Technical Report 3

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Lateral System Analysis

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University Sciences Building

Northeast USA

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Executive Summary

The main purpose of this technical report is to evaluate the effectiveness of the lateral system of the University Sciences Building (USB). This is a new, 138,000 square foot laboratory and classroom building located on an urban university campus in the Northeast USA. It has a construction cost of approximately $50 million, and has several unique architectural features, such as a biowall and a 5-story atrium through the core of the building. The main gravity system consists of voided filigree slabs and beams resting on cast-in-place columns, but the mechanical penthouse is constructed of steel. The lateral system consists of 15 shear walls scattered throughout the building, augmented above the concrete-steel transition by four braced frames.

The analysis contained within this technical report began by verifying dead, live, and snow loads used in the structural drawings. Next, both wind and seismic loads were calculated for the building using the Main Wind Force Resisting System procedure and the Equivalent Lateral Force procedure given in Chapters 6 and 12 of the ASCE 7-05. It was found that the seismic loads controlled the design of the lateral system by a factor of 1.5 in both the North-South (N-S) and East-West (E-W) directions.

Next, a finite element lateral model was built of the USB in ETABS. This first model was built with rigid diaphragms and all gravity elements modeled to accurately represent the stiffness of the structure. This decision was predicated on the knowledge that a semi-rigid diaphragm model would be constructed to check the diaphragm forces that developed in the link element located on the plan-south side of the building, which was observed to be the only path for some lateral forces to reach shear walls. This was of concern because of the significantly reduced cross-section in the link. Upon verification of the accuracy of the rigid diaphragm model, it was transformed into a semi-rigid diaphragm model in order to check all forces to determine what effect the semi-rigid properties had on the behavior of the structure. Both models were built as two sub-models, one with each wall assigned its own pier label to better report shear forces in the walls, and one with the walls grouped to better report the moment capacity of the wall groups. This is based upon the differing behavior of shear walls in shear (which they carry individually) and in bending (which they carry as a group when the walls are cast together).

Upon completion of the models, modal information was used to recalculate seismic forces using the Modal Response Spectrum Analysis procedure given in Chapter 12 of ASCE 7-05. This decision was made because this analysis incorporates more modes than the Equivalent Lateral Force method, and therefore provides a more accurate (and typically lower) base shear value. All loads (wind and seismic) were incorporated into the models using load cases for forces in the N-S (x) and E-W (y) directions as well as accidental moments in both directions due to the applied loads. These accidental moments were applied as their own load case to simplify the process of incorporating them into the required load combinations from Chapter 2 of ASCE 7-05.

In order to verify the accuracy of the models, the centers of mass, center of rigidity, shear forces, moments, and drifts were recorded for both types of diaphragms. The centers of mass and rigidity were verified with hand calculations. The shear forces (and thus the moments and drifts) could not be replicated by hand due to the complexity of the building. In lieu of replicating the values, it was chosen to calculate both shear and moment capacities of the lateral force resisting elements. These were found in most cases to be more than adequate, and where this was not the case, it was attributed to simplifications made in order to be able to perform the calculations easily by hand. Shear and moment demands were found to be similar for the rigid and semi-rigid models. Conversely, drift was found to be very sensitive to the modeling method chosen, and was in fact found to be excessive for the semi-rigid model. This will be investigated more in coming studies.

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).

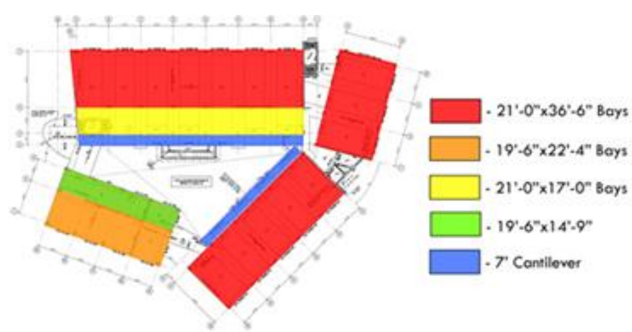


Figure 4 Floor plan from Sheet S203 showing typical bay sizes.

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.

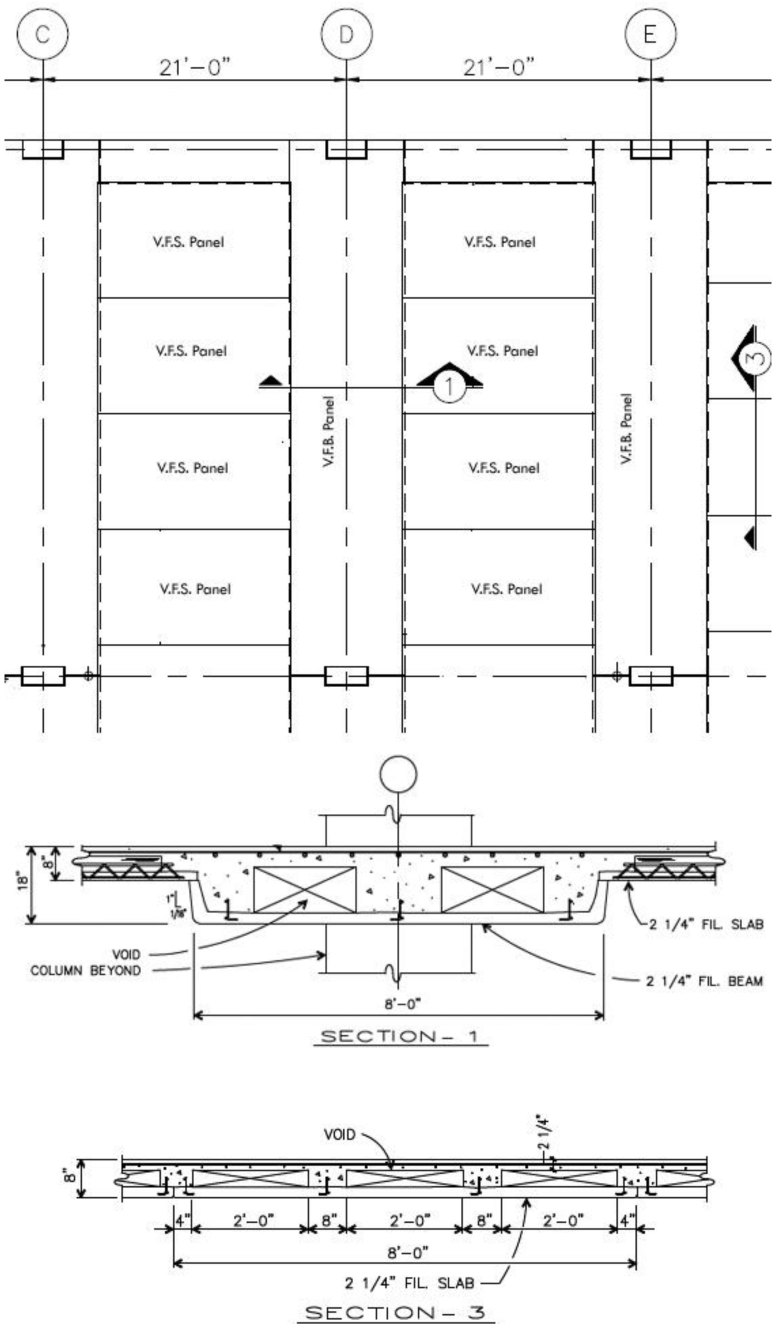


Figure 5 Typical bay with section cuts showing the condition within the beam and the slab. Modified from the filigree slab shop drawings and not to scale (NTS).

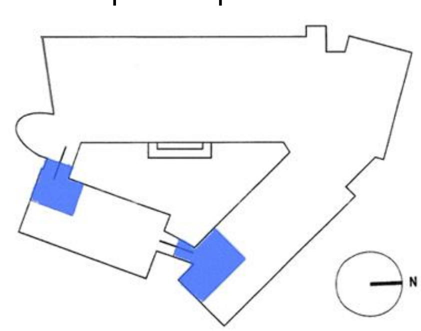


Figure 6 Modified keyplan from Sheet S202 showing the "link" areas in blue.

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.

Roof Systems

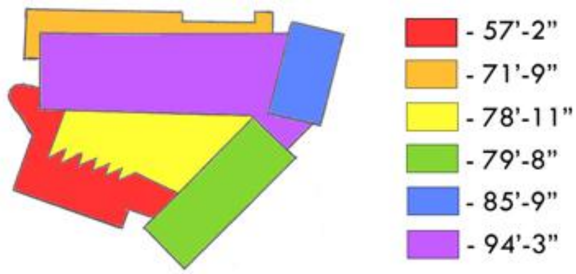


Figure 7 Modified keyplan image from Sheets S205, & S206 showing different roof heights in relation to 0'-0"

There are six different roofs on the USB, due mostly to architectural reasons. Figure 7 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".

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

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. 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. For ease of reference, the walls were numbered, as displayed in Figure 8. Figure 9 shows the shear wall elevations of all 15 shear walls, taken from the ETABS model used for lateral analysis. The walls are anchored at the base by grade beams that run the full length of the walls. It is important that the foundations are designed to resist any overturning moments that may occur on them due to the in-plane shear forces carried by the shear walls. Although these overturning moments were calculated for each shear wall, the accompanying foundation design was not evaluated in this technical report. The structural engineer of record's calculations with regard to the uplift on the caissons is included in the project documents as Sheet S310 and in this report as Figures D.7 and D.8 in Appendix D.

Five steel braced frames are also included in the structure, at or above the 5th Level. These were also numbered for reference (see Figure 8). These are particularly important to the Atrium Roof level, which has very little capacity to resist lateral forces without them.

A last major consideration for the lateral system was the necessity of transferring large diaphragm forces through the bottle-neck section of the link on the plan-south side of the building. Logic dictates that lateral forces that are accumulated in the plan-southeast portion of the building should distribute to the nearest shear walls (Walls 13 and 14) based upon their stiffness ratios. However, for any diaphragm forces from the plan-southeast portion to reach Wall 14, they have to cross the significantly reduced section of the link. Therefore, it was of particular interest to determine what sort of force was experienced by the link, and to verify the adequacy of the link to carry these forces.

