A
Grout	ASTM C475	3000-5000

Miscellaneous	
Type	Strength (psi)
Non-Shrink Grout	10,000

Figure 9 Summary of materials used on the USB project with design standards and strengths.

Gravity Loads

As a part of this technical report, dead, live and snow loads were all calculated and compared to loads listed on the structural drawings. Following basic load documentation, several gravity members in the structure were checked to verify their adequacy. Detailed calculations for these gravity member checks can be found in Appendix A.

Dead and Live Loads

The structural drawings list superimposed dead loads, summarized in Figure 10. Analyses found that these loads are accurate, although conservative in some cases. The ceiling and mechanical load applied is potentially higher than usual, but this can be explained by the large ductwork required to bring 100% outside air into the laboratory spaces. The uniform application of housekeeping pad loads to mechanical and electrical spaces is conservative because these pads are scattered over these spaces. However, these loads seem to be calculated by weight of concrete required for the depth of the pad specified. The masonry walls in the structure are 8" concrete masonry unit (CMU), weighing approximately 60 pounds per square foot (psf). Thus, the masonry wall load corresponds to a 14 foot high 8" CMU wall.

Superimposed Dead Loads	
Description	Load
1st Level Ceiling/Mechanical	10 psf
Other Levels Ceiling/Mechanical	15 psf
Electrical Room 4" Housekeeping Pad	55 psf
Mechanical Rooms 6" Housekeeping Pads	80 psf
Roofing	20 psf
Topping on Office Roof	36 psf
Masonry Wall	840 plf

Figure 10 Summary of Superimposed Dead Loads.

Following the verification of the superimposed dead loads, estimations were made in order to calculate the overall building weight (which was also used in seismic calculations). By looking at typical sections through filigree slabs and beams, it was decided to consider the slabs 80% solid concrete and the beams 90% solid concrete.

Also considered in the building weight calculation were the weights of the columns, shear walls, superimposed dead loads, roofs, and wall loads (both exterior and interior). The exterior walls were considered to be 60 psf, as they are 8" CMU back-up walls with a cladding that weighs approximately 1 psf. The results of this calculation are summarized per level with the weights of a typical level shown in more detail in Figure 11. The overall building weight was found to be approximately 30,500 k.

Live loads were also listed on the structural drawings. These were compared to the live loads in Table 4-1 in ASCE 7-05 based on the usage of the spaces, and the results are summarized in Figure 12. Although many of these loads matched their ASCE 7-05 counterparts, some exceed the minimum significantly.

The large classrooms on the first floor were all designed for 100 psf, which is the design load for assembly areas with movable seating. These classrooms all have fixed seating, but it is possible that this was not yet decided at the time of the initial structural design, and therefore the more conservative load was used.

There is no provision for laboratories in classroom or research facilities, so the provision for "Hospitals – Operating Rooms, Laboratories" was used for comparison. It is possible that this was exceeded because most of these labs are to be teaching facilities, where occupant loads could exceed typical values depending on class sizes.

Weight per Level		
Level	Area (ft^2)	Weight (psf)
Ground	25,459	131.62
2nd	21,135	217.83
3rd	21,135	216.39
4th	21,135	216.39
5th	22,215	234.24
Penthouse	22,602	265.50
Roof	12,780	170.28

Weight of a Typical Floor (3rd Level)			
Description	Weight	Quantity	Total Weight (k)
8" VFS/18" VFB	127 psf	17,200 ft^2	2184.40
10" VFS	100 psf	2,890 ft^2	289.00
12" VFS	120 psf	1,045 ft^2	125.40
Superimposed DL	15 psf	21,135 ft^2	317.03
(43) 36"x16" Columns	600 plf/col	14 ft/col	361.20
Shear Wall	2100 plf	350 ft	735.00
Exterior Wall	840 plf	670 ft	562.80
		Total Weight=	4574.83 k
		Weight per Square Foot=	216.46 psf

Note: Values may differ slightly from values in "Weight per Level" table due to simplifications made in this table to allow for grouping

Figure 11 Summary of building weight per level and a typical level.

The last major discrepancy was the live load on the Office Roof. This roof was accessible during construction, and was used for materials storage during this phase of the building's life. It is possible this load was increased to account for the loads associated with this, such as workers on the roof to access materials stored there.

It was also noted on the structural drawings that live load reduction was used where allowed by code. Therefore, live load was reduced wherever possible for all gravity calculations in this technical report.

Snow Loads

The roof snow load was calculated using the procedure outlined in Chapter 7 of ASCE 7-05, and the factors required for this calculation are summarized in Figure 13. The structural drawings used a C_t of 0.8, but this does not seem to be permissible by code. Therefore, the

Live Loads			
Space	Design Live Load (psf)	ASCE 7-05 Live Load (psf)	Notes
Atrium	100	100	N/A
Large Classrooms	100	60	Fixed Seating in all
Laboratories	80	60	Based on "Hospitals - Laboratories"
Offices	50+20	50+20	Office Load+Partition Load
Links/Stairs	100	100	N/A
5th Level Lab	80+20	60+20	Based on "Hospitals - Laboratories"+ Partition Load
5th Level Mech. Room	100	N/A	N/A
Electrical Room	150	N/A	N/A
Office Roof	50	20	May be due to construction loading
AHU Mechanical Room	100	N/A	N/A
Chiller Mechanical Room	150	N/A	N/A
Other Roofs	20	20	N/A

Figure 12 Summary of design live loads, compared to ASCE 7-05 typical live loads.

Flat Roof Snow Load Calculations	
Variable	Value
Ground Snow Load, p_g (psf)	30
Temperature Factor, C_t	1.0
Exposure Factor, C_e	1.0
Importance Factor, I_s	1.1
Flat Roof Snow Load , p_f (psf)	23.1

Figure 13 Summary of roof snow load calculations.

drawings used a flat roof snow load of 20 psf, whereas 23.1 psf was calculated (and used for all subsequent calculations) in this technical report.

Due to the different roof heights, ten locations of possible drifting were identified. The magnitudes of these drifts were calculated, and the results can be found in Appendix A. The structural drawings only contain additional snow loads for four of these locations, but the loads listed on the drawings seem to coincide with the loads calculated for this technical report reasonably well.

Column D/2 Gravity Check

This column was chosen because it is an interior column not located near a shear wall (see Figure 14). As the columns are not a part of the lateral force resisting system, lateral influences are unlikely to be a significant concern for this column, and subsequently second order effects were disregarded in this calculation. It is a 36"x16" rectangular concrete column reinforced with (12) #8 vertical bars and #3 ties at 12" on center for the first five levels, and then transitions to a W8x40 at the penthouse level. Loads were calculated at each level, and the final check was performed at the Ground Level. The column schedule entry for this column has been included as Figure 7 in Appendix D. This lists column service design loads that were used for comparison to the hand calculations.

It was found that Column D/2 is more than adequate to carry the associated gravity loads. The design live loads were used as opposed to the ASCE 7-05 live loads for comparison purposes. The only major discrepancy for live load occurred at the 2nd Level, where 33% of the live load could not be accounted for. Dead load calculations were extremely accurate, except the dead load calculated at the Penthouse Level, which fell short by approximately 19%. This could be explained if the column carries more than the assumed 10,000 lbs of mechanical equipment. Another possibility is that the 80% solid slab/90% solid beam assumption no longer applies at this level. This may be the case if a more solid structure was used to carry the heavy mechanical equipment located above this column at the Penthouse Level.

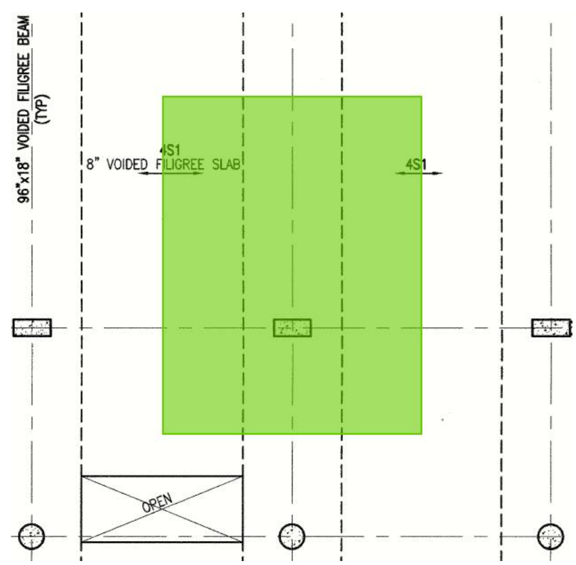


Figure 14 Column D/2, with its approximate tributary area shown in green. Modified from Sheet S204.

Voided Filigree Slab Gravity Check

In the interest of performing a calculation that would be applicable to several areas, this check was done on a voided filigree slab (V.F.S.) panel spanning between column lines C & D on the 4th Level (see Figure 15). The 2nd, 3rd, and 4th Levels are identical in design, and the 21'-0" spacing between column lines is used in several areas. This is an 8" V.F.S. spanning between 96"Wx18"D voided filigree beams. From comparison with positive design moments listed on the drawings for this and adjacent spans with the same length (A to B and B to C), it was concluded that the ACI moment coefficients for continuous beams would

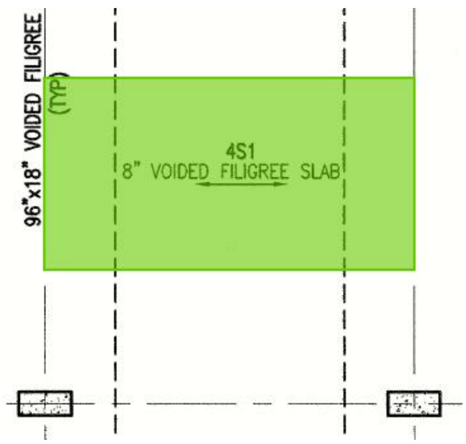


Figure 15 V.F.S., with its approximate tributary area shown in green. Modified from Sheet S204.

produce similar design values (Figure 6 in Appendix D shows this bay's reinforcing and design moments). Therefore, these coefficients were used for this report.

Checks were performed for positive moment capacity, negative moment capacity, vertical shear, horizontal shear, and deflections for this slab panel. The positive design moment calculated in this technical report was slightly greater than the original design moment. This could be attributed to discrepancies in calculations of clear length, continuous beam moment coefficients, or assumptions regarding the dead load values. The last option seems unlikely, since dead loads matched so closely to design loads for this level in the calculations for Column D/2. Even with a slightly larger design moment, the member was found to be adequate for all of the aforementioned conditions.

Voided Filigree Beam Gravity Check

Again in the interest of performing a calculation that would be applicable to several areas, this check was done on a voided filigree beam (V.F.B.) spanning between column lines 1 & 2 along column line D on the 4th Level (see Figure 16). This beam spans 36'-4", which is the most common span for a V.F.B. in the structure. With a cross section of 96"Wx18"D, this is also the most typical V.F.B. size used in the project. Since the slab was designed with moments close to those obtained using ACI moment coefficients, it was assumed that the beam would also have a good correlation this way, and the coefficients were used for the calculation of all moments in the beam (Figure 6 in Appendix D shows this bay's reinforcing and design moments).

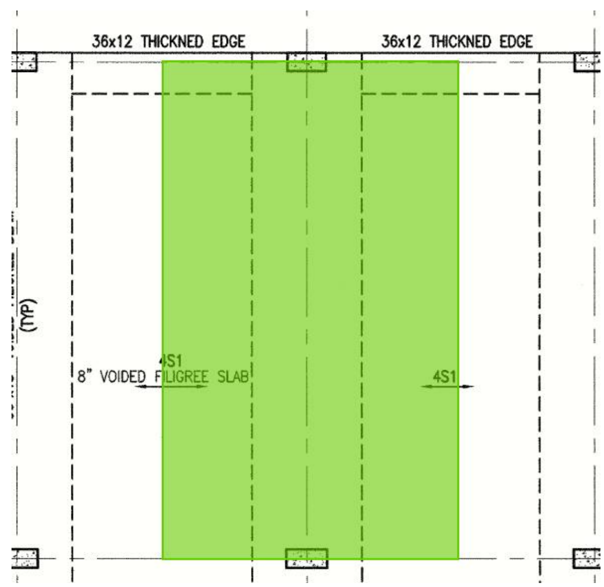


Figure 16 V.F.B., with its approximate tributary area shown in green. Modified from Sheet S204.

Checks were performed for positive moment capacity, negative moment capacity at both supports (acknowledging that the support at column line 1 is an exterior column, and therefore the support at column line 2 is the exterior face of the first interior support), vertical shear, horizontal shear, and deflections. Again, the positive design moment calculated in this technical report was slightly greater than the original design moment. The discrepancy is likely due to one of the reasons listed for the V.F.S., although an excessive dead load still seems unlikely.

Lateral Loads

In order to better understand the lateral systems, wind loads and seismic loads were calculated for this technical report. At this point in the evaluation of this structure, it is difficult to know exactly how much force is distributed to each shear wall because of the irregularity of the structure and the simplifying assumptions necessary to be able to perform hand calculations. However, a more extensive analysis of the lateral system will be conducted for Technical Report 3. For Technical Report 1, the hand calculations associated with wind loading and seismic loading can be found in Appendices B and C, respectively.

Wind Loads

Wind loads were calculated with the Method 2 Main Wind Force Resisting System (MWRFS) procedure identified in ASCE 7-05 Chapter 6. In order to be able to use this procedure, several simplifying assumptions had to be made. First, the building was modeled with a single roof height of 94'-3". Next, the surface areas were projected onto North-South (N-S) and East-West (E-W) axes, and the projected lengths were used to calculate wind pressures. However, using these projected building lengths for the calculation of L and B would be potentially unconservative. Thus, a "pseudo-footprint" was developed, and the area of the pseudo-footprint was transformed into a representative rectangle. The dimensions of this rectangle were then used as L and B.

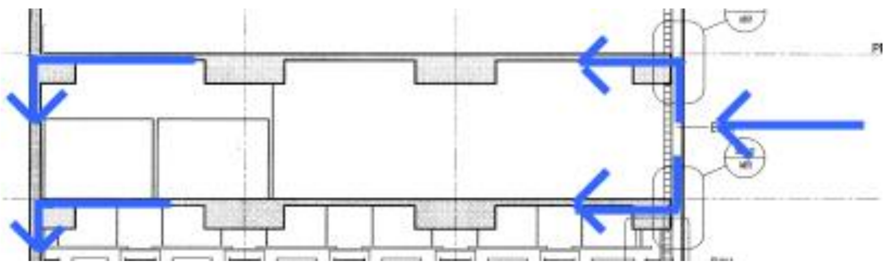


Figure 17 Diagram of the lateral load path for a wind load.

The wind loads on this building are collected by the cladding on the exterior of the building. As a result, a more detailed analysis of wind pressures on the cladding will be required in Technical Report 3, with particular attention paid to uplift on the roof. The cladding transfers these loads to the CMU back-up walls, which are in turn anchored to the slabs with masonry dowels. This transfers the load into the slabs, which then carry the load to the shear walls. These return the loads the foundations, and therefore to grade. This load path is illustrated in Figure 17.

Most calculations were performed using Microsoft Excel to simplify a potentially repetitive process. Wind pressures, including windward, leeward, sidewall, and internal pressure were found. These were then used to calculate the story forces at each level. It should be noted that the story forces include windward and leeward pressures, but not internal pressure, because internal pressure is effectively self-cancelling.

The wind pressures in the N-S direction are listed and diagramed in Figure 18. These were resolved into wind forces in the N-S direction, which are listed and diagramed in Figure 19. The resulting base shear is 281.4 k, which is about 13% less than the base shear for this wind direction listed on Sheet S001 (325 k).

Wind pressures were also calculated for the E-W direction, and are listed and diagramed in Figure 20. These were resolved into wind forces in the E-W direction, which are listed and diagramed in Figure 21. The resulting base shear is 407.6 k, which is about 12% less than the base shear for this wind direction listed on Sheet S001 (465 k). These discrepancies may be due to differing simplifying assumptions. However, this is not a major concern because the lateral system is controlled in both directions by seismic loads.

Wind Pressures - N-S Direction							
Type	Floor	Distances (ft)	Wind Pressure (psf)	Internal Pressure (psf)		Net Pressure (psf)	
				(+)(GC_{pi})	(-)(GC_{pi})	(+)(GC_{pi})	(-)(GC_{pi})
Windward Walls	Ground	0.00	7.82	3.55	-3.55	4.28	11.37
	2nd	15.17	7.85	3.55	-3.55	4.30	11.39
	3rd	29.17	9.52	3.55	-3.55	5.97	13.06
	4th	43.17	10.65	3.55	-3.55	7.10	14.20
	5th	57.17	11.51	3.55	-3.55	7.97	15.06
	Penthouse	71.75	12.31	3.55	-3.55	8.77	15.86
	Roof	94.25	13.34	3.55	-3.55	9.80	16.89
Leeward Walls	All	All	-6.50	3.55	-3.55	-10.05	-2.96
Side Walls	All	All	-11.67	3.55	-3.55	-15.22	-8.13
Roof	N/A	0-47	-15.01	3.55	-3.55	-18.56	-11.46
	N/A	47-94	-15.01	3.55	-3.55	-18.56	-11.46
	N/A	94-188	-8.34	3.55	-3.55	-11.88	-4.79
	N/A	>188	-5.00	3.55	-3.55	-8.55	-1.46

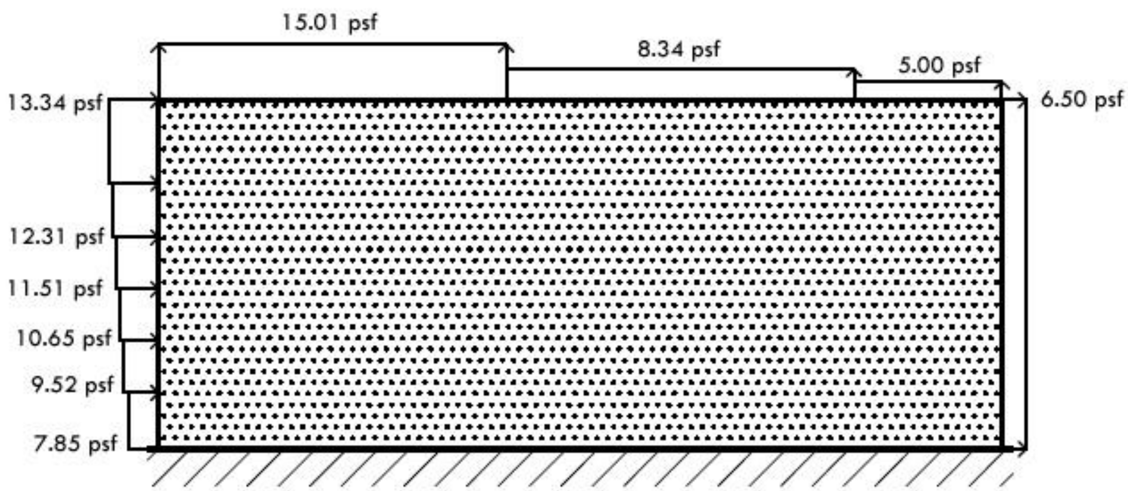


Figure 18 List and diagram of N-S direction wind pressures.

Wind Forces - N-S Direction								
Floor Level	Elevation (ft)	Trib. Below		Trib. Above		Story Force (k)	Story Shear (K)	Overturning Moment (k-ft)
		Height (ft)	Area (ft^2)	Height (ft)	Area (ft^2)			
Ground	0.00	N/A	0.00	7.59	1289.45	18.50	281.37	0.00
2nd	15.17	7.59	1289.45	7.00	1190	37.57	262.87	569.88
3rd	29.17	7.00	1190.00	7.00	1190	39.47	225.30	1151.47
4th	43.17	7.00	1190.00	7.00	1190	41.85	185.83	1806.59
5th	57.17	7.00	1190.00	7.29	1239.3	44.75	143.98	2558.64
Penthouse	71.75	7.29	1239.30	11.25	1912.5	61.27	99.22	4396.17
Roof	94.25	11.25	1912.50	N/A	0.00	37.95	37.95	3577.06
							Total Base Shear=	281.37 k
							Total Overturning Moment=	14,059.80 k-ft

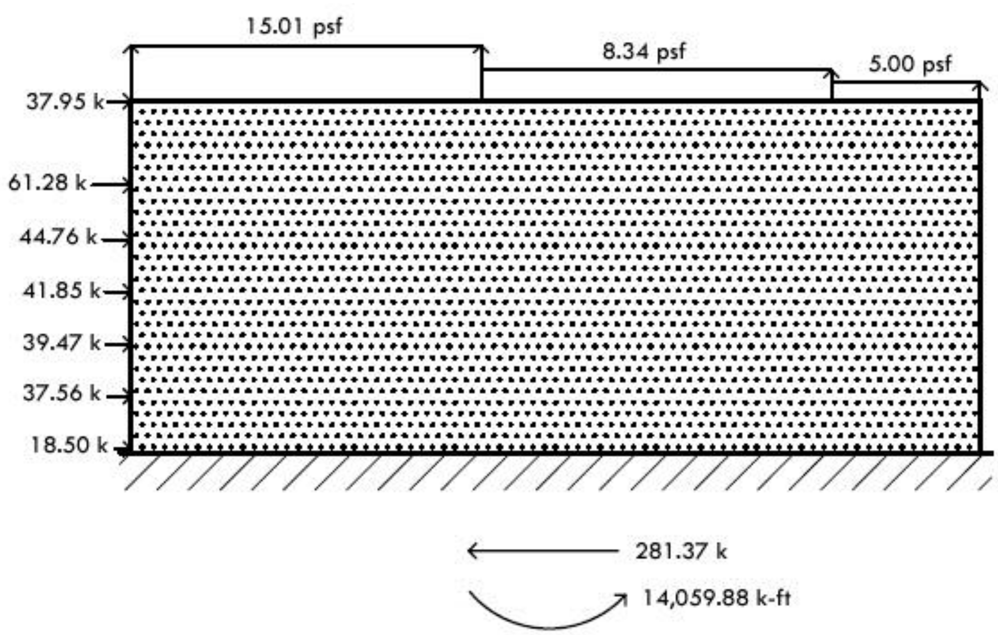


Figure 19 List and diagram of N-S direction wind forces.

Wind Pressures - E-W Direction							
Type	Floor	Distances (ft)	Wind Pressure (psf)	Internal Pressure (psf)		Net Pressure (psf)	
				(+)(GC_{pi})	(-)(GC_{pi})	(+)(GC_{pi})	(-)(GC_{pi})
Windward Walls	Ground	0.00	7.65	3.55	-3.55	4.10	11.20
	2nd	15.17	7.67	3.55	-3.55	4.13	11.22
	3rd	29.17	9.31	3.55	-3.55	5.76	12.85
	4th	43.17	10.41	3.55	-3.55	6.87	13.96
	5th	57.17	11.26	3.55	-3.55	7.71	14.80
	Penthouse	71.75	12.04	3.55	-3.55	8.49	15.59
	Roof	94.25	13.05	3.55	-3.55	9.50	16.59
Leeward Walls	All	All	-8.15	3.55	-3.55	-11.70	-4.61
Side Walls	All	All	-11.42	3.55	-3.55	-14.96	-7.87
Roof	N/A	0-47	-17.66	3.55	-3.55	-21.21	-14.11
	N/A	47-94	-13.19	3.55	-3.55	-16.73	-9.64
	N/A	94-188	-9.65	3.55	-3.55	-13.19	-6.10
	N/A	>188	N/A	N/A	N/A	N/A	N/A

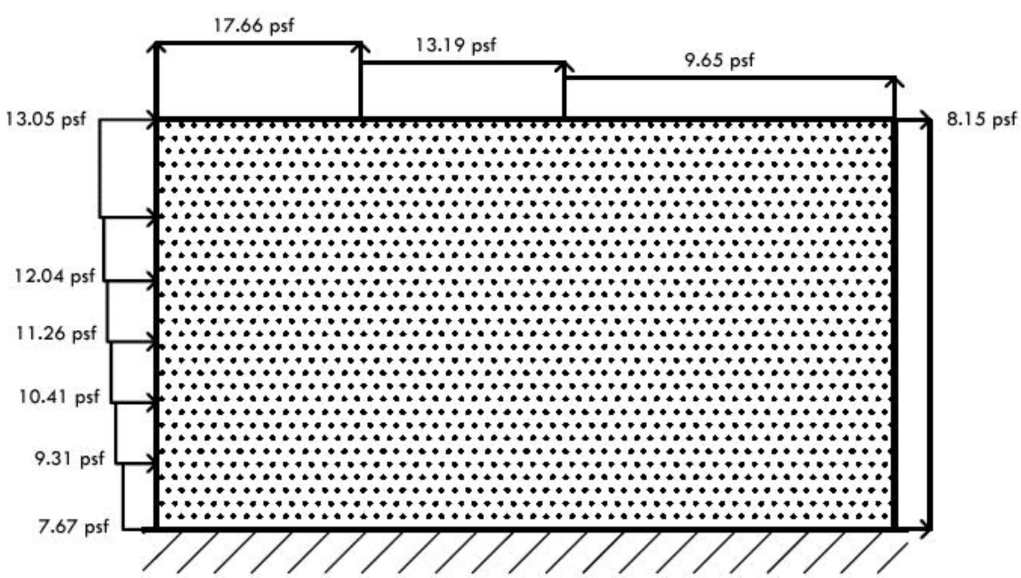


Figure 20 List and diagram of E-W direction wind pressures.

Wind Forces - E-W Direction								
Floor Level	Elevation (ft)	Trib. Below		Trib. Above		Story Force (k)	Story Shear (K)	Overturning Moment (k-ft)
		Height (ft)	Area (ft^2)	Height (ft)	Area (ft^2)			
Ground	0.00	N/A	0.00	7.59	1729.38	27.37	407.59	0.00
2nd	15.17	7.59	1729.38	7.00	1596.00	55.24	380.22	837.96
3rd	29.17	7.00	1596.00	7.00	1596.00	57.50	324.98	1677.29
4th	43.17	7.00	1596.00	7.00	1596.00	60.61	267.48	2616.70
5th	57.17	7.00	1596.00	7.29	1662.12	64.54	206.87	3690.02
Penthouse	71.75	7.29	1662.12	11.25	2565.00	87.94	142.32	6310.01
Roof	94.25	11.25	2565.00	N/A	0.00	54.38	54.38	5125.28
							Total Base Shear=	407.59 k
							Total Overturning Moment=	20,257.25 k-ft

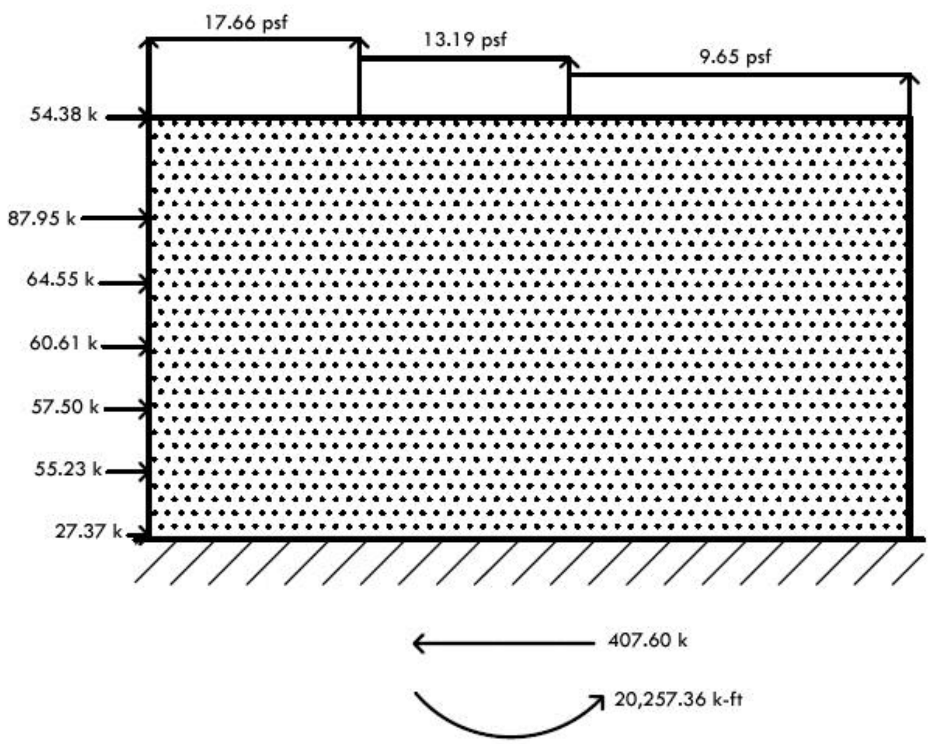


Figure 21 List and diagram of E-W direction wind forces.

Seismic Loads

Seismic loads were calculated with the Equivalent Lateral Force procedure outlined in Chapters 11 and 12 of ASCE 7-05. This procedure also assumes a simple building footprint, but the simplifications required for this were much less drastic than those required for wind calculations. The approximate fundamental period for shear walls can be calculated using the generic designation of “other structures” or the specific equation for shear walls. Both were evaluated for this technical report, and it was determined that it was more likely that the original calculations were performed with the specific equation. Therefore, the specific solution was used for the finalization of the seismic load calculations in this technical report. To perform this specific solution, the shear walls had to be resolved onto North-South (N-S) and East-West (E-W) axes. This was accomplished with trigonometry. All shear wall data can be found in Appendix C.

The loads from seismic forces originate from the inertia of the structure itself, which is related to the mass of the structure. Most of the mass of the structure is locked in the slabs, which are directly connected to the shear walls. When seismic loads are generated by a ground motion, the slabs transfer the loads directly into the shear walls, which then carry the loads down to the foundations and therefore to grade.

Seismic forces in the N-S direction are listed and diagramed in Figure 22. The resultant base shear in this direction is 938.9 k, which is about 1.7% less than the base shear listed for this direction on Sheet S001 (955 k). This extremely minor discrepancy is likely due to a combination of small differences in the calculated weight of the building and slightly different shear wall dimensions. The calculation is much more sensitive to the shear wall dimensions, and efforts will be made to model these shear walls as accurately as possible for Technical Report 3.

Seismic forces for the E-W direction are listed and diagramed in Figure 23. The resultant base shear in this direction is 1,094.5 k, which is about 4.4% less than the base shear listed for this direction on Sheet S001 (1145 k). Again, this difference is very minor, and is probably accounted for by the same combination of discrepancies indicated for the N-S direction.

Seismic Forces - N-S Direction							
Level	Story Weight, w_x (k)	Story Height, h_x (ft)	$w_x h_x^K$	C_{vx}	Story Force (k) $F_x=C_{vx}V$	Story Shear (k)	Overturning Moment (k-ft)
Ground	3350.80	0.00	0.00	0.00	0.00	938.89	0.00
2nd	4603.74	15.17	102057.42	0.04	40.92	938.89	620.65
3rd	4573.38	29.17	213605.01	0.09	85.65	897.97	2498.10
4th	4573.38	43.17	333918.50	0.14	133.89	812.32	5779.64
5th	5203.64	57.17	523280.82	0.22	209.82	678.42	11994.70
Penthouse	6000.83	71.75	781794.88	0.33	313.48	468.61	22491.92
Roof	2176.23	94.25	386884.07	0.17	155.13	155.13	14620.89
						Base Shear [$V=C_sW$]=	938.89 k
						Total Overturning Moment=	58,005.90 k-ft

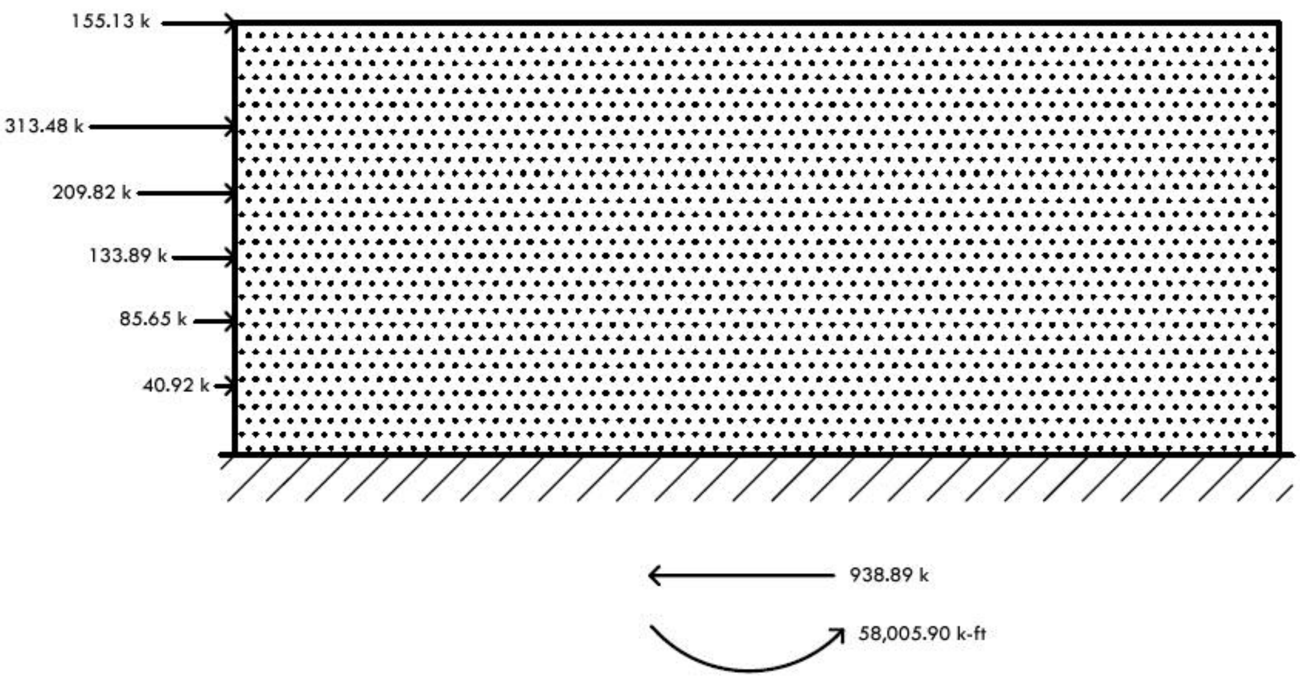


Figure 22 List and diagram of N-S direction seismic forces.