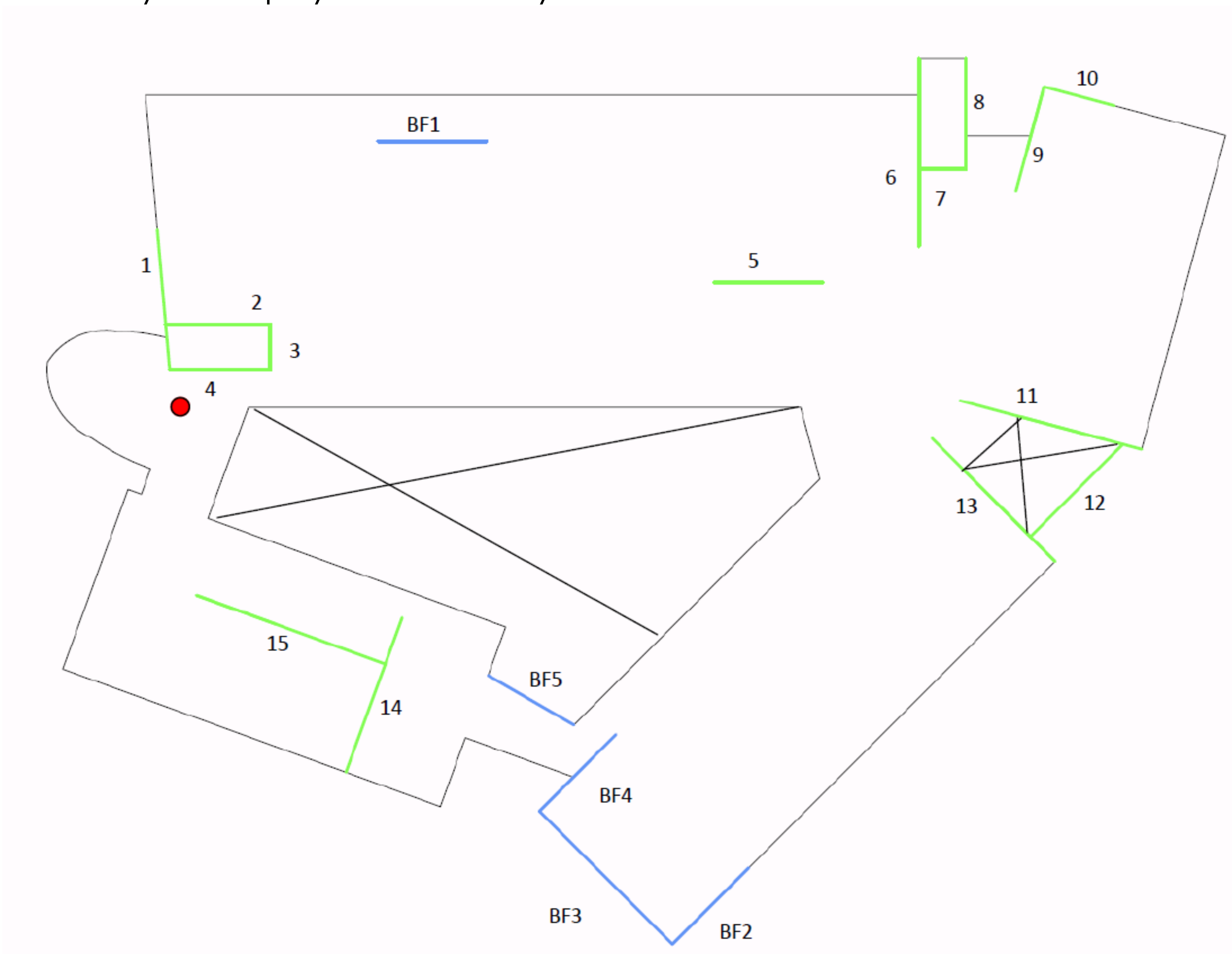


Figure 8 Floor plan with shear walls indicated in green and braced frames (present only above Level 5) indicated in blue. All are labeled for ease of reference. Red dot is the reference location for (0,0) ft.

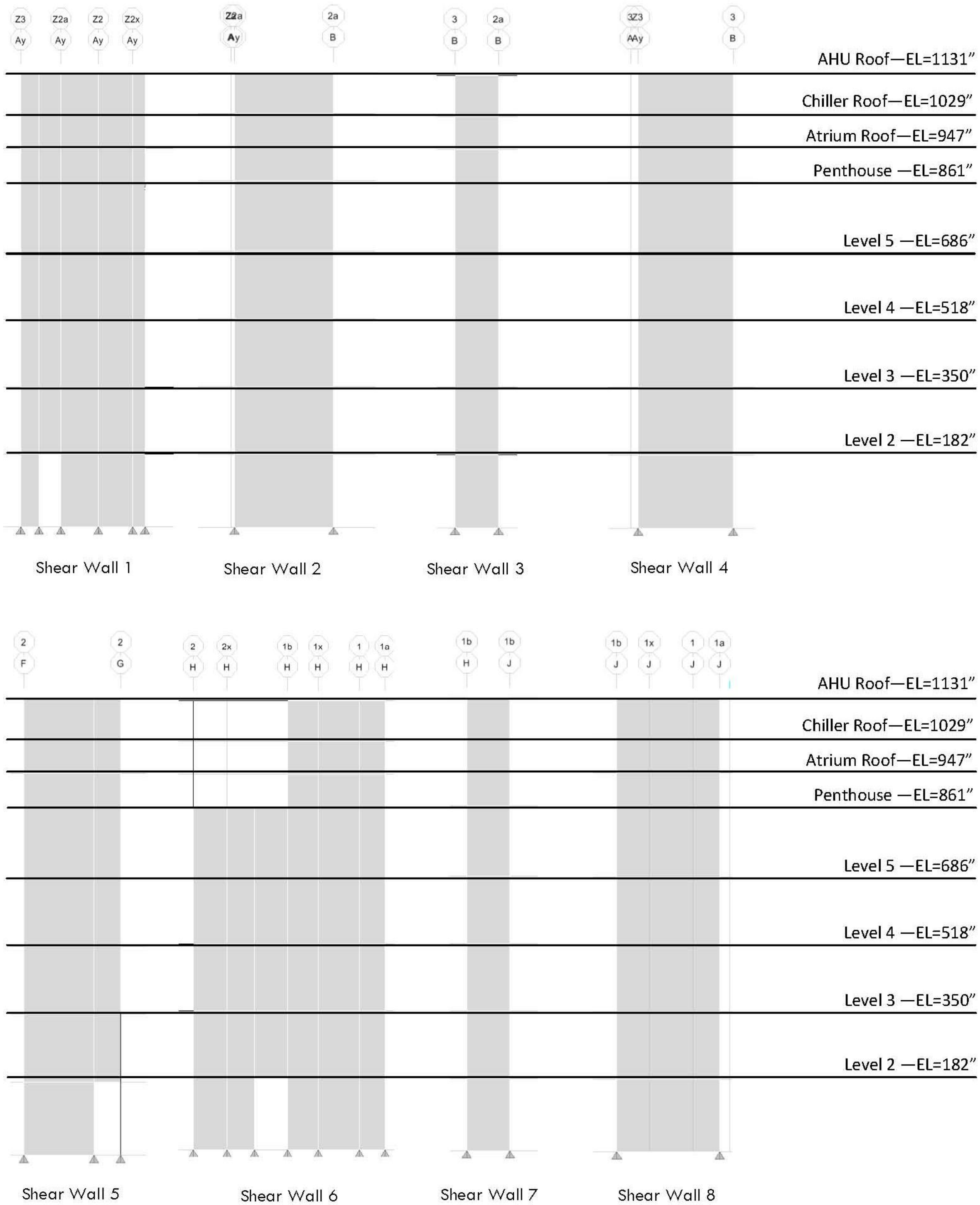


Figure 9 Shear wall elevations from ETABS model.

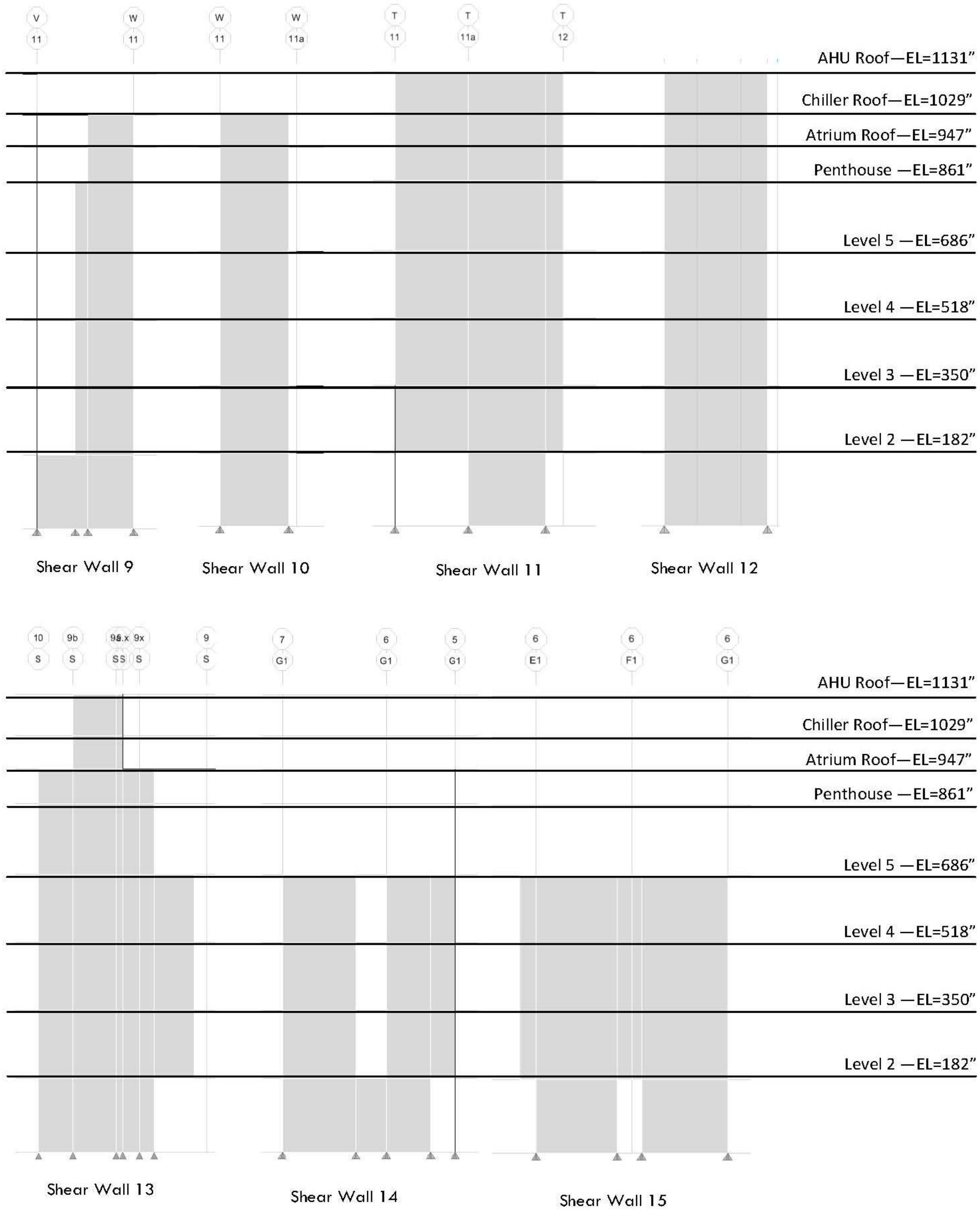


Figure 9 (cont.) Shear wall elevations from ETABS model.