Seismic Forces - E-W Direction							
Level	Story Weight, w_x (k)	Story Height, h_x (ft)	$w_x h_x^K$	C_{vx}	Story Force (k) $F_x=C_{vx}V$	Story Shear (k)	Overturning Moment (k-ft)
Ground	3350.80	0.00	0.00	0.00	0.00	1094.48	0.00
2nd	4603.74	15.17	87789.25	0.05	51.32	1094.48	778.41
3rd	4573.38	29.17	177206.27	0.09	103.60	1043.15	3021.65
4th	4573.38	43.17	271068.00	0.14	158.47	939.56	6840.78
5th	5203.64	57.17	418230.74	0.22	244.51	781.08	13977.76
Penthouse	6000.83	71.75	617033.37	0.33	360.73	536.57	25882.68
Roof	2176.23	94.25	300770.92	0.16	175.84	175.84	16572.80
						Base Shear [$V=C_sW$]=	1,094.48 k
						Total Overturning Moment=	67,074.08 k-ft

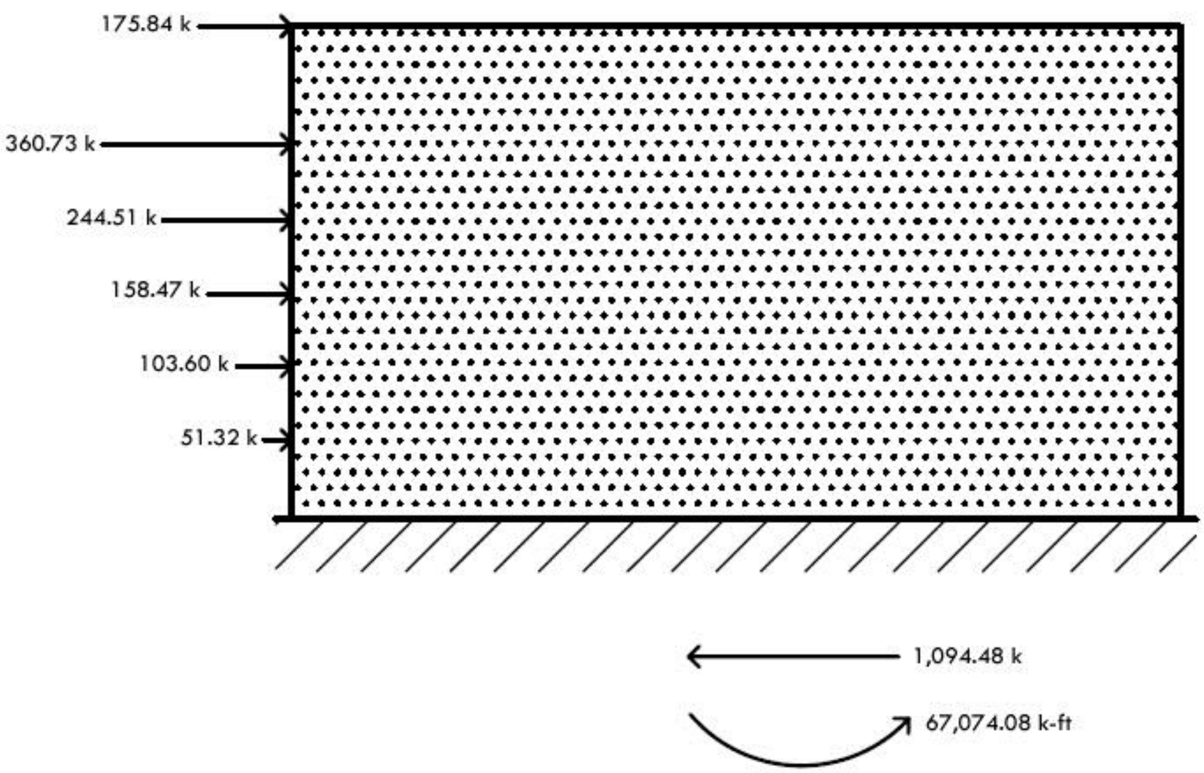


Figure 23 List and diagram of E-W direction seismic forces.

Conclusion

Technical Report 1 analyzed the existing structural conditions of the University Sciences Building. The foundations, floor systems, framing systems, lateral systems and roof systems were all summarized with descriptions and figures intended to fully describe the structure as it is presently designed. The use of voided filigree slab/beam construction makes this structure interesting and complicated to analyze.

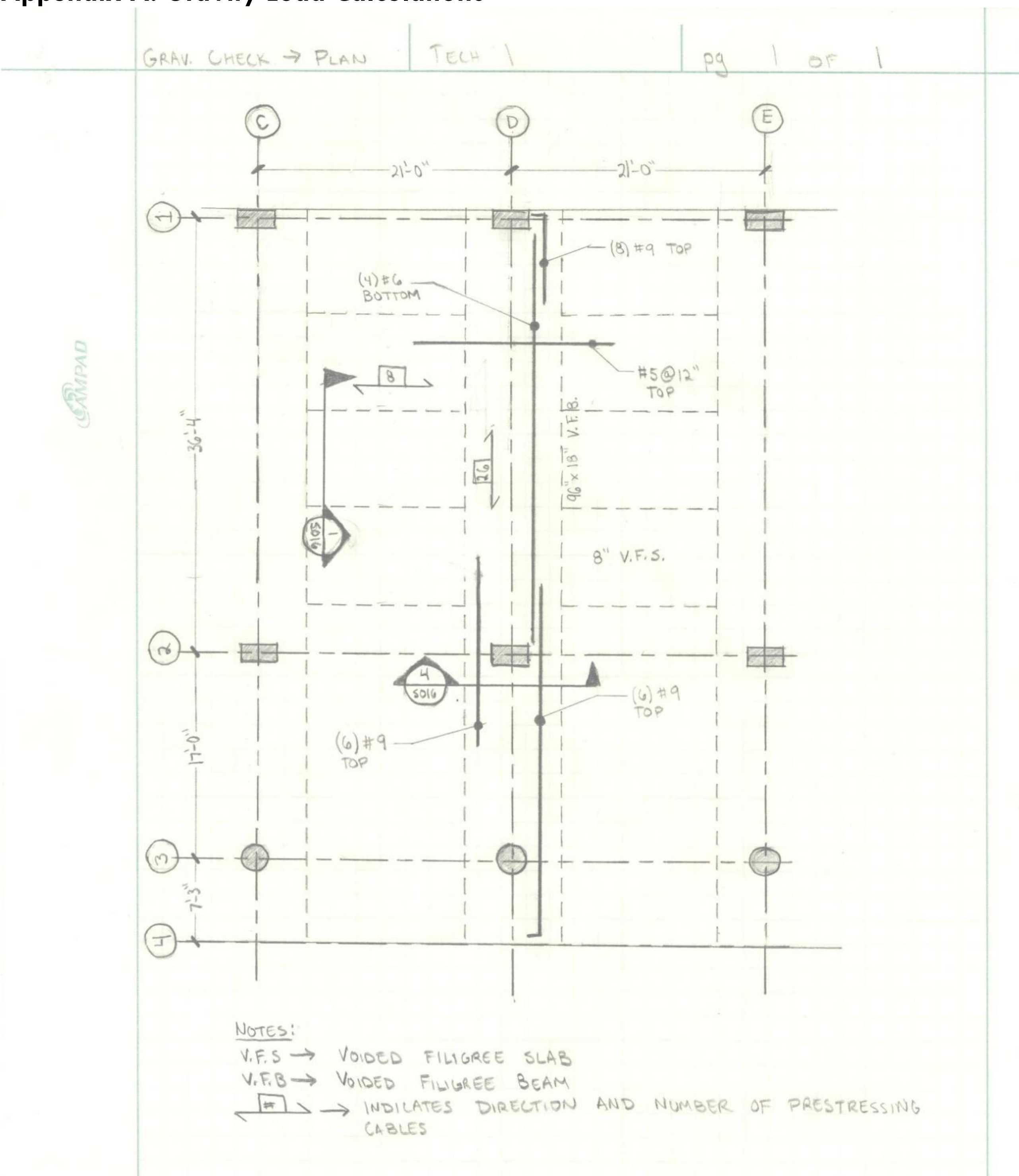
Also included was a determination of gravity and lateral loads. This process relied heavily on information from ASCE 7-05, as well as loads listed in the structural drawings. Superimposed dead loads and live loads were tabulated and checked for practicality. Discrepancies between these loads and the commonly assumed design loads are all easily explainable. Assumptions were also made regarding the percentage of solid concrete of the filigree slabs and beams, which were proved reasonably accurate by the gravity load checks also performed in this technical report. With this information, it was possible to calculate an overall building weight.

Gravity checks were performed on three members in this structure to encompass a representative range of concrete sections used. A typical column, voided filigree slab, and voided filigree beam were all analyzed to verify the adequacy of their design. It was found that each member was satisfactory, and that design gravity loads were able to be replicated within a reasonable margin of error.

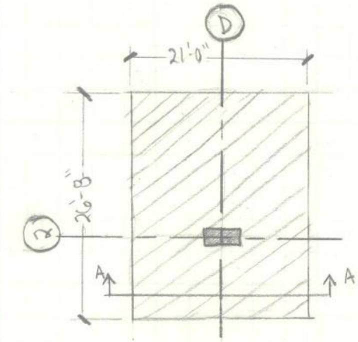
In addition to gravity checks, wind and seismic loads were calculated. Wind loads on this structure were not found to control, and were calculated to match the design loads indicated on the structural drawings within a reasonable margin of error. Seismic loads were approximately twice the wind loads in the East-West direction and 1.7 times larger than the wind loads in the North-South direction, and thus will control the lateral design of this building. This is likely due to the very heavy structure used in the USB. The design seismic loads listed on the structural drawings were also matched within a reasonable margin of error by the calculations contained in this technical report, and were in fact much closer to the design loads than the wind loads.

Appendices

Appendix A: Gravity Load Calculations



GRAV. CHECK → COL. D/2 | TECH 1 | Pg 1 OF 2



① PARTIAL PLAN
NTS

▨ = TRIBUTARY AREA, A_T = 560 SF

ASSUMPTIONS
- VOIDED FILIGREE SLABS (V.F.S) 80% SOLID
- VOIDED FILIGREE BEAMS (V.F.B) 90% SOLID

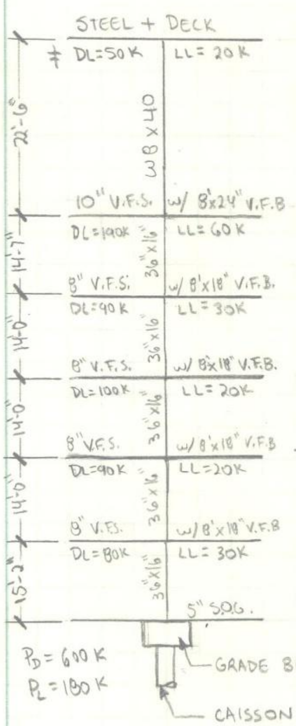
SELF WEIGHTS OF SLABS/BEAMS

10" V.F.S. w/ 8'x24" V.F.B. →

$$\left[\left(\frac{8'}{21'}\right)\left(\frac{24''}{12''}\right)(0.9)+\left(\frac{13'}{21'}\right)\left(\frac{10''}{12''}\right)(0.8)\right]150\text{ pcf} = 165\text{ psf}$$

8" V.F.S. w/ 8'x18" V.F.B →

$$\left[\left(\frac{8'}{21'}\right)\left(\frac{18''}{12''}\right)(0.9)+\left(\frac{13'}{21'}\right)\left(\frac{8''}{12''}\right)(0.8)\right]150\text{ pcf} = 130\text{ psf}$$



ROOF
↳ FRAMING = 15 psf
↳ ROOFING = 20 psf
↳ SDL = 50 psf
↳ SL = 23 psf *

PENTHOUSE
↳ SDL = 90 psf
↳ LL = 100 psf
↳ MECH. EQUIP. = 10 K

5th
↳ SDL = 15 psf
↳ LL = 80 psf

4TH
↳ SDL = 15 psf
↳ LL = 80 psf

3RD
↳ SDL = 15 psf
↳ LL = 80 psf

2ND
↳ SDL = 15 psf
↳ LL = 80 psf

GROUND

② SECTION A-A
NTS

NOTES:
* STRUCTURAL DRAWINGS USE 50 psf
‡ LOADS LISTED ON SECTION ARE SERVICE LOADS FROM THE COLUMN SCHEDULE, SHEET S301

COLUMN LOADS

ROOF
↳ $P_D = 560(15+20+50) = 47.6\text{ K} \rightarrow 50\text{ K}$ ✓
↳ $P_L = 560(23) = 12.9\text{ K} \rightarrow 15\text{ K}$ ✓

PENTHOUSE
↳ $P_D = 560(90+165)+40(22.5')+10{,}000 = 154\text{ K}$
↳ $P_L = 560(100) = 56\text{ K} \rightarrow 60\text{ K}$ ✓

5th
↳ $P_D = 560(15+130)+\left(\frac{36\times16}{144}\right)(150)(14.58') = 90\text{ K}$ ✓
↳ $LL_r = 0.25+\frac{15}{\sqrt{(4)(560)}} = 0.57 \rightarrow 0.6$ OK
$P_L = 560(0.6)(80) = 26.9\text{ K} \rightarrow 30\text{ K}$ ✓

4th
↳ $P_D = 560(15+130)+\left(\frac{36\times16}{144}\right)(150)(14') = 90\text{ K}$
↳ $LL_r = 0.25+\frac{15}{\sqrt{4(2)(560)}} = 0.47 \rightarrow 0.5$ OK
$P_L = 560(0.5)(80) = 22.4\text{ K} \rightarrow 20\text{ K}$ ✓

3rd
↳ $P_D = 90\text{ K}$ (SEE 4th) ✓
↳ $LL_r = 0.25+\frac{15}{\sqrt{4(3)(560)}} = 0.43 \rightarrow 0.4$ OK
$P_L = 560(0.4)(80) = 17.9\text{ K} \rightarrow 20\text{ K}$ ✓

2nd
↳ $P_D = 90\text{ K}$ (SEE 4th)
↳ $P_L = 20\text{ K}$ (SEE 3rd)

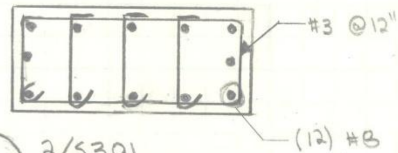
TOTALS
↳ $P_D = 564\text{ K}$
↳ $P_L = 165\text{ K}$
$> P_U = 1.2(564)+1.6(165)$ $\boxed{= 940\text{ K}}$ OK ✓

$P_{U,\text{GIVEN}} = 1.2(600)+1.6(180) = 1010\text{ K}$ (~7% DIFF)

GRAV. CHECK → COL. D/2 | TECH 1 | PG 2 OF 2

CHECK REINFORCING

$A_s = 12(0.79) = 9.48\ in^2$



3 2/S301
NTS

COLUMN IS CHECKED FOR PURE COMPRESSION BECAUSE IT IS AN INTERIOR COLUMN NOT IN A MOMENT FRAME OR NEAR A SHEAR WALL. THEREFORE, COLUMN ECCENTRICITY (e) IS NEGLIGIBLE.

$$\phi P_o = \phi\left(0.85 f'_c A_c + A_s f_y\right) = 0.65\left[0.85(5\ ksi)(16(36") - 9.48\ in^2) + 9.48\ in^2(60\ ksi)\right]$$
$$= 1935\ K$$

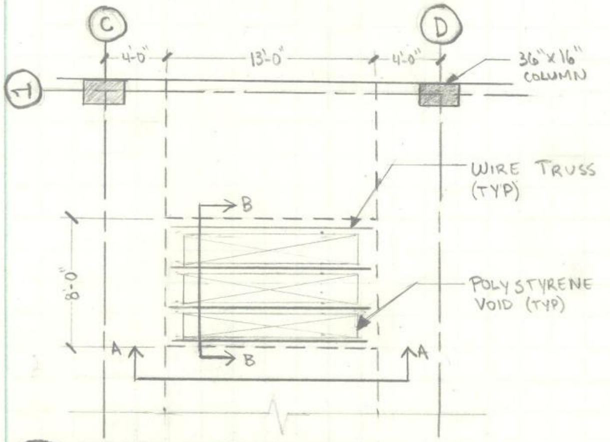
PURE COMPRESSION LIMTED BY α (ACCOUNTS FOR MINIMUM ECCENTRICITY)

$\phi P_n = \alpha \phi P_o = 0.8(1935\ K) = 1548\ K >> P_u$ OK ✓

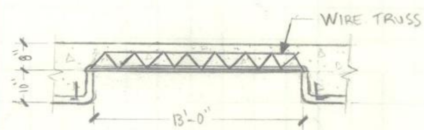
$\rho = \frac{9.48}{36(16)} = 0.016 > 1\%$ OK ✓ (MINIMUM ALLOWABLE ρ, PER ACI 318-08 SECTION 10.9.1)

GRAV. CHECK → V.F.S. | TECH 1 | pg 1 OF 3

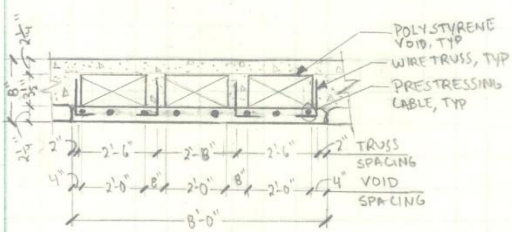
CHECK AN INTERIOR VOIDED FILIGREE SLAB (V.F.S.) PANEL SPANNING BETWEEN COLUMN LINES C & D ON THE 4th LEVEL



1 PARTIAL PLAN
NTS



2 SECTION A-A
NTS



3 SECTION B-B
NTS

ASSUMPTIONS:

- VOIDED FILIGRE SLAB (V.F.S.) IS 80% SOLID
- VOIDED FILIGREE BEAMS (V.F.B.) ARE 90% SOLID

DESIGN MOMENT GIVEN ON STRUCTURAL DRAWINGS → $5.1 \frac{k\text{-}ft}{ft}$

FROM COMPARISON WITH MOMENTS ON SLAB AT ADJACENT BAYS, IT SEEMS AS THOUGH ACI MOMENT COEFFICIENTS WERE USED

SPAN A-B $M_{u+} = 8.0 \frac{k\text{-}ft}{ft} = \frac{w\ell^2}{11}$

SPAN B-C $M_{u+} = 5.5 \frac{k\text{-}ft}{ft} = \frac{w\ell^2}{16}$

$\frac{5.5}{8.0} = 0.6875 = \frac{11}{16}$ OK ✓

SPAN CD $M_u = 5.1 \frac{k\text{-}ft}{ft}$

$\frac{5.1}{8} = \frac{11}{x} \Rightarrow x = 17.25$

$M_{u+} = \frac{w\ell^2}{17}$

LOADS FROM STRUCTURAL DRAWINGS →

SDL = 15 psf

LL = 80 psf

SLAB SELF-WEIGHT → $\left[\left(\frac{8''}{21'}\right)\left(\frac{18''}{12''}\right)(0.9) + \left(\frac{13'}{21'}\right)\left(\frac{8''}{12''}\right)(0.8)\right] 150\text{ pcf} = 130\text{ psf}$

CLEAR LENGTH → $\ell_n = 21' - 2\left(\tfrac{1}{2}\right)\left(\frac{36''}{12''}\right) = 18\text{ ft}$

$W_D = 130 + 15 = 145\text{ plf/ft}$

$W_L = 80\text{ plf/ft}$

$> w_u = 1.2(145) + 1.6(80) = 302\text{ plf/ft} = 0.302\text{ klf/ft}$

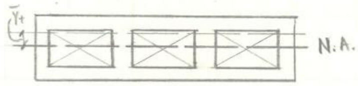
$M_{u+} = \frac{w_u \ell_n^2}{17} = \frac{0.302(18)^2}{17} = 5.76\text{ k-ft/ft} \rightarrow M_{u,serv.} = \frac{5.76\text{ k-ft/ft}}{1.5} = 3.84\text{ k-ft/ft}$

$M_{u-} = \frac{w_u \ell_n^2}{11} = \frac{0.302(18)^2}{11} = -8.9\text{ k-ft/ft}$

$V_u = 0.302(18')\left(\tfrac{1}{2}\right) = 2.72\text{ k/ft}$

GRAV. CHECK → V.F.S. TECH 1 pg 2 of 3

POSITIVE MOMENT CHECK



BOTTOM REINFORCING → (8) 3/8" ∅ P.S. STRANDS

$P_e = 0.085 \text{ in}^2/\text{strand} (8 \text{ strands})(0.6)(270 \text{ ksi}) = 110.2 \text{ K}$

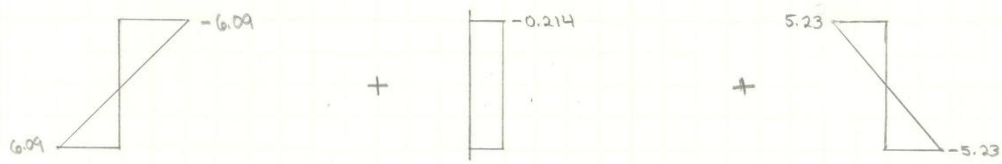
$$\bar{Y}_t = \frac{(4"+4"+8"+8")(1.75")(0.875") + 2.25"(96")(2.875")}{(4"+4"+8"+8")(1.75") + 2.25"(96")} = 2.55 \text{ in}$$

$$I_t = \frac{(4"+4"+8"+8")(1.75")^3}{12} + (4"+4"+8"+8")(1.75")(2.55"-0.875")^2 + \frac{96"(2.25")^3}{12} + 96"(2.25")(2.55"-2.875")^2$$
$$= 242.5 \text{ in}^4$$

$$S_t = \frac{I_t}{\bar{Y}_t} = \frac{242.5 \text{ in}^4}{2.55 \text{ in}} = 60.625 \text{ in}^3 = S_b$$

$$A = [(4"+4"+8"+8")(1.75") + (96")(2.25")]\,2 = 516 \text{ in}^2$$

$$M_{SERV} = 3.84 \tfrac{\text{k-ft}}{\text{ft}} (8 \text{ ft})(12 \text{ in}/\text{ft}) = 369 \text{ k-in}$$



$$\frac{M}{S} = \frac{369 \text{ k-in}}{60.625 \text{ in}^3} = 6.09 \text{ ksi} \quad + \quad \frac{P_e}{A} = \frac{110.2 \text{ k}}{516 \text{ in}^2} = 0.214 \text{ ksi} \quad + \quad \frac{P_e e}{S} = \frac{110.2 \text{ k}(6.875"-4")}{60.625 \text{ in}^3} = 5.23 \text{ ksi}$$

$f_t = -6.09 - 0.214 + 5.23 = -1.074$ ksi $< 0.45 f'_c = 0.45(5) = 2.25$ ksi OK✓

$f_b = 6.09 - 0.214 - 5.23 = 0.646$ ksi $< \underbrace{12\sqrt{f'_c}}_{\text{LIKELY}} = 0.849$ ksi OK✓
LIKELY DESIGNED AS CLASS T

$f_{t,u} = 1.5(-1.074) = -1.611$ ksi

$f_{b,u} = 1.5(0.646) = 0.969$ ksi

$\boxed{\text{OK ✓}}$

NEGATIVE MOMENT CHECK

REINFORCING PROVIDED → #5 @ 12" WITH WWF (FOR SHRINKAGE & TEMP.)

$A_s = 0.31 \text{ in}^2/\text{ft}$

$d = 8 - 0.75 - \frac{1}{2}(5/8") = 6.94 \text{ in}$

$$a = \frac{A_s F_y}{0.85 f'_c b} = \frac{0.31 \text{ in}^2/\text{ft}(60 \text{ ksi})}{0.85(5 \text{ ksi})(12 \text{ in}/\text{ft})} = 0.365 \text{ in}$$ ← IN SOLID PORTION OF SLAB ∴ BEHAVES AS A RECTANGULAR SECT.

$$\phi M_n = \phi A_s F_y \left(d - \frac{a}{2}\right) = 0.9(0.31 \text{ in}^2/\text{ft})(60 \text{ ksi})\left(6.94" - \frac{0.365"}{2}\right) = 9.43 \tfrac{\text{k-ft}}{\text{ft}} > M_u = 8.9 \tfrac{\text{k-ft}}{\text{ft}}$$

$\boxed{\text{OK ✓}}$

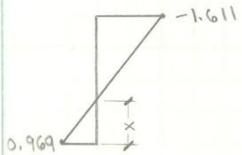
GRAV. CHECK → V.F.S. TECH. 1 PG 3 OF 3

VERTICAL SHEAR CHECK

$\phi V_c = \phi 2\sqrt{f'_c}\, b_w d = 0.75(2)\sqrt{5000}\,(12\ ^{in}/_{ft})(6.94\ in)\left(\frac{1}{1000}\right) = 8.83\ ^{k}/_{ft}$

$\phi V_c > V_u = 2.72\ ^{k}/_{ft}$ $\boxed{OK ✓}$

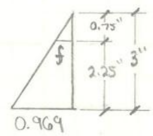
HORIZONTAL SHEAR CHECK



$\frac{0.969}{x} = \frac{1.611}{8-x}$

$0.969(8) - 0.969x = 1.611x$

$x = 3.00\ in$



$\frac{f}{0.75} = \frac{0.969}{3}$

$f = 0.242\ ksi$

$V_{u,h} = \frac{1}{2}(0.242\ ksi)(12\ ^{in}/_{ft})(0.75\ in) = 1.09\ ^{k}/_{ft}$

FROM ACI 318-08 SECT. 17.5.3.1,

$\phi V_{nc,h} = \phi\, 80\, b_v d = 0.75(80)(12\ ^{in}/_{ft})(6.94'')\left(\frac{1}{1000}\right) = 5.0\ ^{k}/_{ft} > V_{u,h}$ $\boxed{OK ✓}$

DEFLECTION

PER ACI 318-08 SECT. 9.5.5.1, COMPOSITE MEMBERS CAN BE CONSIDERED EQUIVALENT TO A MONOLITHICALLY CAST MEMBER DUE TO USE OF SHORING DURING CASTING OF ADDITIONAL CONCRETE.

TRY CALCULATING DEFLECTIONS AS IF SIMPLY SUPPORTED
↳ THIS IS CONSERVATIVE, AND IF IT PASSES DEFLECTION REQUIREMENTS, NO MORE SPECIFIC / COMPLICATED CALCULATIONS WILL BE REQUIRED

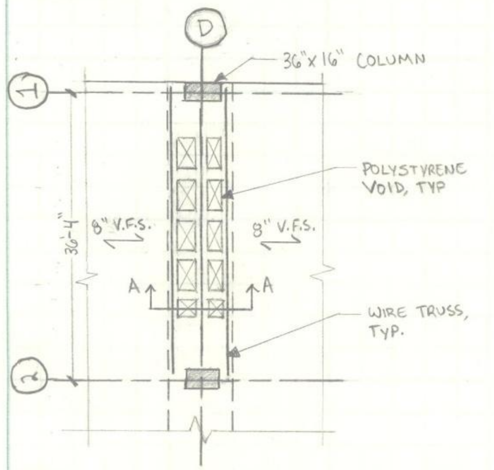
$\Delta_{LL} = \frac{5(1.6)(0.080\ ^{klf}/_{ft})(18')^4(1728)}{384(57\sqrt{5000})[2(242.5)]} = 0.155\ in < \Delta_{LL,max} = \frac{18' \times 12\ ^{in}/_{ft}}{360} = 0.6\ in$ OK ✓

$\Delta_{TL} = \frac{5(0.302\ ^{klf}/_{ft})(18')^4(1728)}{384(57\sqrt{5000})[2(242.5)]} = 0.365\ in < \Delta_{TL,max} = \frac{18' \times 12\ ^{in}/_{ft}}{240} = 0.9\ in$ OK ✓

$\boxed{OK ✓}$

GRAV. CHECK → V.F.B. | TECH 1 | PG 1 OF 5

CHECK 36'-4" BEAM SPANNING BETWEEN COLUMN LINES 1 & 2 ALONG COLUMN LINE D ON THE 4th LEVEL.



1 PARTIAL PLAN
NTS

ASSUMPTIONS:

- VOIDED FILIGREE SLAB (V.F.S.) IS 80% SOLID
- VOIDED FILIGREE BEAM (V.F.B.) IS 90% SOLID

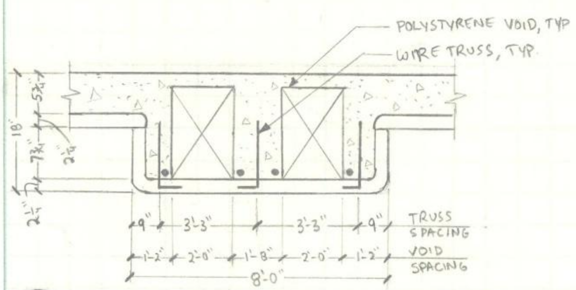
POSITIVE DESIGN MOMENT GIVEN ON STRUCTURAL DRAWINGS → 560 k-ft

IT SEEMS LIKELY THAT ACI MOMENT COEFFICIENTS WOULD BE USED FOR THE BEAMS, SINCE THEY WERE USED FOR THE SLAB. THEREFORE, THIS IS HOW I CHECKED THE BEAM.

LOADS FROM STRUCTURAL DRAWINGS →

SDL = 15 psf

LL = 80 psf



2 SECTION A-A
NTS

SLAB SELF-WEIGHT → $\left[\left(\frac{8'}{21'}\right)\left(\frac{18"}{12"}\right)(0.9) + \left(\frac{13'}{21'}\right)\left(\frac{8"}{12"}\right)(0.8)\right] 150 \text{ pcf} = 130 \text{ psf}$

CLEAR LENGTH → $l_n = 36.33' - 2\left(\frac{1}{2}\right)\left(\frac{16"}{12"}\right) = 35 \text{ ft}$

$LL_r = 0.25 + \frac{15}{\sqrt{2(36.33)(21)}} = 0.63 > 0.5$ OK ✓

$1526 > 400$ OK ✓

$w_D = 21'(130 + 15) = 3045 \text{ plf} = 3.045 \text{ klf}$

$w_L = 21'(0.63)(80) = 1058 \text{ plf} = 1.058 \text{ klf}$

$w_u = 1.2(3.045) + 1.6(1.058) = 5.3468 \text{ klf}$

$M_{u,+} = \frac{w_u l_n^2}{11} = \frac{5.3468(35')^2}{11} = 595 \text{ k-ft} \rightarrow M_{u,\,serv} = \frac{595 \text{ k-ft}}{1.5} = 397 \text{ k-ft}$

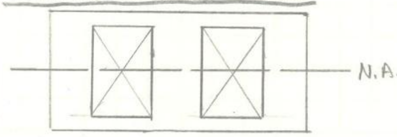
$M_{u,-,1} = \frac{w_u l_n^2}{16} = \frac{5.3468(35)^2}{16} = 409 \text{ k-ft}$

$M_{u,-,2} = \frac{w_u l_n^2}{9} = \frac{5.3468(35)^2}{9} = 728 \text{ k-ft}$

$V_u = 5.3468(35')\left(\frac{1}{2}\right) = 93.6 \text{ k}$

GRAV. CHECK → V.F.B. TECH 1 pg 2 OF 5

POSITIVE MOMENT CHECK



BOTTOM PRESTRESSING → (26) 3/8" ∅ STRANDS

$P_e = 0.085 \frac{in^2}{STRAND} (26 \text{ STRANDS})(0.6)(270 \text{ ksi}) = 358 \text{ k}$

BOTTOM REINFORCING → (4) #6

$A_s = 0.44 (4) = 1.76 \text{ in}^2$ $d = 18" - 2.5" = 15.5"$

$a = \frac{A_s F_y}{0.85 f'_c b} = \frac{1.76 (60)}{0.85 (5)(96)} = 0.259 \text{ in}$

$\phi M_n = \phi A_s F_y (d - \frac{a}{2}) = 0.9 (1.76)(60)(15.5 - \frac{0.259}{2}) = 122 \text{ k-ft}$

$M_u = 397 - 122 = 275 \text{ k-ft} \times 12 \text{ in/ft} = 3300 \text{ k-in}$

$\bar{y}_+ = \frac{(14"+14"+20")(6.75")(3.375") + 96"(2.25")(7.875")}{(14"+14"+20")(6.75") + 96"(2.25")} = 5.175 \text{ in}$