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, year not specified)
- 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 and ASCE 7-05. ACI 318-08 was used in the production of this technical report, although ACI318-05 is the version required by IBC 2006.

Materials Used

Due to the variety of structural types on this project, there are also many different kinds of materials. These are listed in Table 1 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

Table 1 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.

Dead and Live Loads

The structural drawings list superimposed dead loads, summarized in Table 2. 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

Table 2 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 Table 3. The overall building weight was found to be approximately 25,500 k (not including the Ground Level, which is a slab-on-grade, and thus does not contribute to seismic building weight).

Live loads were also listed on the structural drawings. These were compared to live loads in Table 4-1 in ASCE 7-05 based on the usage of the spaces, and the results are summarized in Table 4. 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

Table 3 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 Table 5. The structural drawings used a C_t of 0.8, but this does not seem to be

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

Table 4 Summary of design live loads, compared to ASCE 7-05 typical live loads.

permissible by code. Therefore, the 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.

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

Table 5 Summary of roof snow load calculations.

Lateral Loads

In order to better understand the lateral systems, wind loads and seismic loads were calculated for this technical report. These were calculated by hand, and then applied to a lateral model of the structure created in ETABS. The hand calculations for the wind loads can be found in Appendix A, and the hand calculations for the seismic loads can be found in Appendix B.

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 used as L and B (see Appendix A).

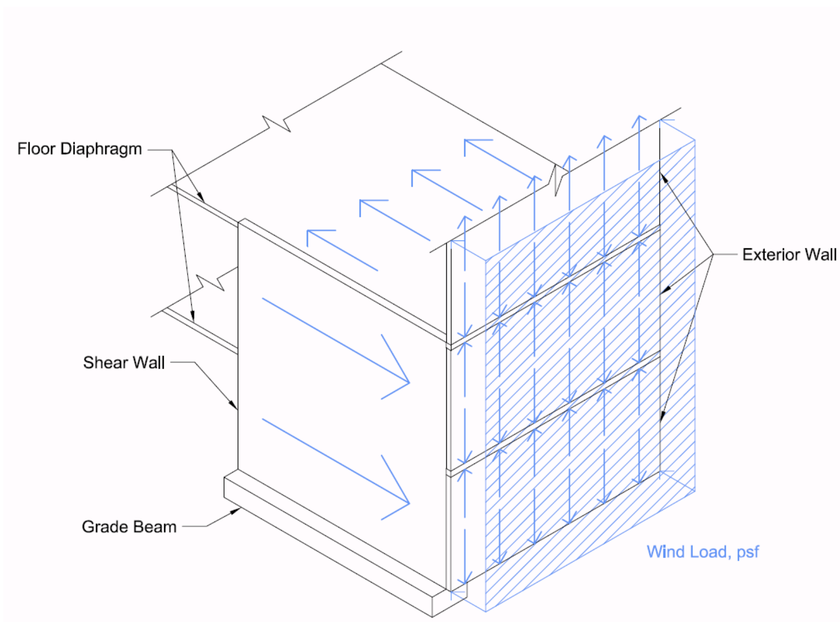


Figure 10 Diagram of the lateral load path for wind loads.

The wind loads on this building are collected by the cladding on the exterior of the building. 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 to the foundations, and therefore to grade. This load path is illustrated in Figure 10.

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 as there are no building expansion joints in the USB.

For this technical report, accidental moments were also calculated. This was achieved through the use of the four load cases for torsion due to wind, given in Figure 6-9 of ASCE 7-05 and included as Figure 11. For ease of manipulation, wind loads were entered into the model in four basic static load cases: wind forces in the N-S direction (WX), wind forces in the E-W direction (WY), accidental moments due to the N-S loads (WMX), and accidental moments due to the E-W loads (WMY). These were then combined using load combinations to account for both the required load combinations in Chapter 2 of ASCE 7-05 and the four required cases specified in Chapter 6, resulting in 90 different load combinations for wind loads (these are listed in Appendix A). The accidental moments were calculated with the following formula:

$$WMX = W_X * 0.15 * B_X$$

$$WMY = W_Y * 0.15 * B_Y$$

Where Wx or Wy are the story force at a given level in the direction under consideration and Bx or By are the building dimension in the direction under consideration. For this calculation, the pseudo-footprint dimensions were used.

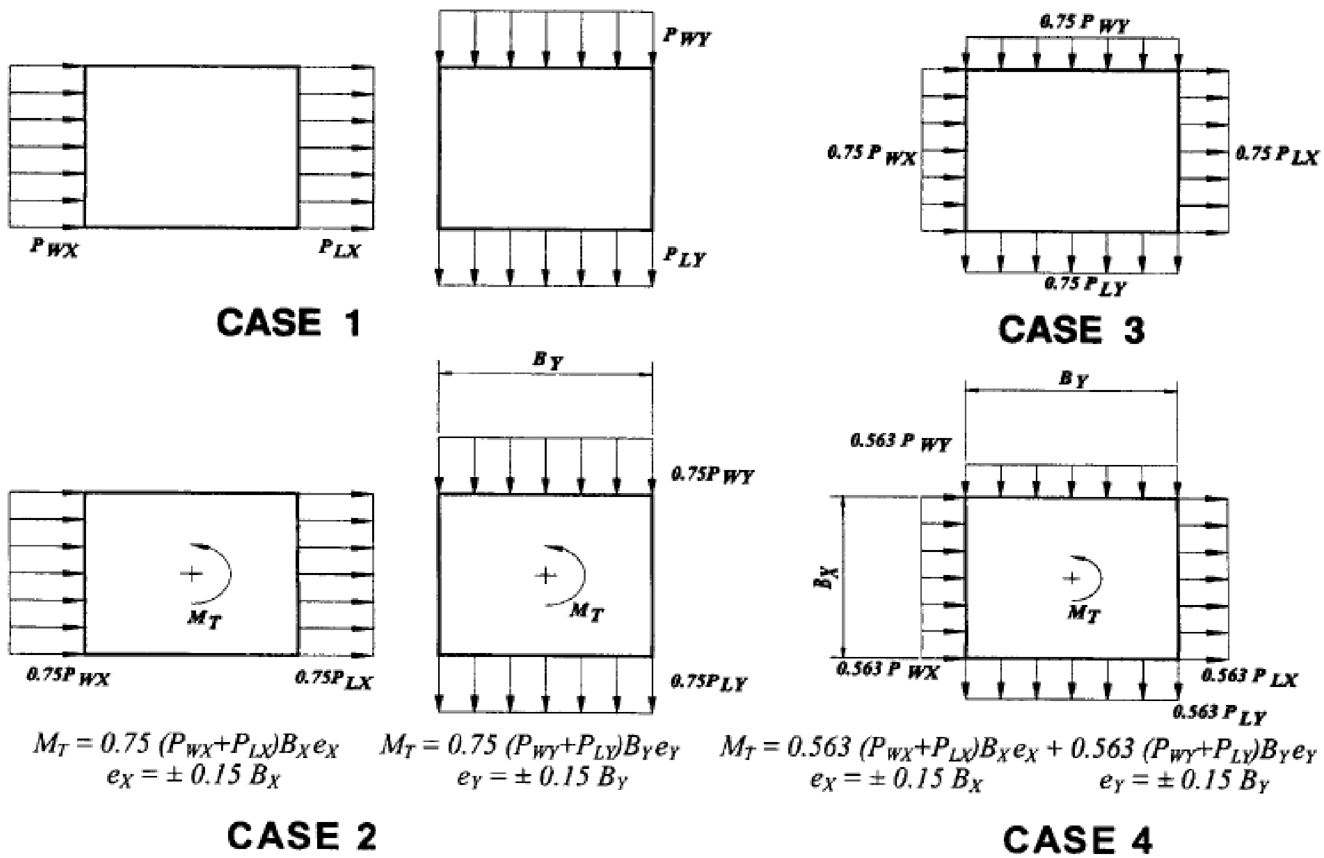


Figure 11 Torsional wind load cases from Figure 6-9 in ASCE 7-05.

The wind pressures in the N-S direction are listed and diagramed in Figure 12. These were resolved into wind forces in the N-S direction, which are listed and diagramed in Figure 13. 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 14. These were resolved into wind forces in the E-W direction, which are listed and diagramed in Figure 15. 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

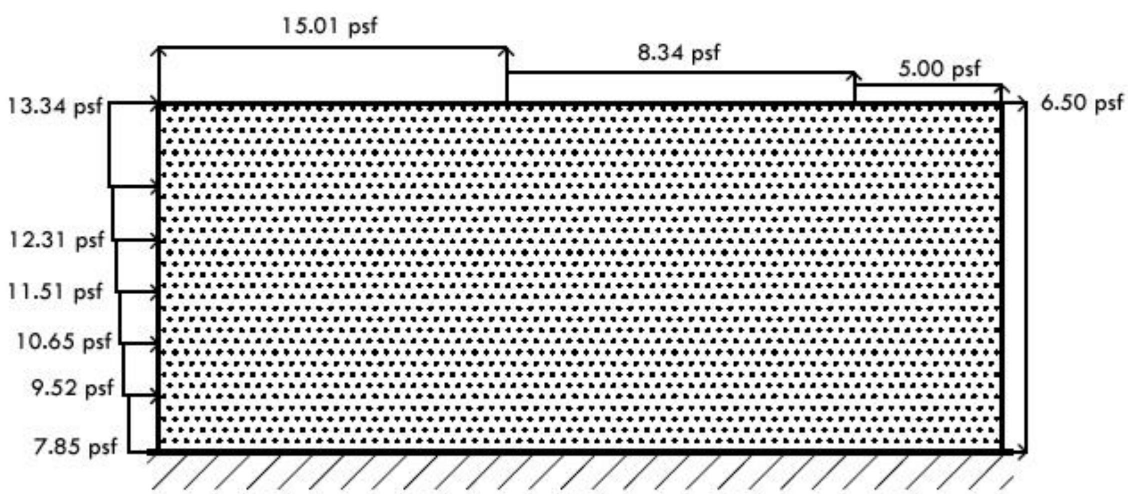


Figure 12 List and diagram of N-S direction wind pressures.

Wind Forces - N-S Direction								
Floor Level	Elevation (ft)	Trib. Below		Trib. Above		Story Force (k)	Story Shear (K)	Accidental Moment (k-in)
		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	4296.43
2nd	15.17	7.59	1289.45	7.00	1190	37.57	262.87	8722.94
3rd	29.17	7.00	1190.00	7.00	1190	39.47	225.30	9165.94
4th	43.17	7.00	1190.00	7.00	1190	41.85	185.83	9717.15
5th	57.17	7.00	1190.00	7.29	1239.3	44.75	143.98	10392.10
Penthouse	71.75	7.29	1239.30	11.25	1912.5	61.27	99.22	14227.03
Roof	94.25	11.25	1912.50	N/A	0.00	37.95	37.95	8812.67
							Total Base Shear=	281.37 k
							Total Accidental Moment=	65,334.25 k-in

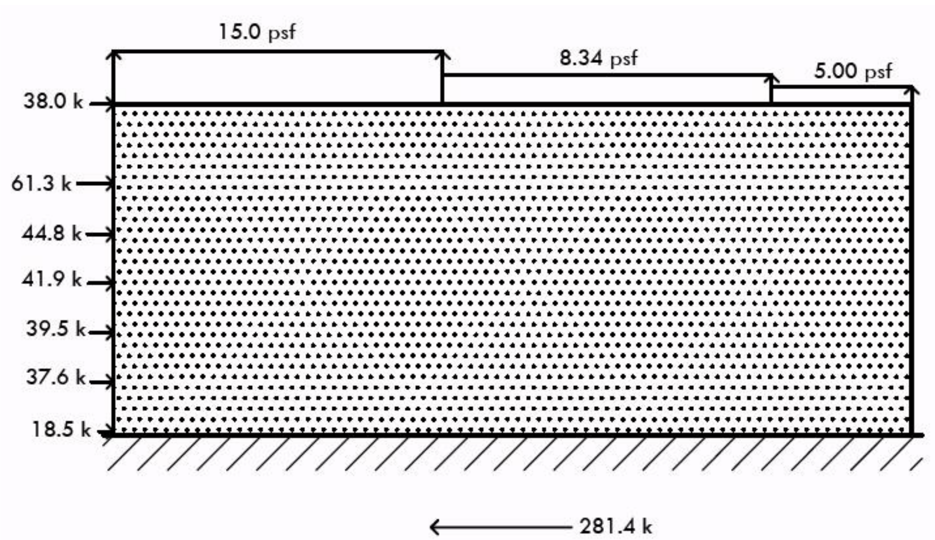


Figure 13 List and diagram of N-S direction wind forces.

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

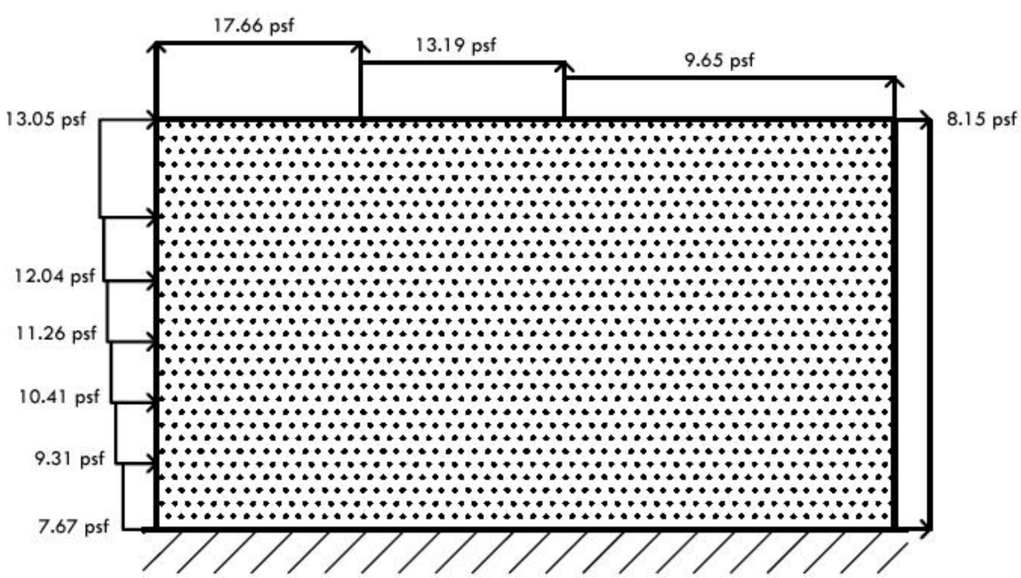


Figure 14 List and diagram of E-W direction wind pressures.

Wind Forces - E-W Direction								
Floor Level	Elevation (ft)	Trib. Below		Trib. Above		Story Force (k)	Story Shear (K)	Accidental Moment (k-in)
		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	9853.65
2nd	15.17	7.59	1729.38	7.00	1596.00	55.24	380.22	19885.62
3rd	29.17	7.00	1596.00	7.00	1596.00	57.50	324.98	20700.17
4th	43.17	7.00	1596.00	7.00	1596.00	60.61	267.48	21820.98
5th	57.17	7.00	1596.00	7.29	1662.12	64.54	206.87	23236.09
Penthouse	71.75	7.29	1662.12	11.25	2565.00	87.94	142.32	31659.98
Roof	94.25	11.25	2565.00	N/A	0.00	54.38	54.38	19576.66
							Total Base Shear=	407.59 k
							Total Accidental Moment=	146,733.14 k-in

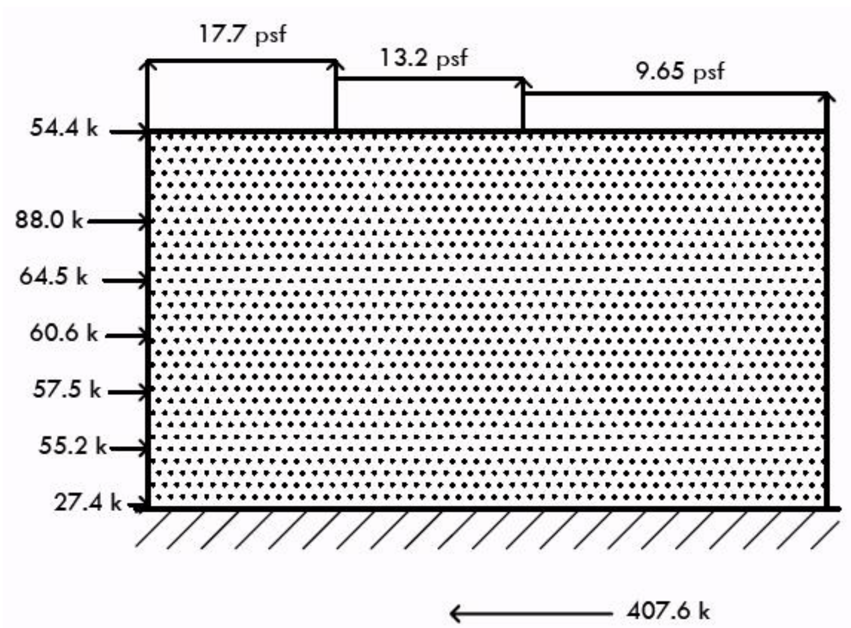


Figure 15 List and diagram of E-W direction wind forces.

Seismic Loads

Seismic loads were first calculated with the Equivalent Lateral Force (ELF) 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. The simplified shear wall data used in this calculation can be found in Appendix B.

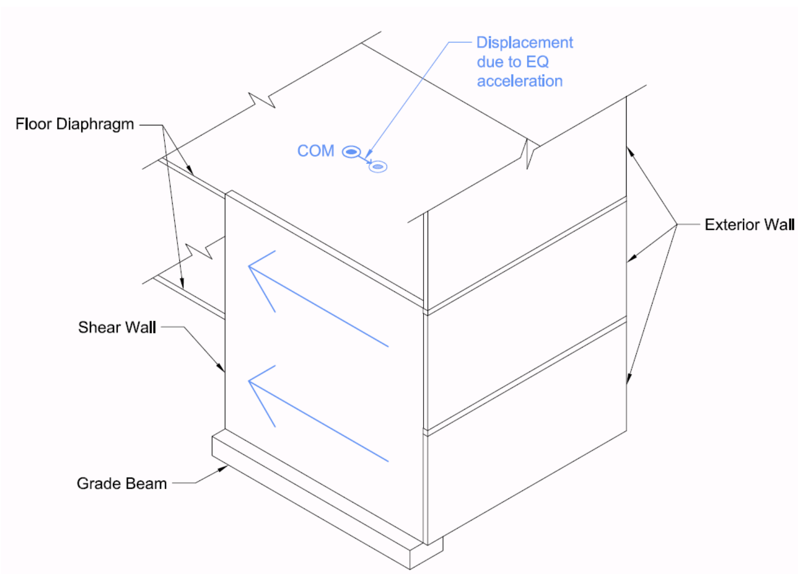


Figure 16 Diagram of the lateral load path for a seismic load.