$I_+ = \frac{(14"+14"+20")(6.75")^3}{12} + (14"+14"+20")(6.75")(5.175" - 3.375")^2 + \frac{96"(2.25")^3}{12} + 96"(2.25)(7.875" - 5.175")^2$
$= 3946 \text{ in}^4$

$S_+ = \frac{I}{y} = \frac{3946 \text{ in}^4}{9 \text{ in}} = 438 \text{ in}^3$

$A = [(96")(2.25") + (14"+14"+20")(6.75")] 2 = 1080 \text{ in}^2$

$\frac{M}{S} = \frac{3300 \text{ k in}}{438 \text{ k in}} = 7.53 \text{ ksi}$ + $\frac{P_e}{A} = \frac{358 \text{ k}}{1080 \text{ k}} = 0.331 \text{ ksi}$ + $\frac{P_e e}{S} = \frac{358 \text{ k} (16.875" - 9")}{438 \text{ in}^3} = 6.44 \text{ ksi}$

$f_t = -7.53 - 0.331 + 6.44 = 1.42 \text{ ksi}$ $< 0.45 f'_c = 0.45 (5) = 2.25 \text{ ksi}$ OK ✓

$f_b = 7.53 - 0.331 - 6.44 = 0.759 \text{ ksi}$ $< 12\sqrt{f'_c} = 12\sqrt{5000} = 0.849 \text{ ksi}$ OK ✓

$f_{t,u} = 1.5 (1.42) = 2.13 \text{ ksi}$ $\boxed{\text{OK ✓}}$

$f_{b,u} = 1.5 (0.759) = 1.14 \text{ ksi}$

GRAV. CHECK → V.F.B. | TECH 1 | PG 3 OF 5

NEGATIVE MOMENT CHECK @ 1

REBAR PROVIDED → (8) #9

$A_s = 8(1.0) = 8.0 \text{ in}^2$

$d = 18 - 0.75 - \frac{1}{2}(9/8") = 16.69 \text{ in}$

$a = \frac{A_s F_y}{0.85 f'_c b} = \frac{8(60)}{0.85(5)(96)} = 1.18 \text{ in}$

$\phi M_n = \phi A_s F_y (d - \frac{a}{2}) = 0.9(8 \text{ in}^2)(60 \text{ ksi})(16.69" - \frac{1.18"}{2}) = 579 \text{ k·ft} > M_u = 409 \text{ k·ft}$ $\boxed{\text{OK} ✓}$

NEGATIVE MOMENT CHECK @ 2

REBAR PROVIDED → (12) #9

$A_s = 12(1.0) = 12 \text{ in}^2$

$d = 16.69 \text{ in}$

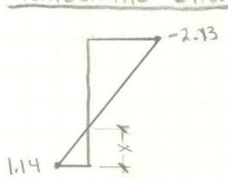
$a = \frac{A_s F_y}{0.85 f'_c b} = \frac{12(60)}{0.85(5)(96)} = 1.76 \text{ in}$

$\phi M_n = \phi A_s F_y (d - \frac{a}{2}) = 0.9(12 \text{ in}^2)(60 \text{ ksi})(16.69" - \frac{1.76"}{2}) = 854 \text{ k·ft} > M_u = 728 \text{ k·ft}$ $\boxed{\text{OK} ✓}$

VERTICAL SHEAR

$\phi V_n = \phi 2\sqrt{f'_c}\, b_w d = 0.75(2)\sqrt{5000}(96")(16.69") = 170 \text{ k} > V_u = 93.6 \text{ k}$ $\boxed{\text{OK} ✓}$

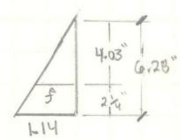
HORIZONTAL SHEAR



$\frac{1.14}{x} = \frac{2.13}{18 - x}$

$1.14(18) - 1.14x = 2.13x$

$x = 6.28 \text{ in}$



$\frac{1.14}{6.28} = \frac{f}{4.03"}$

$f = 0.732 \text{ ksi}$

$V_{u,h} = \frac{1}{2}(0.732 \text{ ksi})(96")(4.03") = 142 \text{ K}$

$\phi V_{nc,h} = \phi 80\, b_w d = 0.75(80)(96")(16.69")(\frac{1}{1000}) = 96.1 \text{ K} < V_{u,h}$ ∴ MUST CONSIDER STEEL

FROM ESR 96-14, A TYPICAL FILIGREE TRUSS IS MADE OF 6mm DIAMETER WIRE AND PENETRATES THE HORIZONTAL SHEAR PLANE EVERY 5". THERE ARE THREE TRUSSES.

GRAV. CHECK → V.F.B. | TECH 1 | pg 4 of 5

$0.6\ cm \times \frac{1\ in}{2.54\ cm} = 0.236\ in$

$A = \frac{\pi}{4}d^2 = \frac{\pi}{4}(0.236\ in)^2 = 0.0438\ in^2$ $\quad A_s = 3(0.0438) = 0.1314\ in^2$

$\rho_v = \frac{A_s}{b_v s} = \frac{0.1314}{96(5)} = 0.000274$

1.0 FOR NWC

$\phi V_{n,h} = \phi(260 + 0.6\rho_v f_y)\lambda b_v d = 0.75[260 + 0.6(0.000274)(60{,}000)](96")(16.69")(\frac{1}{1000})$

$= 324\ k > V_u = 142\ k$ $\boxed{OK ✓}$

DEFLECTION

PER ACI 318-08 SECT. 9.5.5.1, COMPOSITE MEMBERS CAN BE CONSIDERED EQUIVALENT TO A MONOLITHICALLY CAST MEMBER DUE TO USE OF SHORING DURING CASTING OF ADDITIONAL CONCRETE

TRY CALCULATING DEFLECTIONS AS SIMPLY-SUPPORTED
↳ THIS IS CONSERVATIVE, AND IF IT PASSES DEFLECTION REQUIREMENTS, NO MORE SPECIFIC/COMPLICATED CALCULATIONS WILL BE REQUIRED

$\Delta_{LL} = \frac{5(1.6)(1.058\ klf)(35')^4(1728)}{384(57\sqrt{5000})[2(3946\ in^4)]} = 1.80\ in > \Delta_{LL,max} = \frac{35' \times 12}{360} = 1.17 in$

∴ MORE ACCURATE CALCULATION REQUIRED.

$M_{LL@1} = \frac{1.6(1.058)(35)^2}{16} = 130\ k{\cdot}ft$ $\quad M_{LL@2} = \frac{1.6(1.058)(35)^2}{9} = 230.4\ k{\cdot}ft$

FROM AISC TBL 3-23, FIG. 32,

Δ_{max} OCCURS AT $x = \frac{l}{2} + \frac{M_1 - M_2}{wl} = \frac{35}{2} + \frac{130 - 230.4}{1.6928(35)} = 15.81\ ft$

$$\Delta_{LL} = \frac{wx}{24EI}\left[x^3 - \left(2l + \frac{4M_1 - 4M_2}{wl}\right)x^2 + \frac{12M_1}{w}x + l^3 - \frac{8M_1 l}{w} - \frac{4M_2 l}{w}\right]$$

$$= \frac{1.6928(15.81)(1728)}{24(57\sqrt{5000})[2(3946)]}\left[15.81^3 - \left(2(35') + \frac{4(130) - 4(230.4)}{1.6928(35)}\right)15.81^2 + \frac{12(130)(15.81)}{1.6928} + 35^3 - \frac{8(130)(35)}{1.6928} - \frac{4(230.4)(35)}{1.6928}\right]$$

$\Delta_{LL} = 0.225\ in < \Delta_{LL,max}$ OK✓

ASSUME MORE ACCURATE CALCULATION REQUIRED FOR TOTAL LOAD DEFLECTION ALSO

GRAV. CHECK → V.F.B. | TECH 1 | pg 5 OF 5

$M_{TL@1} = 409 \text{ k}\cdot\text{ft}$ $\qquad$ $M_{TL@2} = 728 \text{ k}\cdot\text{ft}$

Δ_{max} OCCURS @ $x = \frac{\ell}{2} + \frac{M_1 - M_2}{w\ell} = \frac{35}{2} + \frac{409-728}{5.3468(35)} = 15.80 \text{ ft}$

$$\Delta_{TL} = \frac{5.3468(15.8)(1728)}{24(57\sqrt{5000'})[2(3946)]}\left[15.8^3 - \left(2(35') + \frac{4(409)-4(728)}{5.3468(35)}\right)15.8^2 + \frac{12(409)(15.8)}{5.3468} + 35^3 - \frac{8(409)(35)}{5.3468} - \frac{4(728)(35)}{5.3468}\right]$$

$\Delta_{TL} = 0.701 \text{ in} \underset{\text{OK} \checkmark}{<} \Delta_{TL,max} = \frac{35' \times 12}{240} = 1.75 \text{ in}$

$\boxed{\text{OK} \checkmark}$

SNOW LOAD CALC'S | TECH 1 | PG 1 OF 2

GROUND SNOW LOAD → FROM FIG. 7-1, $P_g = 30$ psf

TEMPERATURE FACTOR → FROM TBL. 7-3, $C_t = 1.0$
↳ STRUCTURAL DRAWINGS USE $C_t = 0.80$. I AM UNABLE TO EXPLAIN THIS CHOICE, AS THIS IS NOT A PERMISSABLE VALUE BY CODE. I HAVE USED 1.0

EXPOSURE FACTOR → FROM TBL. 7-2, $C_e = 1.0$

IMPORTANCE FACTOR → FROM TBL. 7-4 $I_s = 1.1$

FLAT ROOF SNOW LOAD

$$p_f = 0.7 C_t C_e I_s p_g = 0.7(1.0)(1.0)(1.1)(30) = \boxed{23.1 \text{ psf}}$$

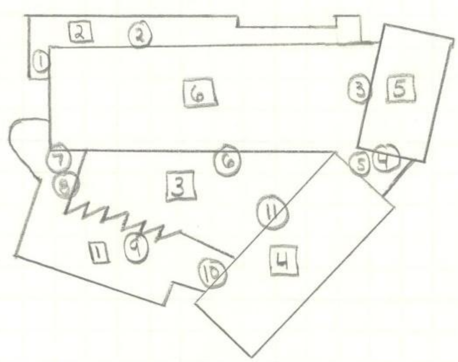
SNOW SPECIFIC GRAVITY

$$\gamma = 0.13 p_g + 14 = 0.13(30) + 14 = 17.9 \text{ pcf} < \gamma_{MAX} = 30 \text{ pcf OK}$$

BASE SNOW ACCUMULATION HEIGHT

$$h_b = p_f/\gamma = \frac{23.1}{17.9} = 1.29 \text{ ft}$$

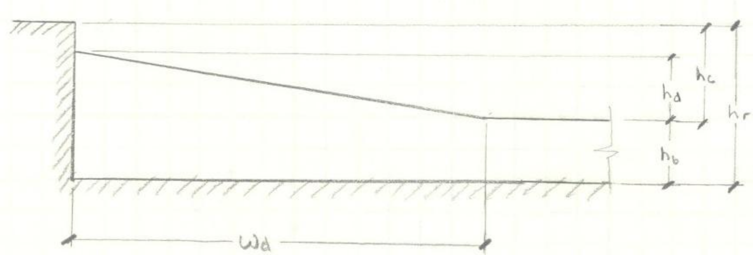
SNOW DRIFTING



SIMPLIFIED ROOF PLAN
[#] ⇒ ROOF NUMBER
(#) ⇒ POTENTIAL DRIFT LOCATION

ROOF HEIGHTS

[1] → 57'-2"
[2] → 71'-9"
[3] → 78'-11"
[4] → 79'-8"
[5] → 85'-9"
[6] → 94'-3"



SNOW LOAD CALC'S | TECH 1 | pg 2 OF 2

LOCATION ① → ROOF $\boxed{6}$ TO ROOF $\boxed{2}$

$h_r = 94'\text{-}3'' - 57'\text{-}2'' = 37'\text{-}1''$

$h_c = h_r - h_B = 37'\text{-}1'' - 1.29' = 35.79$ ft

$\frac{h_c}{h_b} = \frac{35.79}{1.29} = 27.7 > 0.2$ ∴ MUST CALCULATE DRIFT

WINDWARD

$l_u = 6$ ft < 25 ft ∴ USE 25'

$h_d = 0.75\left(0.43\sqrt[3]{l_u}\sqrt[4]{p_g+10} - 1.5\right)$
$= 0.75\left(0.43\sqrt[3]{25}\sqrt[4]{30+10} - 1.5\right) = 1.25$ ft $< h_c$ OK ✓

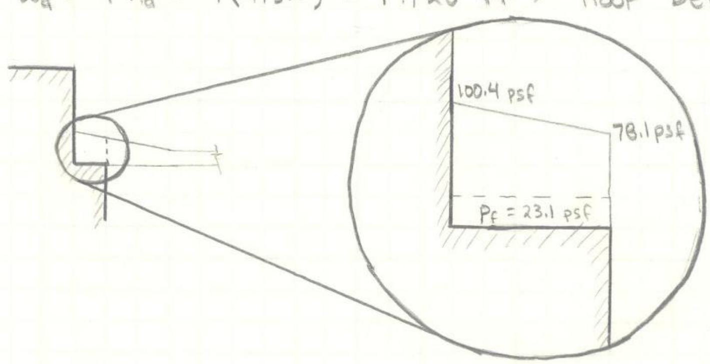
$w_d = 4h_d = 4(1.25) = 5$ ft

LEEWARD → CONTROLS OVER WINDWARD

$l_u = 156$ ft > 25 ft OK ✓

$h_d = 0.43\sqrt[3]{l_u}\sqrt[4]{p_g+10} - 1.5$
$= 0.43\sqrt[3]{156}\sqrt[4]{30+10} - 1.5 = 4.32$ ft $< h_c$ OK ✓

$w_d = 4h_d = 4(4.32) = 17.28$ ft > ROOF DEPTH ∴ ADJUST LOAD



$p_d = 4.32$ ft $(17.9$ pcf$) = 77.3$ psf

TOTAL SNOW LOAD → 77.3 psf + 23.1 psf = 100.4 psf

SNOW LOAD @ END OF ROOF → $100.4 - \left(\frac{5}{17.28}\right)77.3 = 78.1$ psf

ADDITIONAL LOCATIONS CALCULATED WITH SPREADSHEETS

NOTE: LOCATION ⑪ NOT REQUIRED (ROOF $\boxed{4}$ SLOPES DOWN TO ROOF $\boxed{3}$)

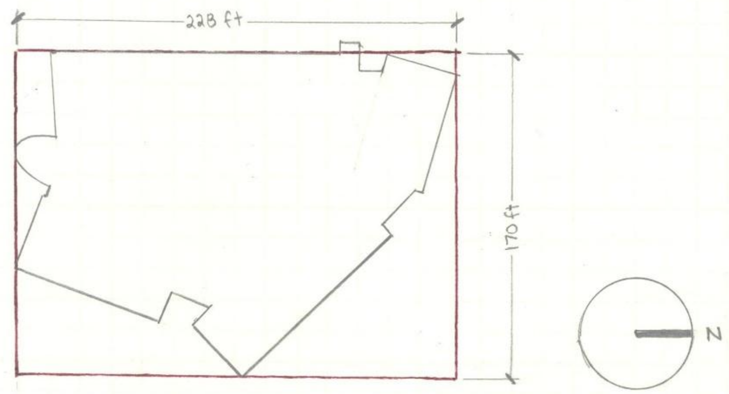
Snow Drift Load Calculations						
Location Number	Windward			Leeward		
	L_u (ft)	p_d (psf)	w_d (ft)	L_u (ft)	p_d (psf)	w_d (ft)
1	6.0	22	5.0	150.0	77	17.3
2	16.0	22	5.0	51.0	45	10.0
3	36.5	28	6.3	156.0	77	17.3
4	63.0	38	8.4	25.0	30	6.6
5	105.0	48	10.8	25.0	30	6.6
6	48.0	33	7.3	51.0	45	10.0
7	68.0	39	8.7	51.0	45	10.0
8	19.5	22	5.0	95.0	61	13.7
9	37.0	28	6.3	48.0	43	9.7
10	97.0	47	10.4	36.5	37	8.3

Note: Pressures are in addition to flat roof snow load (i.e. total snow load at Location 1=100.1 psf)

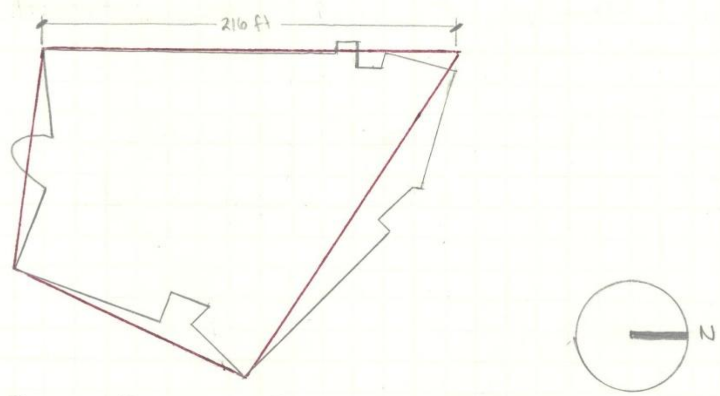
Appendix B: Wind Load Calculations

WIND ANALYSIS | TECH 1 | pg 1 OF 4

SIMPLIFYING ASSUMPTIONS



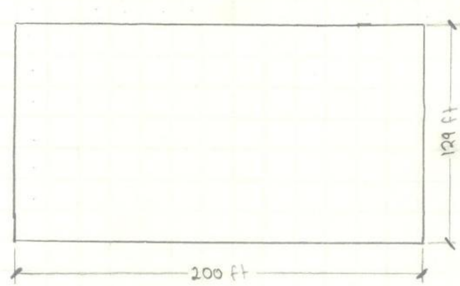
PROJECTED BUILDING DIMENSIONS FOR CALCULATING WIND FORCES



PSEUDO FOOTPRINT FOR CALCULATION OF L & B

↳ AREA = 25,750 SF

↳ 200 ft CHOSEN AS REPRESENTATIVE DIMENSION IN THE LONG DIRECTION, SHORT DIMENSION CALCULATED TO PRESERVE PSEDO-AREA



N-S DIRECTION WIND

L = 200 ft
B = 129 ft

E-W DIRECTION WIND

L = 200 ft
B = 129 ft

REPRESENTATIVE RECTANGLE SHOWING B & L

ROOF HEIGHT → THE BUILDING HAS SIX ROOF HEIGHTS. TO SIMPLIFY THE CALCULATIONS, THE BUILDING IS REPRESENTED AS IF ONLY THE HIGHEST ROOF HEIGHT (94'-3") EXISTS.