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. This is diagrammed in Figure 16.

At the time of this report, the total weight used by the structural engineer for the building was not known. However, as service dead load values for each column were listed on the column schedule, a reasonably close approximation could be made of their final building weights. If the Ground Level is not included, adding service dead loads on the columns gives a building weight of approximately 26,800 k. This is reasonably close to the value obtained for this technical report, which is 25,500 k.

ELF seismic forces in the N-S direction are listed and diagramed in Figure 17. The resultant base shear in this direction is 786.68 k, which is about 20% less than the base shear listed for this direction on Sheet S001 (955 k). This order of discrepancy is potentially due to the original engineer not using the Coefficient for Upper Limit on Calculated Period (Cu, ASCE 7-05 Table 12.8-1). For this building, Cu is 1.7. Assuming Cu was not incorporated, and the basic solution was used to find base shear instead of the specific solution for shear walls, base shear would be 1010 K in both directions (5-10% error).

ELF seismic forces for the E-W direction are listed and diagramed in Figure 17. The resultant base shear in this direction is 917 k, which is about 20% less than the base shear listed for this direction on Sheet S001 (1145 k). Again, this difference is probably accounted for by the same discrepancy indicated for the N-S direction.

Accidental moments were also calculated for all seismic forces using the prescribed procedure for this given in section 12.8.4.2 of ASCE 7-05. This requires accidental torsional moments induced by the story force multiplied by an accidental eccentricity equal to 5% of the dimension of the building perpendicular to the forces applied. For ease of manipulation, seismic loads were entered into the model in four basic static load cases: seismic forces in the N-S direction (EX), seismic forces in the E-W direction (EY), accidental moments due to the N-S loads (EMX), and accidental moments due to the E-W loads (EMY). These were then combined using load combinations to account for the required load combinations in Chapter 2 of ASCE 7-05, resulting in 24 different earthquake load combinations (these are listed in Appendix B). The amplification factor for accidental moments (ASCE 7-05, section 12.8.4.3) was not considered as it is not required for SDC B structures.

After the lateral model was constructed in ETABS, base shears were found again using the Modal Response Spectrum Analysis (MRSA) procedure on a finite element model constructed in ETABS with the cracked section properties modeled by a 50% reduction on the modulus of elasticity for all concrete materials. This involves calculating a Cs-like quantity using the modal periods for sufficient modes to obtain 90% mass-participation in two orthogonal translational directions. This base shear is typically lower than that calculated by the ELF procedure. However, it is limited by an absolute minimum of 85% of the base shear calculated by ELF. The equations for this process are as follows:

$$C_{m,i} = min \begin{matrix} \dfrac{S_{D1}}{T_i \ \frac{R}{I}} \\ \dfrac{S_{DS}}{\frac{R}{I}} \end{matrix}$$

$$V_m = W(\Sigma_{i=1}^{n}(C_{m,i}M\%_i)^2)^{1/2} \geq 0.85V_{ELF}$$

Where M%i refers to the mass participation percentage of mode "i" in decimal form. The resulting Cm values can be found in Appendix C, or in the "Building Properties" subsection of the "Lateral System Analysis" section.

As will be discussed in the Computer Modeling Process section, both a rigid and semi-rigid diaphragm model were analyzed in ETABS. These resulted in different periods, and thus different base shears. The rigid diaphragm MRSA seismic forces in the N-S Direction and E-W Direction are listed and diagrammed in Figures 19 and 20, respectively. This model yielded base shears of 716.6 k in the N-S Direction and 936.7 k in the E-W Direction, neither of which was controlled by the 85%V_{ELF} minimum. The semi-rigid diaphragm MRSA seismic forces in the N-S Direction and the E-W Direction are listed and diagrammed in Figures 21 and 22, respectively. This model resulted in base shears of 668.7 k in the N-S Direction and 779.5 k in the E-W Direction, both of which were controlled by the 85% V_{ELF} minimum.

ELF 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)	Accidental Moment (k-in)
2nd	4130.90	15.17	91575.38	0.04	32.18	786.68	4402.69
3rd	4105.92	29.17	191771.96	0.09	67.40	754.49	9219.85
4th	4105.92	43.17	299787.93	0.13	105.36	687.10	14412.96
5th	5510.78	57.17	554166.98	0.25	194.76	581.74	26642.78
Penthouse	4870.93	71.75	634590.24	0.28	223.02	386.98	30509.31
Atrium Rf.	791.87	78.92	114988.38	0.05	40.41	163.96	5528.32
Chiller Rf.	455.68	85.75	72737.75	0.03	25.56	123.55	3497.03
AHU Rm. Rf.	1568.32	94.25	278812.18	0.12	97.99	97.99	13404.50
						Base Shear [$V=C_sW$]=	786.68 k
						Total Accidental Moment=	94,212.93 k-in

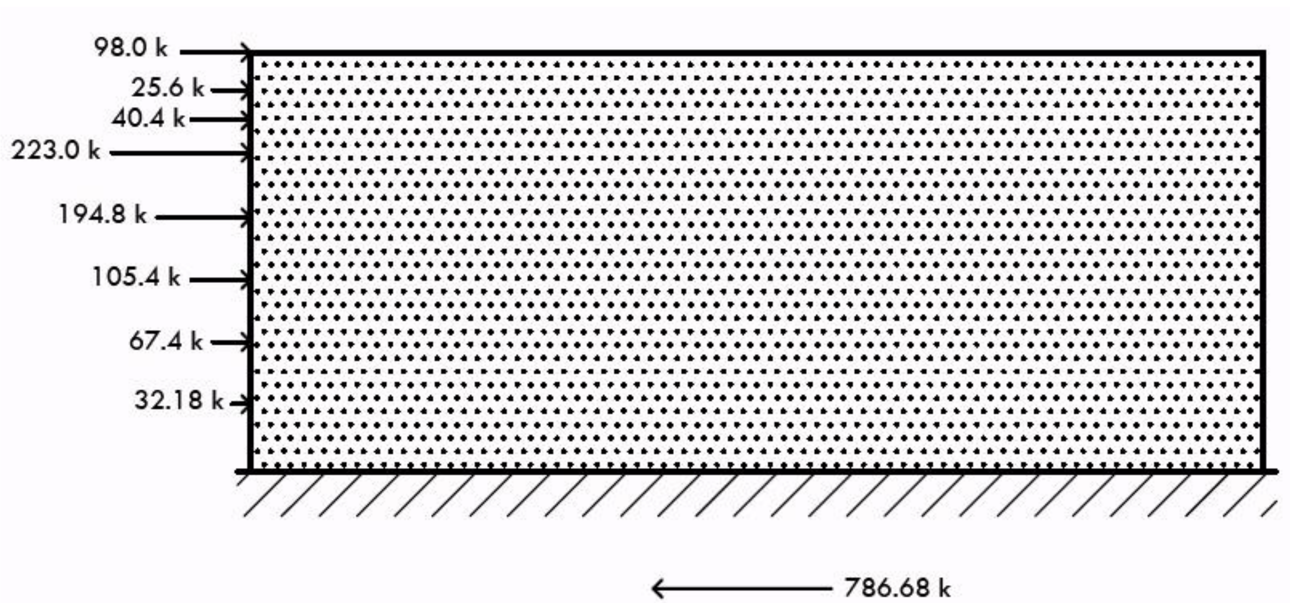


Figure 17 List and diagram of N-S direction seismic forces as calculated by the Equivalent Lateral Force Procedure.

ELF 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)	Accidental Moment (k-in)
2nd	4130.90	15.17	78772.65	0.04	40.38	917.04	5524.24
3rd	4105.92	29.17	159093.62	0.09	81.56	876.66	11157.07
4th	4105.92	43.17	243361.52	0.14	124.76	795.11	17066.69
5th	5510.78	57.17	442916.42	0.25	227.06	670.35	31061.26
Penthouse	4870.93	71.75	500851.78	0.28	256.76	443.29	35124.21
Atrium Rf.	791.87	78.92	90277.56	0.05	46.28	186.54	6331.07
Chiller Rf.	455.68	85.75	56844.48	0.03	29.14	140.26	3986.44
AHU Rm. Rf.	1568.32	94.25	216753.80	0.12	111.12	111.12	15200.72
						Base Shear [$V=C_sW$]=	917.04 k
						Total Accidental Moment=	110,250.99 k-in

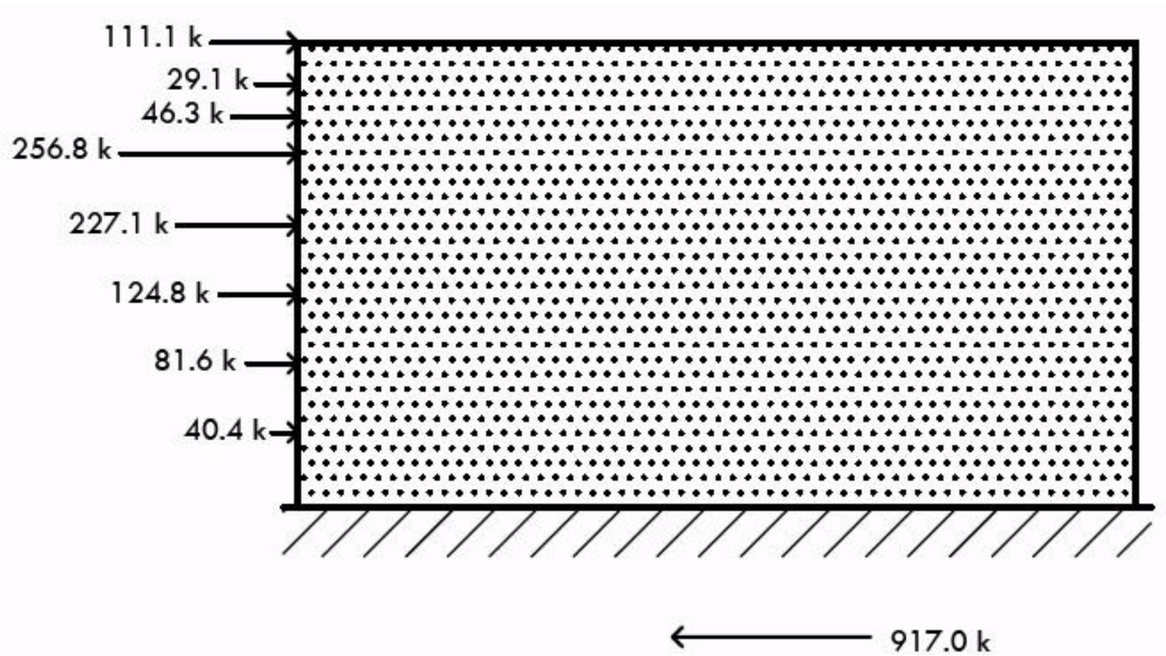


Figure 18 List and diagram of E-W direction seismic forces as calculated by the Equivalent Lateral Force Procedure.

Rigid Diaphragm Model - 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)	Accidental Moment (k-in)
2nd	4130.90	15.17	91575.38	0.04	29.32	716.57	4010.30
3rd	4105.92	29.17	191771.96	0.09	61.39	687.25	8398.15
4th	4105.92	43.17	299787.93	0.13	95.97	625.86	13128.42
5th	5510.78	57.17	554166.98	0.25	177.40	529.89	24268.28
Penthouse	4870.93	71.75	634590.24	0.28	203.14	352.49	27790.21
Atrium Rf.	791.87	78.92	114988.38	0.05	36.81	149.35	5035.61
Chiller Rf.	455.68	85.75	72737.75	0.03	23.28	112.54	3185.36
AHU Rm. Rf.	1568.32	94.25	278812.18	0.12	89.25	89.25	12209.85
						Base Shear [$V=C_sW$]=	716.57 k
						Total Accidental Moment=	85,816.34 k-in

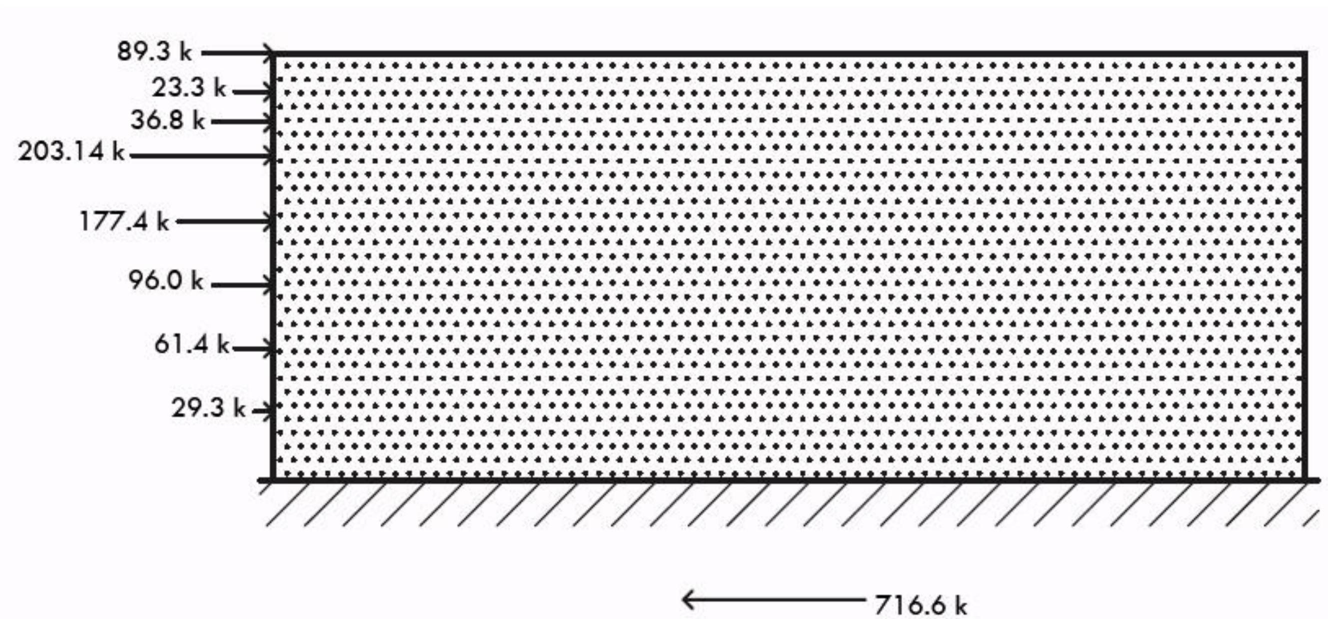


Figure 19 List and diagram of N-S direction seismic forces as calculated by the Modal Response Spectral Analysis Procedure for the Rigid Diaphragm periods.

Rigid Diaphragm Model - 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)	Accidental Moment (k-in)
2nd	4130.90	15.17	78772.65	0.04	41.25	936.72	5642.75
3rd	4105.92	29.17	159093.62	0.09	83.31	895.47	11396.41
4th	4105.92	43.17	243361.52	0.14	127.43	812.16	17432.81
5th	5510.78	57.17	442916.42	0.25	231.93	684.73	31727.60
Penthouse	4870.93	71.75	500851.78	0.28	262.26	452.80	35877.70
Atrium Rf.	791.87	78.92	90277.56	0.05	47.27	190.54	6466.89
Chiller Rf.	455.68	85.75	56844.48	0.03	29.77	143.27	4071.96
AHU Rm. Rf.	1568.32	94.25	216753.80	0.12	113.50	113.50	15526.81
						Base Shear [$V=C_SW$]=	936.72 k
						Total Accidental Moment=	112,616.12 k-in

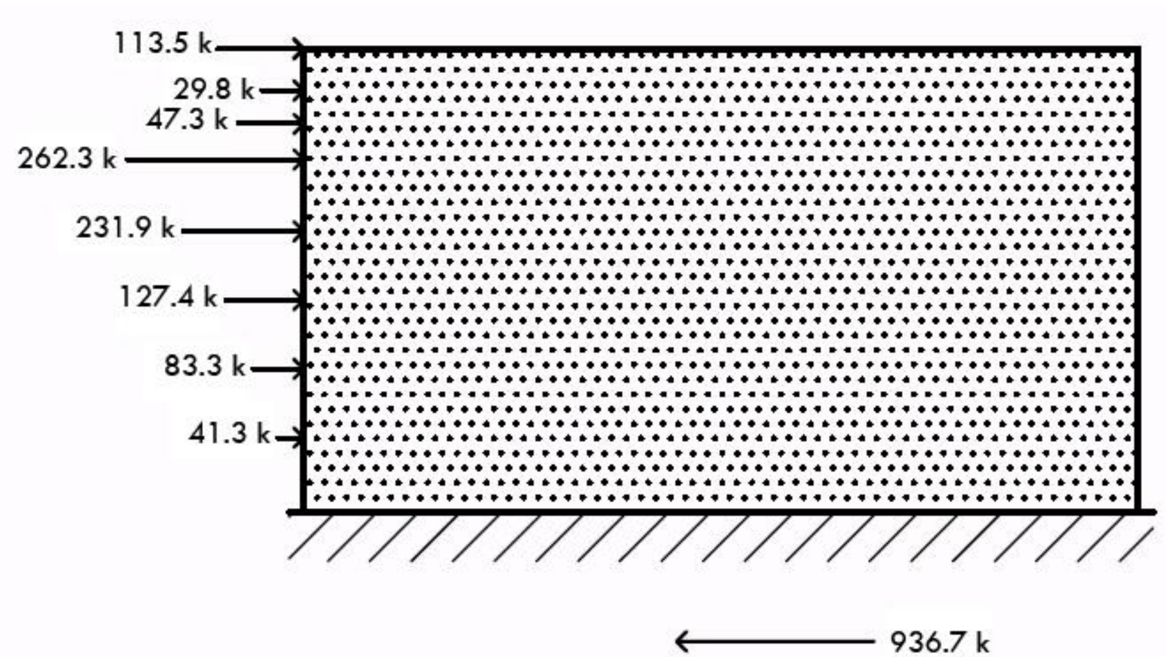


Figure 20 List and diagram of E-W direction seismic forces as calculated by the Modal Response Spectral Analysis Procedure for the Rigid Diaphragm periods.

Semi- Rigid Diaphragm Model - 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)	Accidental Moment (k-in)
2nd	4130.90	15.17	91575.38	0.04	27.36	668.68	3742.28
3rd	4105.92	29.17	191771.96	0.09	57.29	641.32	7836.88
4th	4105.92	43.17	299787.93	0.13	89.55	584.03	12251.01
5th	5510.78	57.17	554166.98	0.25	165.54	494.48	22646.37
Penthouse	4870.93	71.75	634590.24	0.28	189.57	328.93	25932.91
Atrium Rf.	791.87	78.92	114988.38	0.05	34.35	139.37	4699.07
Chiller Rf.	455.68	85.75	72737.75	0.03	21.73	105.02	2972.47
AHU Rm. Rf.	1568.32	94.25	278812.18	0.12	83.29	83.29	11393.83
						Base Shear $[V=C_sW]$=	668.68 k
						Total Accidental Moment=	80,080.99 k-in

Note: Base shear controlled by 85% of ELF base shear requirement.