WIND ANALYSIS | TECH 1 | PG 2 OF 4

USE METHOD 2 SINCE BUILDING (WITH SIMPLIFYING ASSUMPTIONS) MEETS CRITERIA OF 6.5.1 & 6.5.2

BASIC WIND SPEED → USING FIG. 6-1C, $V = 90$ mph

WIND DIRECTIONALITY FACTOR → USING TBL. 6-4, $K_d = 0.85$

OCCUPANCY CATEGORY → USING TBL 1-1, III
↳ COLLEGE FACILITY WITH MORE THAN 500 PERSON CAPACITY

IMPORTANCE FACTOR → USING TBL 6-1, $I = 1.15$

EXPOSURE CATEGORY → USING SECTION 6.5.6.3, B
↳ DUE TO URBAN SURROUNDINGS

TOPOGRAPHIC FACTOR → FROM SECTION 6.5.7.1, $K_{zt} = 1.0$

VELOCITY PRESSURE COEFFICIENTS → FROM TBL 6-3, VARIES
↳ SEE EXCEL SPREADSHEET

VELOCITY PRESSURES →

$$q_z = 0.00256\ K_z K_{zt} K_d V^2 I$$
$$q_h = 0.00256\ K_h K_{zt} K_d V^2 I$$

> SEE EXCEL SPREADSHEET

GUST EFFECT FACTOR

$n_1 = \frac{75}{94} = 0.798$ (LOWER BOUND FROM C6-17)

$n_1 = \frac{100}{94} = 1.064$ (AVERAGE VALUE FROM C6-18) ✱ USE FOR CALCS

BOTH VALUES ARE CLOSE TO 1.0 Hz, SO CALCULATE G_f IN THE EVENT THE BUILDING IS FLEXIBLE.

$$g_Q = g_v = 3.4$$

$$g_R = \sqrt{2\ln(3600(1.064))} + \frac{0.577}{\sqrt{2\ln(3600(1.064))}} = 4.204$$

$$\bar{z} = 0.6h = 0.6(94) = 56.4 \text{ ft} > \bar{z}_{min} = 30 \text{ ft} \quad \text{OK}$$

FROM TBL. 6-2, $\bar{\alpha} = 1/4.0$, $\bar{b} = 0.45$, $c = 0.30$, $\ell = 320$ ft, & $\bar{\epsilon} = 1/3.0$

$$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6} = 0.3\left(\frac{33}{56.4}\right)^{1/6} = 0.274$$

$$L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}} = 320\left(\frac{56.4}{33}\right)^{1/3} = 382.59$$

$$\bar{V}_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}} V\left(\frac{88}{60}\right) = 0.45\left(\frac{56.4}{33}\right)^{1/4}(90)\left(\frac{88}{60}\right) = 67.92$$

$$N_1 = \frac{n_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}} = \frac{1.064(382.59)}{67.92} = 5.99$$

WIND ANALYSIS TECH 1 pg 3 OF 4

$R_n = \frac{7.47\ N_1}{(1 + 10.3 N_1)^{5/3}} = \frac{7.47(5.99)}{(1+10.3(5.99))^{5/3}} = 0.045$

$\beta = 0.010$ (ASSUMED, CONSERVATIVE FOR CONCRETE SHEAR WALLS)

NORTH-SOUTH

$h = 94$ ft
$L = 200$ ft
$B = 129$ ft

$\eta_h = 4.6 n_1 h/\bar{V}_{\bar{z}} = 4.6(1.064)\,94/67.92$
$= 6.77$

$R_h = \frac{1}{\eta} - \frac{1}{2\eta^2}(1-e^{-2\eta})$
$= \frac{1}{6.77} - \frac{1}{2(6.77)^2}(1-e^{-2(6.77)}) = 0.137$

$\eta_B = 4.6 n_1 B/\bar{V}_{\bar{z}} = 4.6(1.064)\left(\frac{129}{67.92}\right)$
$= 9.296$

$R_B = \frac{1}{\eta} - \frac{1}{2\eta^2}(1-e^{-2\eta})$
$= \frac{1}{9.296} - \frac{1}{2(9.296)^2}(1-e^{-2(9.296)}) = 0.102$

$\eta_L = 15.4 n_1 L/\bar{V}_{\bar{z}} = 15.4(1.064)\left(\frac{200}{67.92}\right)$
$= 48.252$

$R_L = \frac{1}{\eta} - \frac{1}{2\eta^2}(1-e^{-2\eta})$
$= \frac{1}{48.252} - \frac{1}{2(48.252)^2}(1-e^{-2(48.252)}) = 0.021$

$R = \sqrt{\frac{1}{\beta} R_n R_h R_B (0.53 + 0.47 R_L)}$
$= \sqrt{\frac{1}{0.01}(0.045)(0.137)(0.102)[0.53+0.47(0.021)]}$
$= 0.184$

$Q = \sqrt{\frac{1}{1+0.63\left(\frac{129+94}{382.59}\right)^{0.63}}} = 0.831$

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

$= 0.925\left(\frac{1+1.7(0.274)\sqrt{3.4^2(0.831)^2+4.204^2(0.184)^2}}{1+1.7(3.4)(0.274)}\right)$

$= \boxed{0.846}$

EAST-WEST

$h = 94$ ft
$L = 129$ ft
$B = 200$ ft

$\eta_h = 6.77$ (SEE N-S DIRECTION)

$R_h = 0.137$ (SEE N-S DIRECTION)

$\eta_B = 4.6(1.064)\left(\frac{200}{67.92}\right)$
$= 14.413$

$R_B = \frac{1}{14.413} - \frac{1}{2(14.413)^2}(1-e^{-2(14.413)})$
$= 0.067$

$\eta_L = 15.4(1.064)\left(\frac{129}{67.92}\right)$
$= 31.123$

$R_L = \frac{1}{31.123} - \frac{1}{2(31.123)^2}(1-e^{-2(31.123)})$
$= 0.032$

$R = \sqrt{\frac{1}{0.01}(0.045)(0.137)(0.067)[0.57+0.43(0.032)]}$
$= 0.150$

$Q = \sqrt{\frac{1}{1+0.63\left(\frac{200+94}{382.59}\right)^{0.63}}} = 0.808$

$G_f = 0.925\left(\frac{1+1.7(0.274)\sqrt{3.4^2(0.808)^2+4.204^2(0.150)^2}}{1+1.7(3.4)(0.274)}\right)$

$= \boxed{0.828}$

WIND ANALYSIS | TECH 1 | PG 4 OF 4

BUILDING IS FULLY ENCLOSED

SOME AREAS HAVE A PARAPET → WILL BE DISREGARDED DUE TO ASSUMPTION OF UNIFORM ROOF HEIGHT

$p_p = q_p(GC_p - GC_{pi})$

WHERE q_p EQUALS q_z AT THE HEIGHT OF THE PARAPET

EXTERNAL PRESSURE COEFFICIENTS → FROM FIG. 6-6

WALLS → WINDWARD $C_p = 0.8$
LEEWARD C_p ⇒ INTERPOLATE BASED ON L/B VALUES
SIDE $C_p = -0.7$

ROOF → $\theta = 0°$
INTERPOLATE C_p's FOR h/L VALUES

$\frac{h}{2} = \frac{94}{2} = 47$ ft
$h = 94$ ft
$2h = 188$ ft

ROOF AREA → $216 \times 119 = 25{,}704$ ft^2 > 1000 SF
↳ REDUCTION FACTOR = 0.8

INTERNAL PRESSURE COEFFICIENTS → FROM FIG. 6-5, $GC_{pi} = \pm 0.18$

DESIGN WIND PRESSURES

WINDWARD WALLS → $p_z = q_z G_f C_p - q_h(GC_{pi})$

LEEWARD WALLS
SIDE WALLS
ROOF
→ $p_h = q_h(G_f C_p - GC_{pi})$

PARAPET → $p_p = q_p(G_f C_p - GC_{pi})$

General Wind Load Design Criteria		
Design Wind Speed	90 mph	ASCE 7-05, Fig. 6-1C
Directionality Factor (K_d)	0.85	ASCE 7-05, Fig. 6-4
Importance Factor (I_w)	1.15	ASCE 7-05, Tbl. 6-1
Exposure Category	B	ASCE 7-05, Sect. 6.5.6.3
Topographic Factor (K_{zt})	1.0	ASCE 7-05, Sect. 6.5.7.1
Internal Pressure Coefficient (GC_{pi})	0.18	ASCE 7-05, Fig. 6-5

Velocity Pressure Coefficients (K_z) and Velocity Pressures (q_z)			
Level	Elevation (ft)	K_z	q_z
Ground	0.00	0.570	11.55
2nd	15.17	0.572	11.59
3rd	29.17	0.693	14.05
4th	43.17	0.776	15.73
5th	57.17	0.839	17.00
Penthouse	71.75	0.897	18.18
Roof	94.25	0.972	19.70

Building Dimensions		
*	N-S Wind	E-W Wind
B (ft)	129	200
L (ft)	200	129
h (ft)	94	94
W (ft)	170	228

*B= normal to wind direction

L= parallel to wind direction

h= mean roof height

W= Length of face used to calculate wind pressures

Gust Effect Factor (G_f)		
Variable	N-S Wind	E-W Wind
n_1	1.064	
g_Q	3.4	
g_V	3.4	
g_R	4.204	
z_{mean}	56.4	
$I_{z,mean}$	0.274	
$L_{z,mean}$	382.594	
$V_{z,mean}$	67.917	
N_1	5.994	
R_n	0.0452	
β	0.010	
η_h	6.774	
R_h	0.1367	
η_B	9.2963	14.4129
R_B	0.1018	0.0670
η_L	48.2519	31.1225
R_L	0.0205	0.0316
R	0.1842	0.1502
Q	0.8309	0.8075
G_f	0.846	0.828

External Pressure Coefficients (C_p)		
Description	N-S Wind	E-W Wind
L/B	1.550	0.645
Windward Walls	0.8	
Leeward Walls	-0.390	-0.5
Side Walls	-0.7	
h/L	0.470	0.729
Roof - 0 to h/2	-0.9	-1.083
Roof - h/2 to h	-0.9	-0.809
Roof - h to 2h	-0.5	-0.591
Roof - >2h	-0.3	N/A

Appendix C: Seismic Load Calculations

SEISMIC ANALYSIS | TECH 1 | Pg 1 OF 3

SITE CLASS → GIVEN IN THE GEOTECHNICAL REPORT, D

OCCUPANCY CATEGORY → FROM TBL. 1-1, III

IMPORTANCE FACTOR → FROM TBL. 11.5-1, $I_e = 1.25$

SHORT SPECTRAL RESPONSE ACCELERATION → FROM FIG. 22-1, $S_s = 0.28$
1-SEC. SPECTRAL RESPONSE ACCELERATION → FROM FIG. 22-2, $S_1 = 0.06$

SITE COEFFICIENT → FROM TBL. 11.4-1, $F_a = 1.6$
SITE COEFFICIENT → FROM TBL. 11.4-2, $F_v = 2.4$

MODIFIED SHORT S.R.A. → $S_{MS} = F_a S_s = 1.6(0.28) = 0.448$
MODIFIED 1-SEC. S.R.A. → $S_{M1} = F_v S_1 = 2.4(0.06) = 0.144$

DESIGN SHORT S.R.A → $S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3}(0.448) = 0.298$
↳ FROM TBL. 11.6-1, SEISMIC DESIGN CATEGORY B
DESIGN 1-SEC. S.R.A. → $S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3}(0.144) = 0.096$
↳ FROM TBL. 11.6-2, SEISMIC DESIGN CATEGORY B

SEISMIC DESIGN CATEGORY → B

RESPONSE MODIFICATION COEFFICIENT → FROM TBL. 12.2-1, R = 5
↳ ORDINARY REINFORCED CONCRETE SHEAR WALLS

EQUIVALENT LATERAL FORCE (ELF) ANALYSIS USED

APPROXIMATE FUNDAMENTAL PERIOD

$T_a = C_t h_n^x$ FROM TBL. 12.8-2, "OTHER STRUCTURES", $C_t = 0.02$, $x = 0.75$

$T_a = 0.02(94.25)^{0.75} = 0.604$ s

SHEAR WALL EQUATION → $T_a = \frac{0.0019}{\sqrt{C_w}} h_n$

$$C_w = \frac{100}{A_B} \sum_{i=1}^{x} \left(\frac{h_n}{h_i}\right)^2 \frac{A_i}{\left[1 + 0.83\left(\frac{h_i}{D_i}\right)^2\right]}$$

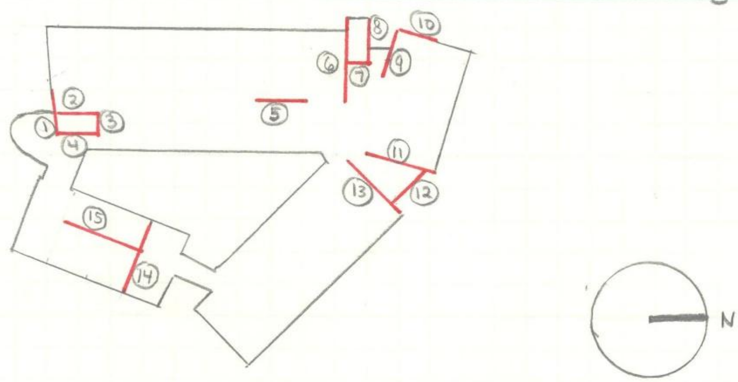
A_B → AREA OF BASE OF STRUCTURE
A_i → WEB AREA OF SHEAR WALL "i" IN FT^2
D_i → LENGTH OF SHEAR WALL "i" IN FT
h_i → HEIGHT OF SHEAR WALL "i" IN FT

SIMPLIFYING ASSUMPTION → RESOLVE LENGTHS OF SHEAR WALLS ONTO N-S & E-W AXES USING TRIG, CALCULATE $A_i = D_i t_i$ (t_i = THICKNESS OF WALL = 12" FOR ALL WALLS)

CALCULATION DONE IN SPREADSHEET

SEE NEXT PAGE FOR SHEAR WALL DIAGRAM / NUMBERING

SEISMIC ANALYSIS TECH 1 Pg 2 OF 3



SHEAR WALL LOCATIONS & NUMBERS

PER SECT. 12.8.2, IT IS PERMITTED TO MULTIPLY T_a BY A COEFFICIENT FOR UPPER LIMIT ON CALCULATED PERIOD, C_u

FROM TBL 12.8-1, FOR $S_{D1} \leq 0.1$, $C_u = 1.7$

C_uT_a's USED TO CALCULATE SEISMIC RESPONSE COEFFICIENTS (C_s)

$$C_{s,calc} = \frac{S_{DS}}{(R/I_e)}$$

$$C_{s,max} = \begin{cases} \dfrac{S_{D1}}{T(R/I)} & \text{FOR } T \leq T_L \\[2ex] \dfrac{T_L S_{D1}}{T^2(R/I)} & \text{FOR } T > T_L \end{cases}$$

$$C_{s,min} = 0.01$$

CALCULATION DONE VIA SPREADSHEET

THE BASIC SOLUTION IS CONTROLLED BY $C_{s,max}$ FOR BOTH DIRECTIONS. THIS WILL RESULT IN IDENTICAL BASE SHEARS IN BOTH DIRECTIONS, WHICH DIFFERS FROM THE STRUCTURAL DRAWINGS.