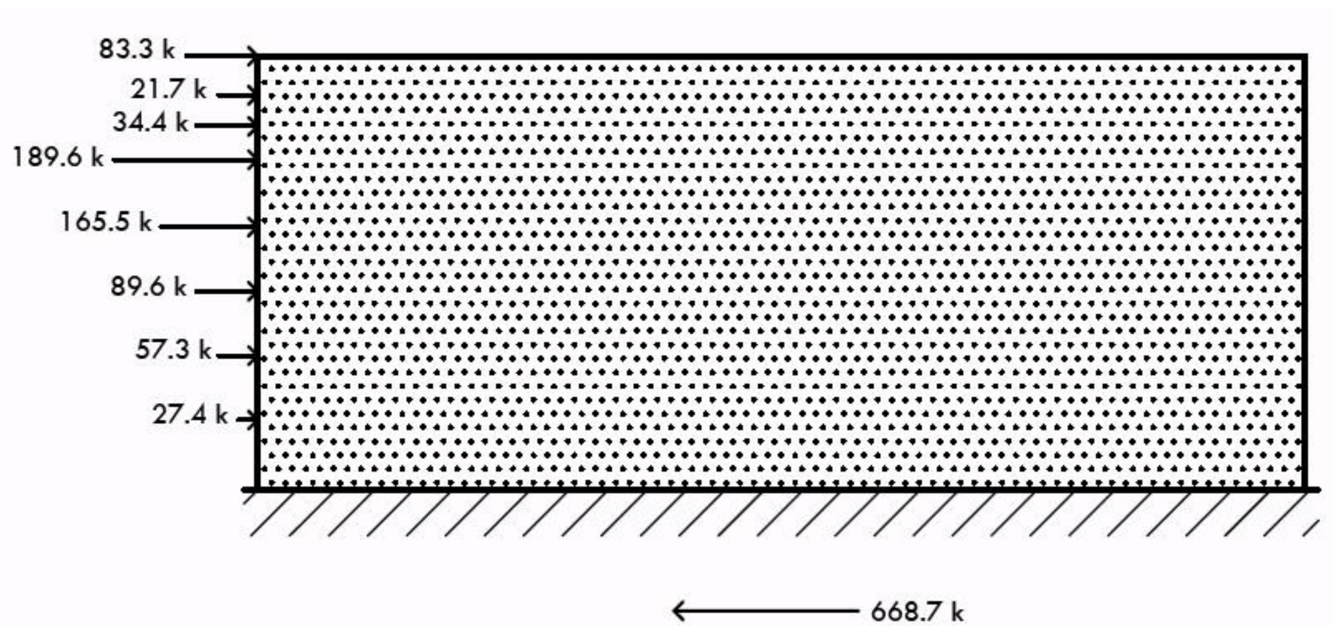


Figure 20 List and diagram of N-S direction seismic forces as calculated by the Modal Response Spectral Analysis Procedure for the Semi-Rigid Diaphragm periods.

Semi-Rigid Diaphragm Model - 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)	Accidental Moment (k-in)
2nd	4130.90	15.17	78772.65	0.04	34.32	779.49	4695.61
3rd	4105.92	29.17	159093.62	0.09	69.32	745.16	9483.51
4th	4105.92	43.17	243361.52	0.14	106.04	675.84	14506.69
5th	5510.78	57.17	442916.42	0.25	193.00	569.80	26402.07
Penthouse	4870.93	71.75	500851.78	0.28	218.24	376.80	29855.58
Atrium Rf.	791.87	78.92	90277.56	0.05	39.34	158.56	5381.41
Chiller Rf.	455.68	85.75	56844.48	0.03	24.77	119.22	3388.48
AHU Rm. Rf.	1568.32	94.25	216753.80	0.12	94.45	94.45	12920.61
						Base Shear [$V=C_sW$]=	779.49 k
						Total Accidental Moment=	93,713.34 k-in

Note: Base shear controlled by 85% of ELF base shear requirement.

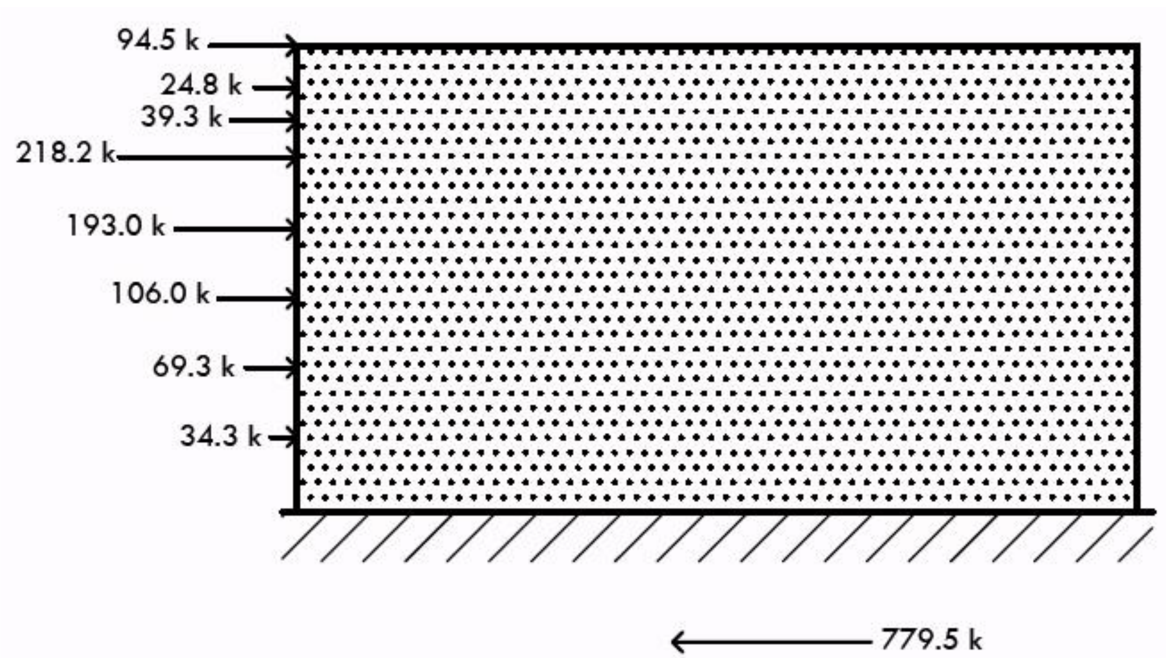


Figure 22 List and diagram of E-W direction seismic forces as calculated by the Modal Response Spectral Analysis Procedure for the Semi-Rigid Diaphragm periods.

Lateral System Analysis

In order to fully understand the behavior of the USB under lateral loading, four finite element models were built in ETABS. Both rigid and semi-rigid diaphragms were considered individually, as well as the behavior of the shear walls in shear vs. bending. Attempts were made to verify all results using hand calculations, although this was not always successful due to the complexity of the lateral system.

Computer Modeling Process

Several assumptions were made while creating all of the lateral models that have a significant impact on the final results given by the models. Firstly, it is required by ACI 318-08 section 8.8.2 that stiffness properties be modified to account for concrete cracking. This can be accomplished either by applying different factors to beams and columns, or by applying a sweeping 50% reduction of gross section properties to all concrete elements. For ease of modeling, the second option was chosen, and this reduction was accomplished by defining the modulus of elasticity for all concrete strengths as 50% of its actual value (i.e. for 4000 psi concrete, E=3600 ksi, but this was included in the model as 1800 ksi).

Material properties were further modified by eliminating self-mass from the material definitions. In order to better control the results of the modal analysis, the masses were directly assigned using the Additional Area Mass function to the floor areas. Weight, however, was left as self-calculating.

The next major assumption was to use shell elements rather than membrane to define all slabs and shear walls. This choice was made because the model had literally thousands of warnings due to lack of restraint when these elements were modeled as membranes. It is believed that this is related to the fact that several shear walls are on axes which have an oblique angle with respect to the forces applied. However, to mimic membrane behavior, the elements were given a "Membrane Thickness" equal to their actual thickness and a "Bending Thickness" equal to 10% of their actual thickness (i.e. the 12" thick shear walls had a Membrane Thickness of 12", but a Bending Thickness of 1.2"). This sufficiently removes the potential for these elements to carry out-of-plane forces while still reducing or eliminating warnings which may render the model less accurate. All shear wall shell elements were meshed into structural elements of a maximum size of 48", and care was taken to ensure that no portion of the shear wall was divided into less than 2 elements wide or tall. This was important because the program requires at least two elements to calculate both tension and compression in a given bending profile with any degree of accuracy.

Lastly, although the model was intended only for lateral analysis, it was decided to model all of the gravity framing as well. This was primarily driven by the knowledge that a semi-rigid analysis was to be a part of this technical report. As the semi-rigid diaphragm is able to deform with respect to itself, it is critical for the full stiffness of the building to be accurately represented, particularly the beams under the slabs. Another influence in this decision was the critical nature of the braced frames at the 5th and Penthouse Levels to the lateral resistance of the Atrium Roof. Without the gravity columns spanning from grade to the Penthouse Level under the braced frames, the frames were not an accurate representation of the structural behavior.

In total, four models were built for this technical report. Due to the concern of the strength of the link element that forms a "bridge" between shear walls 13 and 14, it was very important to construct a semi-rigid diaphragm model. The semi-rigid diaphragm constraint allows the diaphragm to develop stresses and deform with respect to itself, which in turn would enable the checking of stresses at this critical section. In contrast, the rigid diaphragm disregards the stiffness properties of the floor diaphragms, rather considering them rigid bodies, and therefore reports no stresses in the floor diaphragms.

However, the author is significantly more familiar with rigid diaphragm behavior, and thus the rigid diaphragm model was built first to enable some verification of the model's accuracy prior to proceeding to semi-rigid diaphragms. Only two changes were required to transform the model from rigid to semi-rigid diaphragm. First, the rigid diaphragm constraints had to be removed. Then, in order to allow the diaphragm to deform appropriately, area elements with section and material properties had to be assigned to the floor diaphragms and meshed into structural elements with a maximum size of 48" (in comparison, the diaphragms in the rigid diaphragm model were assigned a meshing that deliberately disregards the stiffness of the diaphragms).

Two sub-models had to be built for both rigid and semi-rigid diaphragms. The wall pier function was used to easily report forces in the shear walls at all levels. However, it had to be taken into consideration that shear walls in groups report output forces differently. For shear design, it was important to determine the shear in each individual wall, and therefore each wall was assigned its own individual pier label. The labels given to these walls as well as the pier axes (which were important in interpreting the results given by the pier output) can be seen in Figure 23. Conversely, grouped shear walls resist moments as a group, and therefore to accurately report this behavior, each group was assigned a single wall pier label. These groups and their axes can be seen in Figure 24. As walls cannot be assigned more than one pier label, the shear results and moment results had to be obtained from separate models.