THE SPECIFIC SOLUTION IS CONTROLLED BY $C_{s,max}$ IN BOTH N-S & E-W DIRECTIONS, BUT THE TWO VALUES ARE DIFFERENT. IT WILL PRODUCE A HIGHER BASE SHEAR IN THE E-W DIRECTION, WHICH WILL MIMIC THE CONDITION NOTED IN THE STRUCTURAL DRAWINGS. BOTH C_s VALUES FROM THE SPECIFIC SOLUTION EXCEED THE C_s VALUE FROM THE BASIC SOLUTION.

USE THE SPECIFIC SOLUTION C_s VALUES TO BE CONSERVATIVE AND BECAUSE THEY SEEM MORE REALISTIC.

SEISMIC ANALYSIS | TECH 1 | pg 3 OF 3

BASE SHEAR

$V = C_s W$

W = WEIGHT OF BUILDING (CALCULATED IN A SPREADSHEET)
= 30, 482 K

$C_{s,N\text{-}S} = 0.0308$

$C_{s,E\text{-}W} = 0.0359$

$V_{N\text{-}S} = 0.0308\,(30{,}482) = 939\text{ K}$
↳ 955 K IN STRUCTURAL DRAWINGS → ~1.7% LOW OK ✓

$V_{E\text{-}W} = 0.359\,(30{,}482) = 1{,}095\text{ K}$
↳ 1145 K IN STRUCTURAL DRAWINGS → ~ 4.4% LOW OK ✓

STORY FORCES

BASE SHEAR IS DISTRIBUTED TO EACH LEVEL BY THE EQUATION:

$F_x = C_{vx} V$

WHERE $C_{vx} = \dfrac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$ (VERTICAL DISTRIBUTION FACTOR)

w = WEIGHT OF STORY
h = HEIGHT OF STORY ABOVE GROUND
$k = 1 + \dfrac{T - 0.5}{2}$ $(1 \le k \le 2)$

$K_{N\text{-}S} = 1 + \dfrac{0.7792 - 0.5}{2} = 1.1396$

$K_{E\text{-}W} = 1 + \dfrac{0.6684 - 0.5}{2} = 1.0842$

NOTE: USED $T = C_u T_a$

CALCULATION COMPLETED BY SPREADSHEET

General Seismic Design Criteria		
Site Class	D	Geotechnical Report
Importance Factor (I_E)	1.25	ASCE 7-05, Tbl. 11.5-1
Short Spectral Response Acceleration (S_s)	0.28	ASCE 7-05, Fig. 22-1
1-sec. Spectral Response Acceleration (S_1)	0.06	ASCE 7-05, Fig. 22-2
Site Coefficient (F_a)	1.6	ASCE 7-05, Tbl. 11.4-1
Site Coefficient (F_v)	2.4	ASCE 7-05, Tbl. 11.4-2
Response Modification Coefficient (R)	5	ASCE 7-05, Tbl. 12.2-1
Long-Period Transition Period	6 s	ASCE 7-05, Fig. 22-15

Seismic Design Parameters	
Description	Value
Modified Short S.R.A. (S_{MS})	0.448
Modified 1-sec. S.R.A (S_{M1})	0.144
Design Short S.R.A. (S_{DS})	0.2987
Design 1-sec. S.R.A. (S_{D1})	0.0960
Seismic Design Category	B

Shear Wall Data							
Shear Wall Number	Length (ft)	Angle with NS-axis (deg)	Height (ft)	Length in NS-Dir. (ft)	Area in NS-Dir. (ft^2)	Length in EW-Dir. (ft)	Area in EW-Dir. (ft^2)
1	40	95	94.25	3.49	3.49	39.85	39.85
2	20	0	94.25	20.00	20.00	0.00	0.00
3	8	90	94.25	0.00	0.00	8.00	8.00
4	20	0	94.25	20.00	20.00	0.00	0.00
5	18	0	71.75	18.00	18.00	0.00	0.00
6	48	90	104.25	0.00	0.00	48.00	48.00
7	8	0	104.25	8.00	8.00	0.00	0.00
8	24	90	104.25	0.00	0.00	24.00	24.00
9	18	-105	71.75	4.66	4.66	17.39	17.39
10	13	-15	71.75	12.56	12.56	3.36	3.36
11	30	-15	94.25	28.98	28.98	7.76	7.76
12	25	45	94.25	17.68	17.68	17.68	17.68
13	34	-45	85.75	24.04	24.04	24.04	24.04
14	38	-110	57.17	13.00	13.00	35.71	35.71
15	35	-20	57.17	32.89	32.89	11.97	11.97

Note: "Areas" are web areas, A="Length of Wall"x"Thickness of Wall". All shear walls are 1'-0" thick

Seismic Response Coefficient (C_S)				
	N-S Direction		E-W Direction	
	Basic	Specific	Basic	Specific
C_t	0.02	N/A	0.02	N/A
x	0.75	N/A	0.75	N/A
A_B (ft^2)	N/A	25,460	N/A	25,460
C_W	N/A	0.15	N/A	0.21
h_n (ft)	94.25			
T_a (s)	0.6050	0.4583	0.6050	0.3932
C_U	1.7			
C_UT_a	1.0285	0.7792	1.0285	0.6684
$C_{S,CALC}$	0.0747			
$C_{S,MAX}$	0.0233	0.0308	0.0233	0.0359
$C_{S,MIN}$	0.01			
C_S	0.0233	0.0308	0.0233	0.0359

Appendix D: Typical Plans

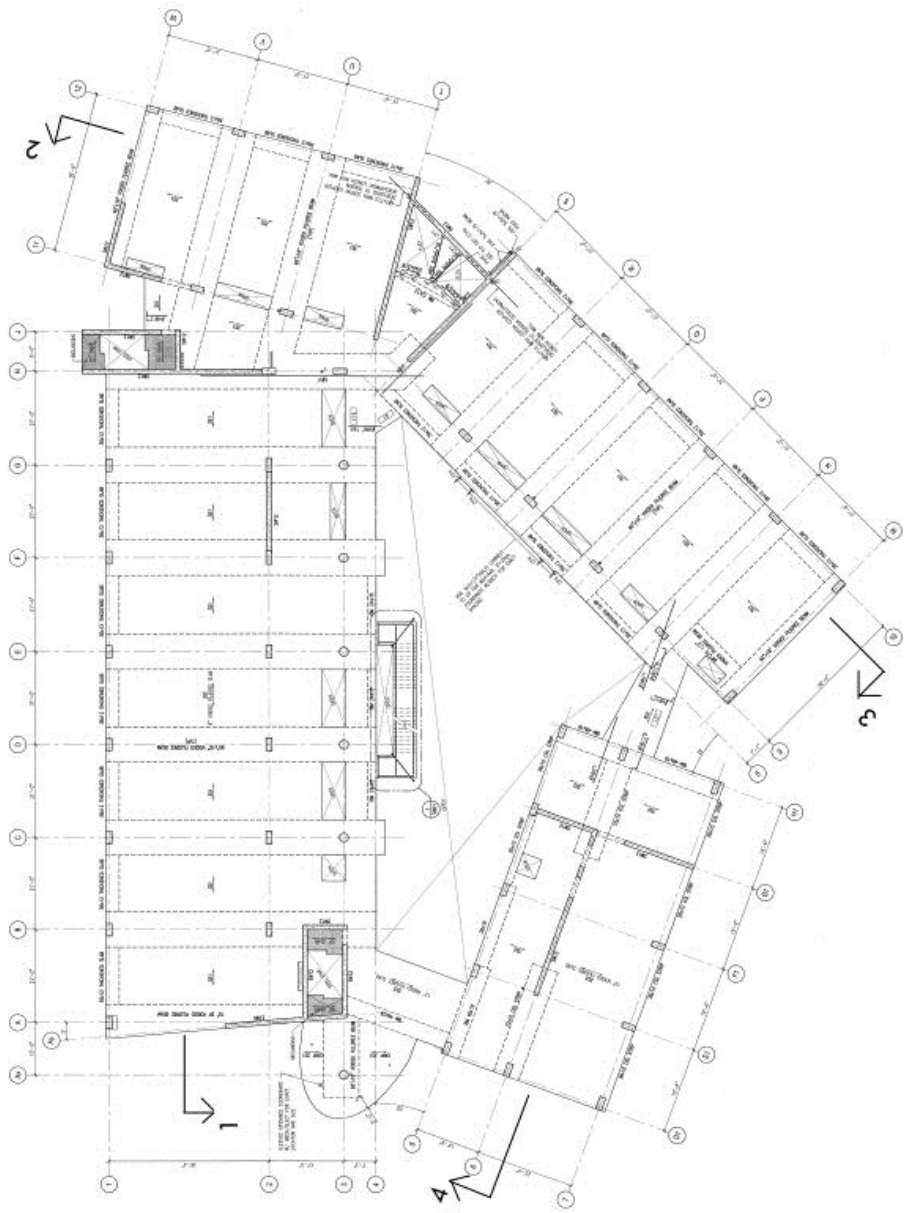


Figure 1 Typical Floor plan, taken from S202. See following figures for sections indicated on the plan.

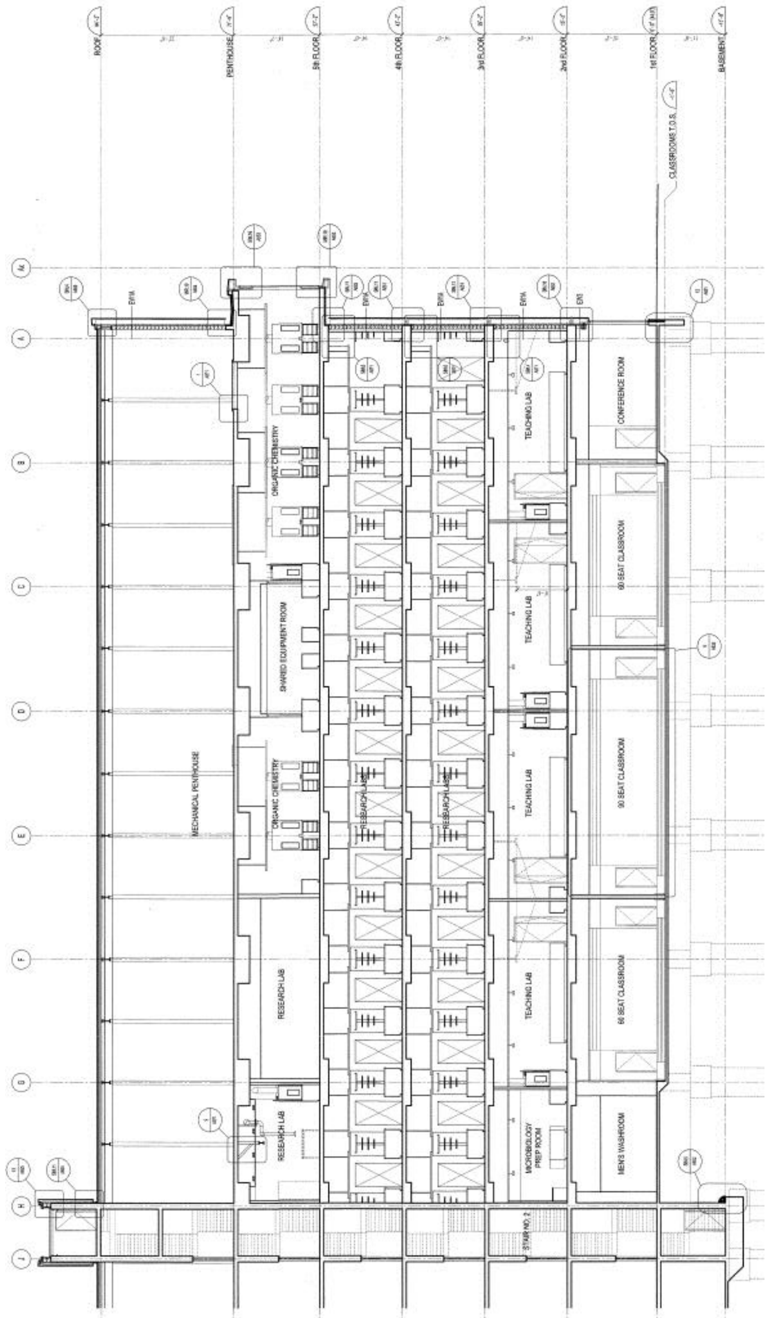


Figure 2 Section 1 through portion of building at 0° rotation (see Figure 1), taken from 3/A401.

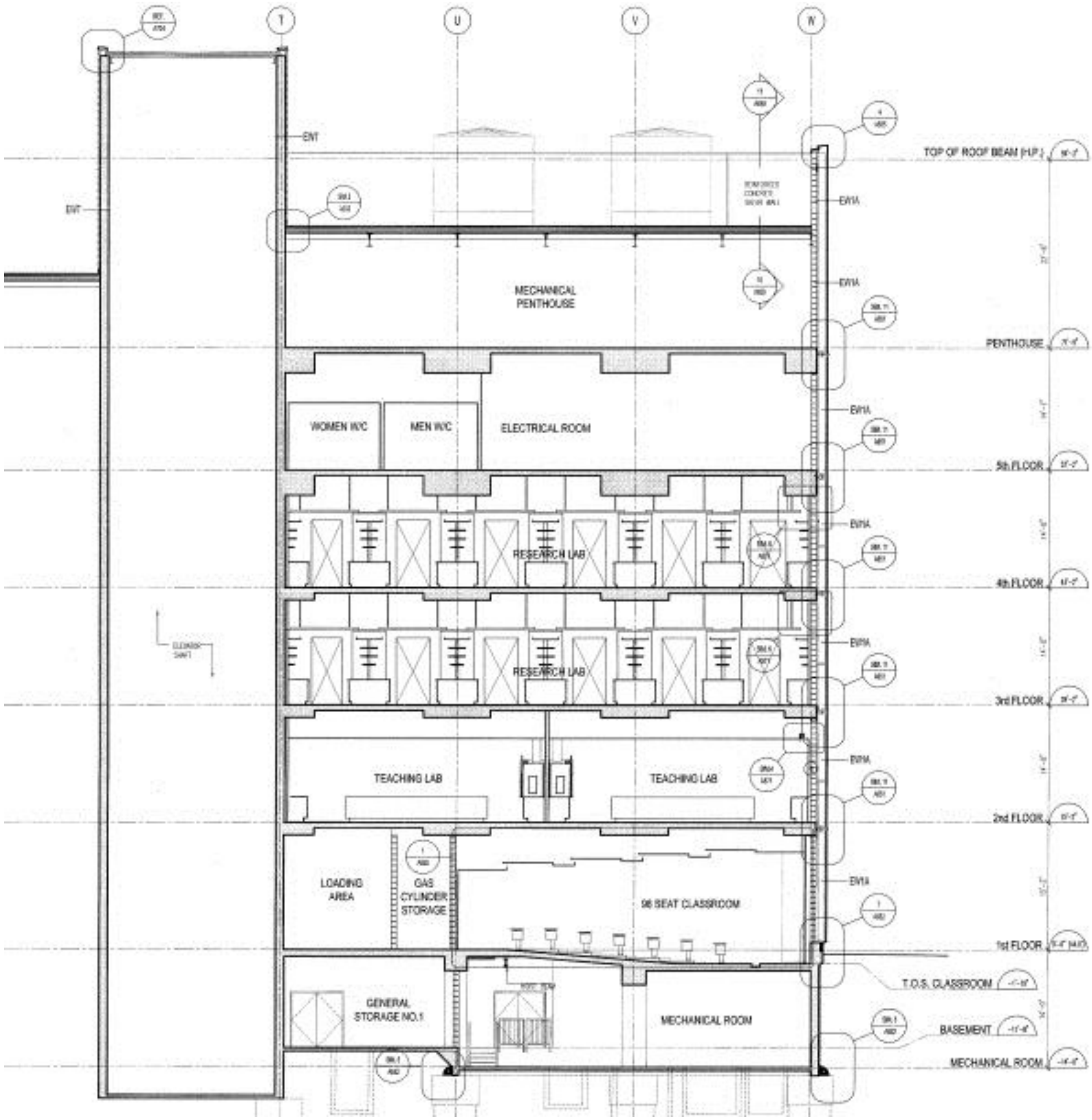


Figure 3 Section 2 through portion of building at -15° rotation (see Figure 1), taken from 2/A402.

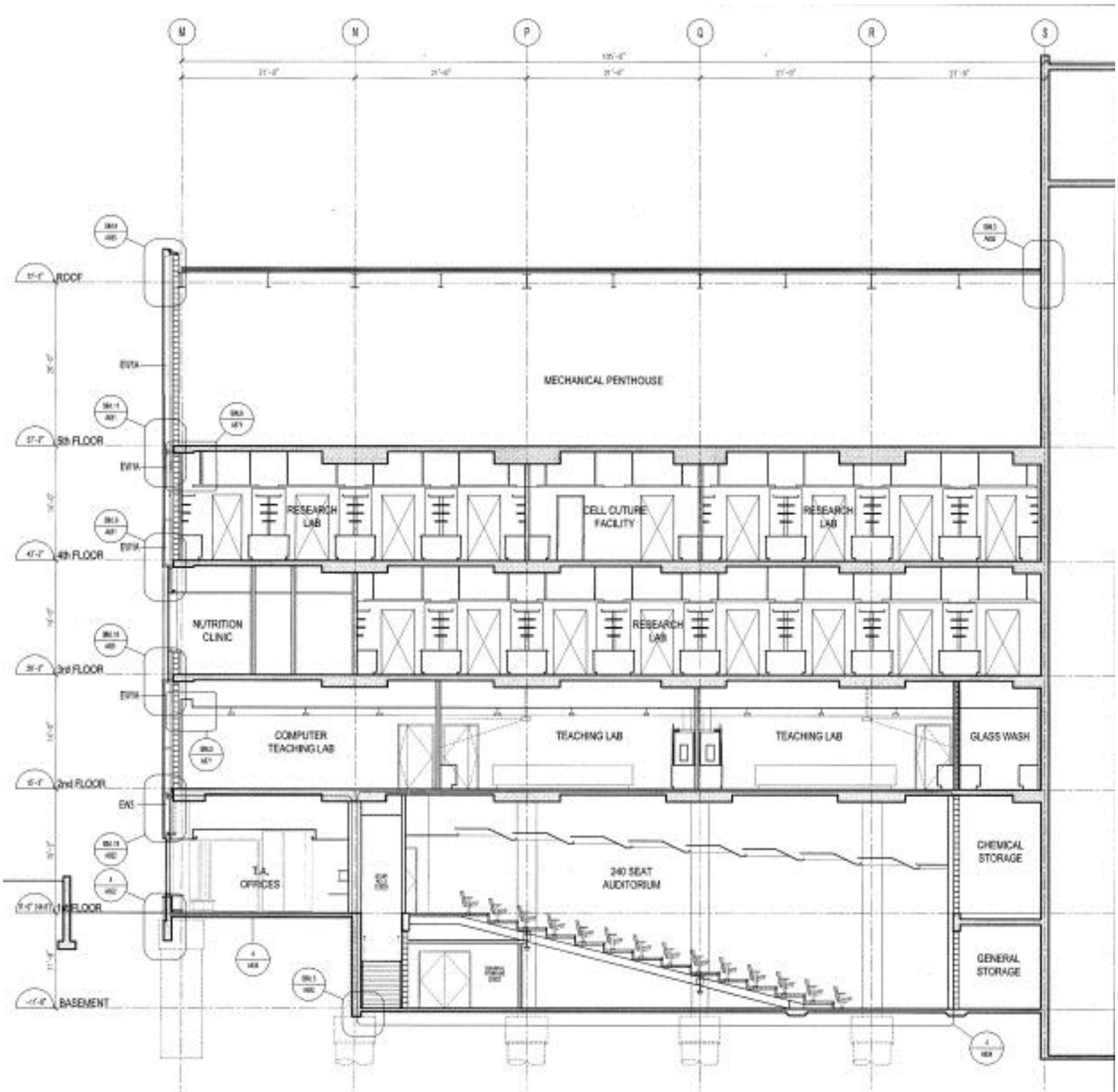


Figure 4 Section 3 through portion of building at -45° rotation (see Figure 1), taken from 4/A402.

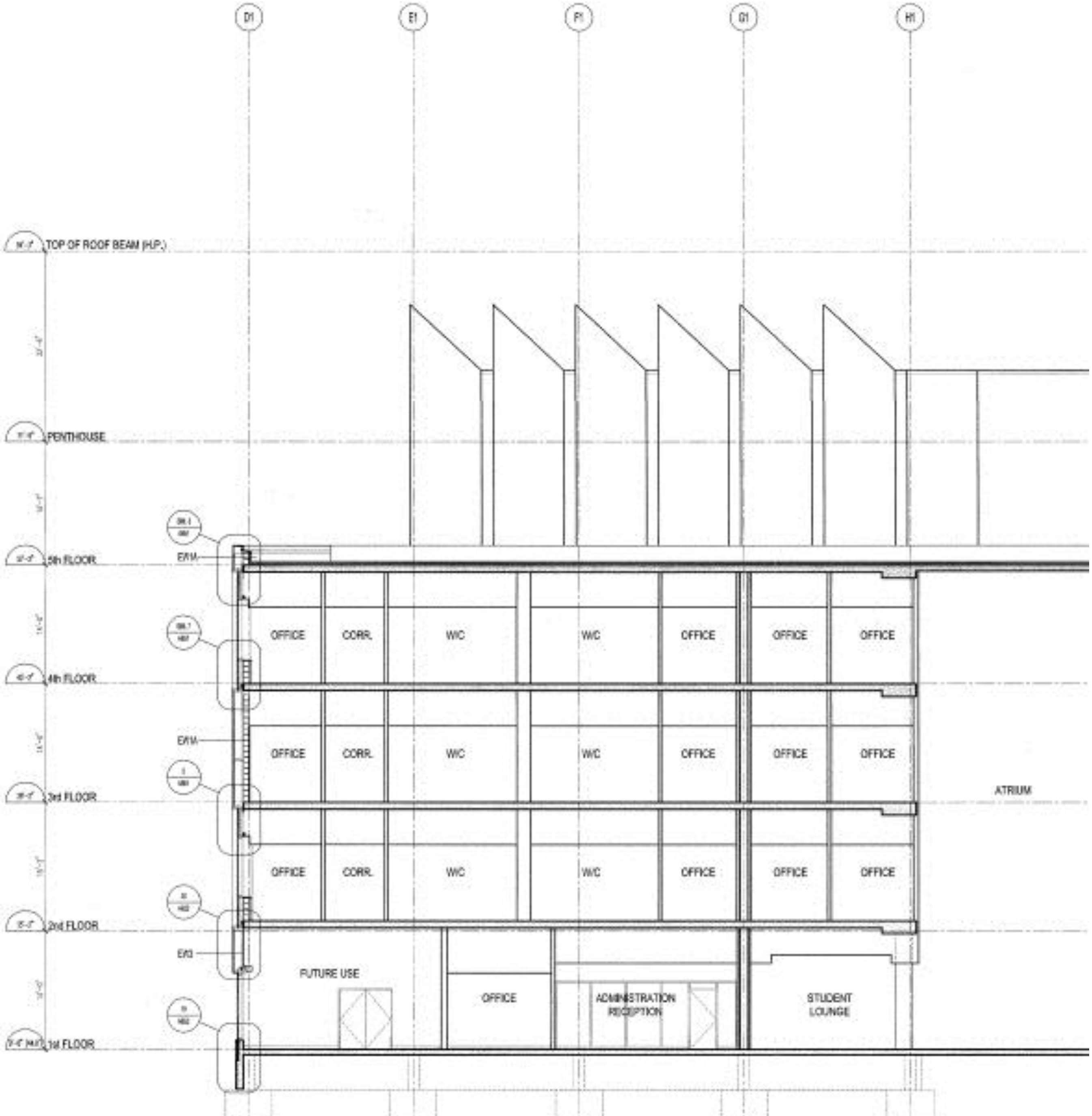


Figure 5 Section 4 through portion of building at -20° rotation (see Figure 1), taken from 3/A403.

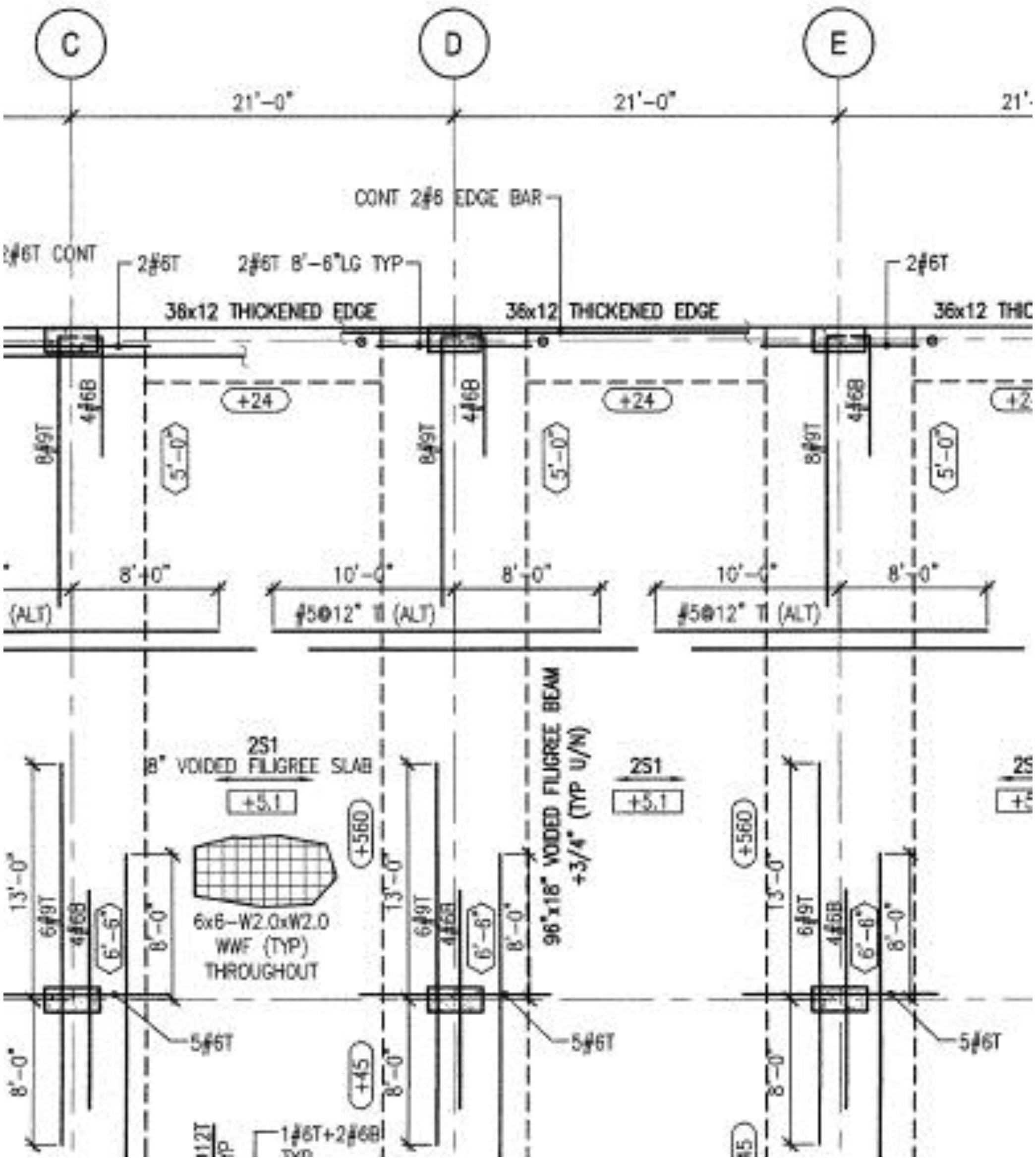


Figure 6 Enlarged floor plan for the area in which the gravity checks were performed, taken from S202 (levels 2 through 4 are identical, and reinforcing is only displayed on level 2). Slab design moments are boxed (k-ft/ft), beam design moments are enclosed in an oval (k-ft), and the location of the first void in the beams with relation to the face of columns is enclosed in a prism-like shape.

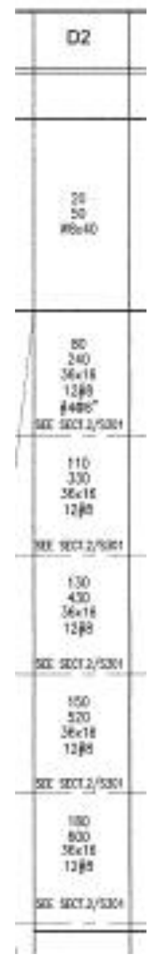
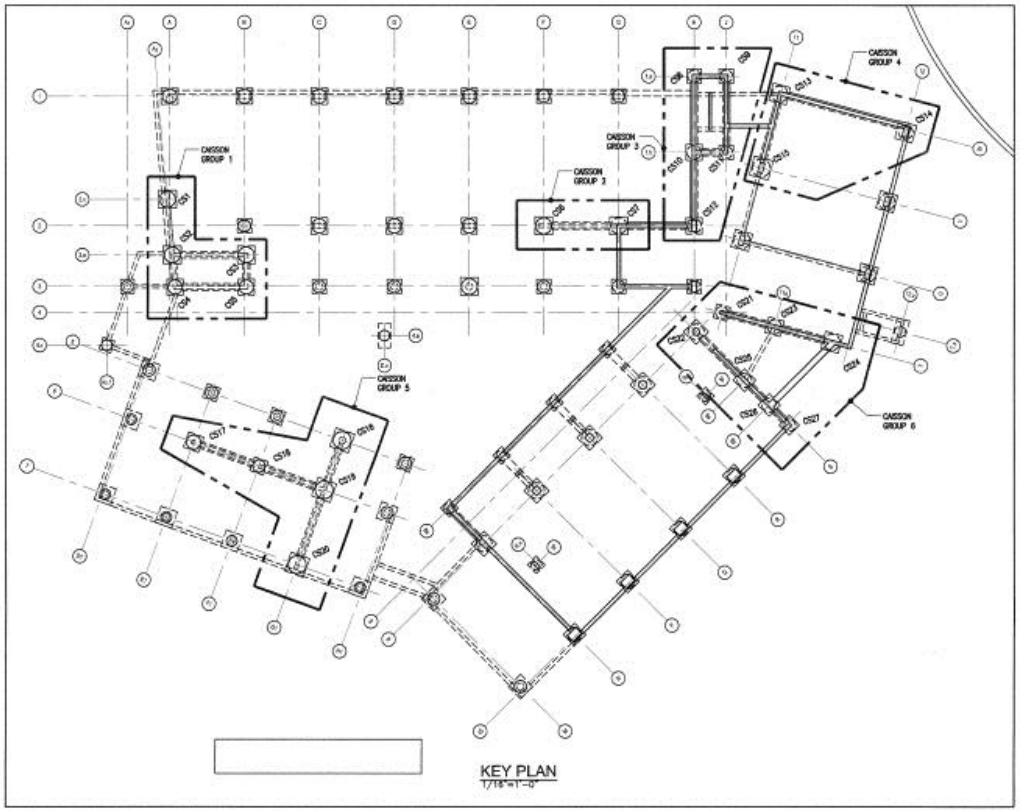


Figure 7 Column D/2 from the column schedule, Sheet S301



CAISSON LOAD TABLE FOR SHEAR WALL FOUNDATION				
CAISSON GROUP	CAISSON ID	DOWN-WARD (kip)	UP-WARD (kip)	LATERAL (kip)
1	CS1	1010	75	90
	CS2	1325	–	75
	CS3	1035	–	80
	CS4	1275	–	75
	CS5	1065	–	80
2	CS6	1875	–	110
	CS7	1050	–	55
3	CS8	855	–	50
	CS9	780	–	50
	CS10	1055	–	45
	CS11	810	–	45
	CS12	1380	–	50
4	CS13	1635	–	40
	CS14	1140	–	50
	CS15	1285	–	35
5	CS16	550	15	85
	CS17	580	–	85
	CS18	685	–	65
	CS19	745	–	105
	CS20	985	25	100
6	CS21	820	–	35
	CS22	480	–	35
	CS23	930	–	30
	CS24	1205	–	45
	CS25	1020	–	35
	CS26	990	–	45
	CS27	955	–	45

NOTE:

1. CAISSON LOADS SHOWN IN THIS DRAWING ARE INTENDED FOR ALTERNATE FOOTING DESIGN. CAISSON GROUPS 1 TO 6 RESIST LATERAL LOADS.
2. SEE DRAWING NO: S301 AND S302 FOR CAISSON GRAVITY LOAD AND CAISSON SIZE.