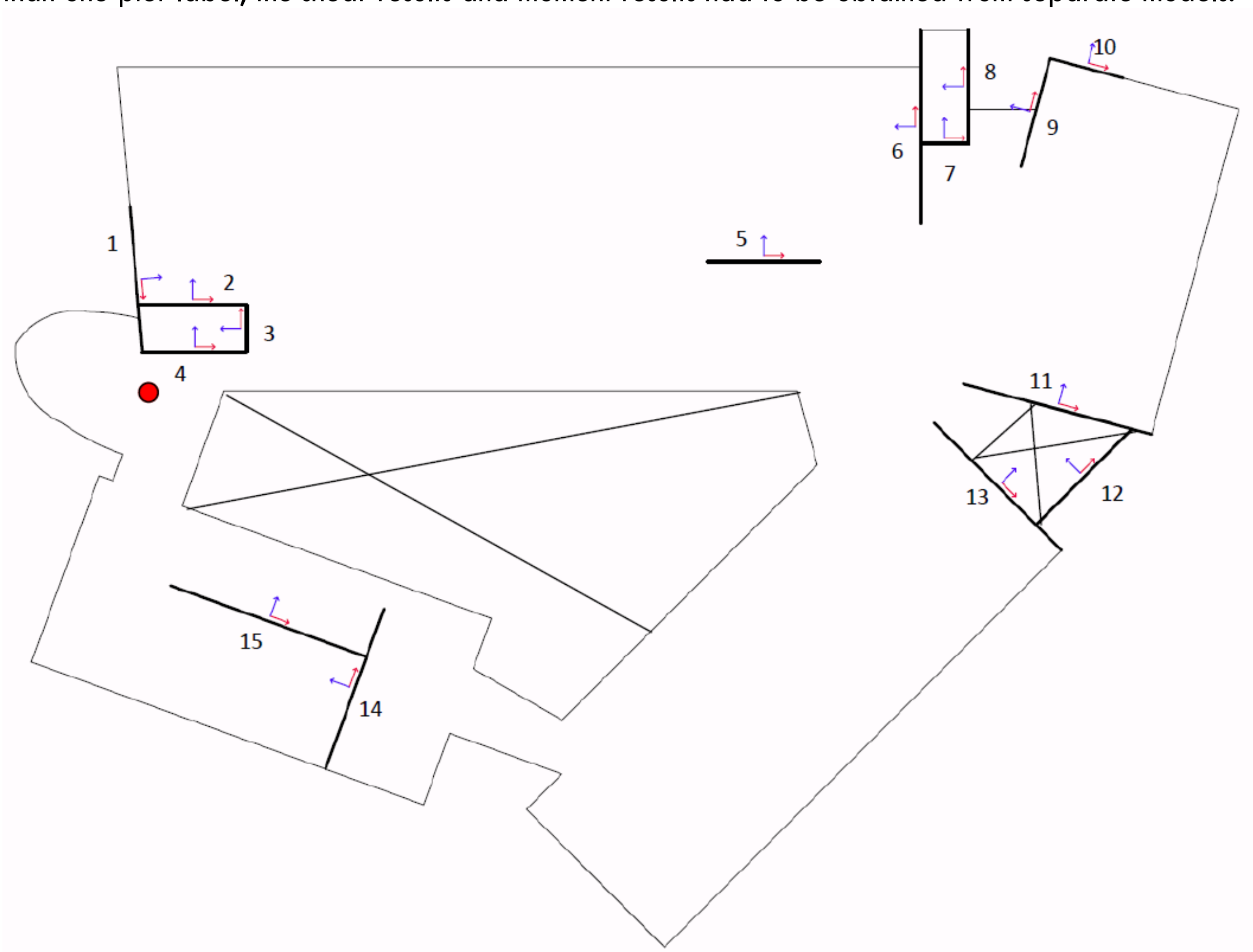


Figure 23 Floor plan showing shear wall pier labels and axes used to obtain shear results. The red axis corresponds to ETABS' "V2" (strong) axis, and the blue axis corresponds to ETABS' "V3" (weak) axis. Red dot is considered (0,0) ft location.

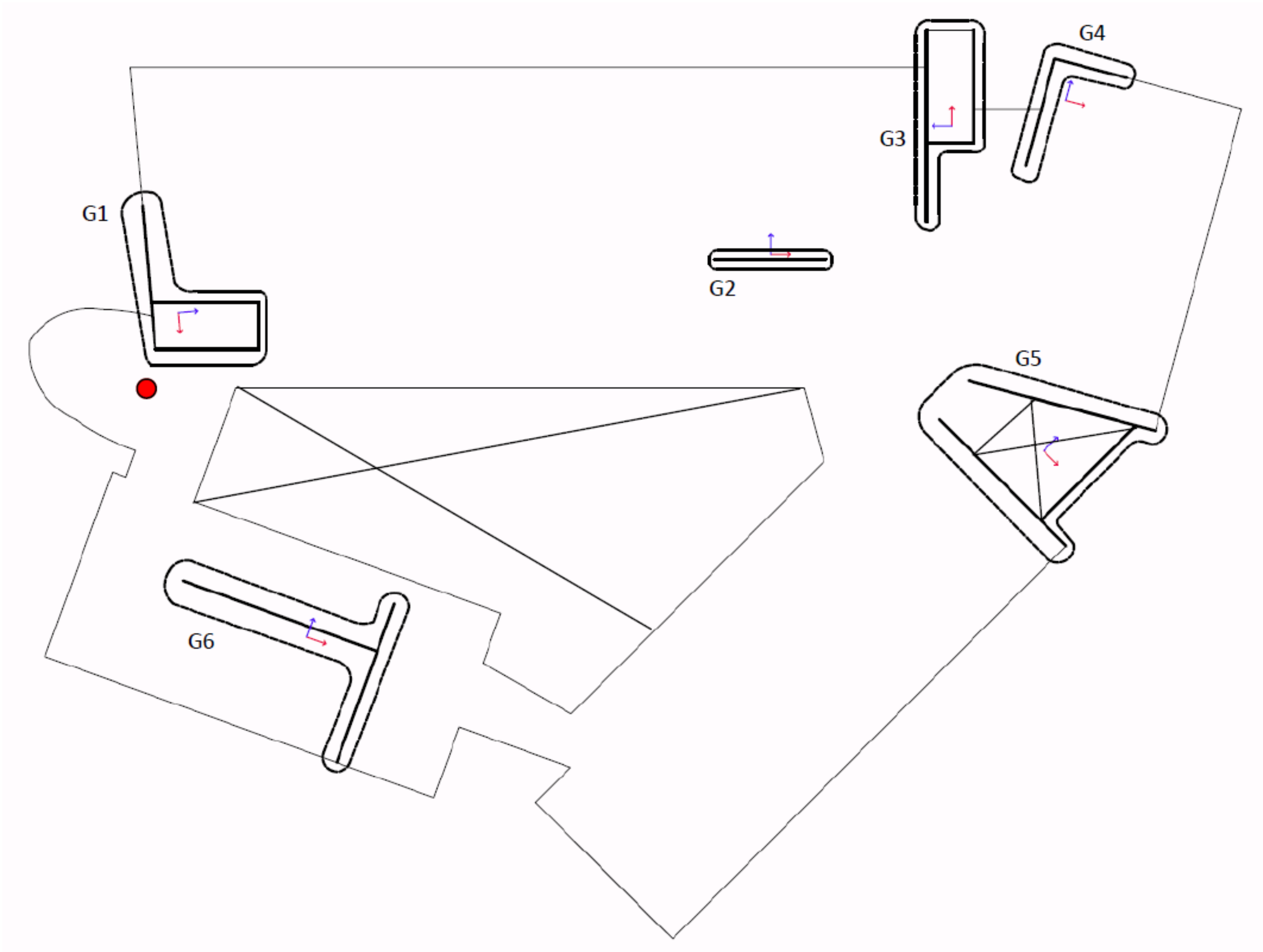


Figure 24 Floor plan showing shear wall pier labels and axes used to obtain moment results. The red axis corresponds to ETABS' "V2" axis, and the blue axis corresponds to ETABS' "V3" axis. Red dot is considered (0,0) ft location.

Building Properties

In order to produce the most accurate model possible, the center of mass and the center of rigidity were both calculated by hand and then compared to the values given by the rigid diaphragm model in ETABS. The center of mass is the location where all mass could be considered effectively lumped and it would produce a nearly identical effect as the distributed masses of the real building. As such, it should be (and was found to be) the same in both the rigid and semi-rigid models.

The center of rigidity is the location at which an applied horizontal load would produce no torsion in a rigid floor diaphragm. However, since a semi-rigid diaphragm is capable of experiencing local deformations, the center of rigidity has no meaning in the semi-rigid model and thus was not documented. The results of this comparison are summarized in Table 6 and Figure 25.

Rigid Diaphragm Model - COM & COR Coordinates								
Story	ETABS				Hand Calculations			
	X_{COM} (ft)	Y_{COM} (ft)	X_{COR} (ft)	Y_{COR} (ft)	X_{COM} (ft)	Y_{COM} (ft)	X_{COR} (ft)	Y_{COR} (ft)
2nd	96.60	0.27	108.99	-4.90	91.06	-0.30	107.79	-2.99
3rd	96.60	0.27	108.95	-18.06	91.21	-0.39	107.39	-4.53
4th	96.60	0.27	111.26	-24.94	91.21	-0.39	107.05	-5.98
5th	98.73	4.24	113.46	-28.31	98.34	-2.63	106.84	-7.20
Penthouse	97.92	30.91	115.75	-28.62	95.15	30.47	128.29	8.31
Atrium Roof	102.66	-39.30	135.11	-31.06	97.83	-40.09	128.48	8.02
Chiller Roof	180.70	25.88	149.15	-14.91	177.57	26.87	169.54	43.26
AHU Mech. Rm. Roof	85.28	25.26	116.98	-27.15	84.90	25.68	124.59	2.18

Table 6 Summary of center of mass and center of rigidity locations as found in ETABS and via hand calculation.

Figure 25 2nd Level floor plan showing the locations of COM and COR as found by ETABS and via hand calculation.

The center of mass was found by breaking up the building into representative areas, and then using a spreadsheet to find the weight of each area. The square footage of each area and the individual area centroid locations were found using AutoCAD. Figure 26 shows the area labels used, and Table 7 shows a typical level's center of mass calculation.

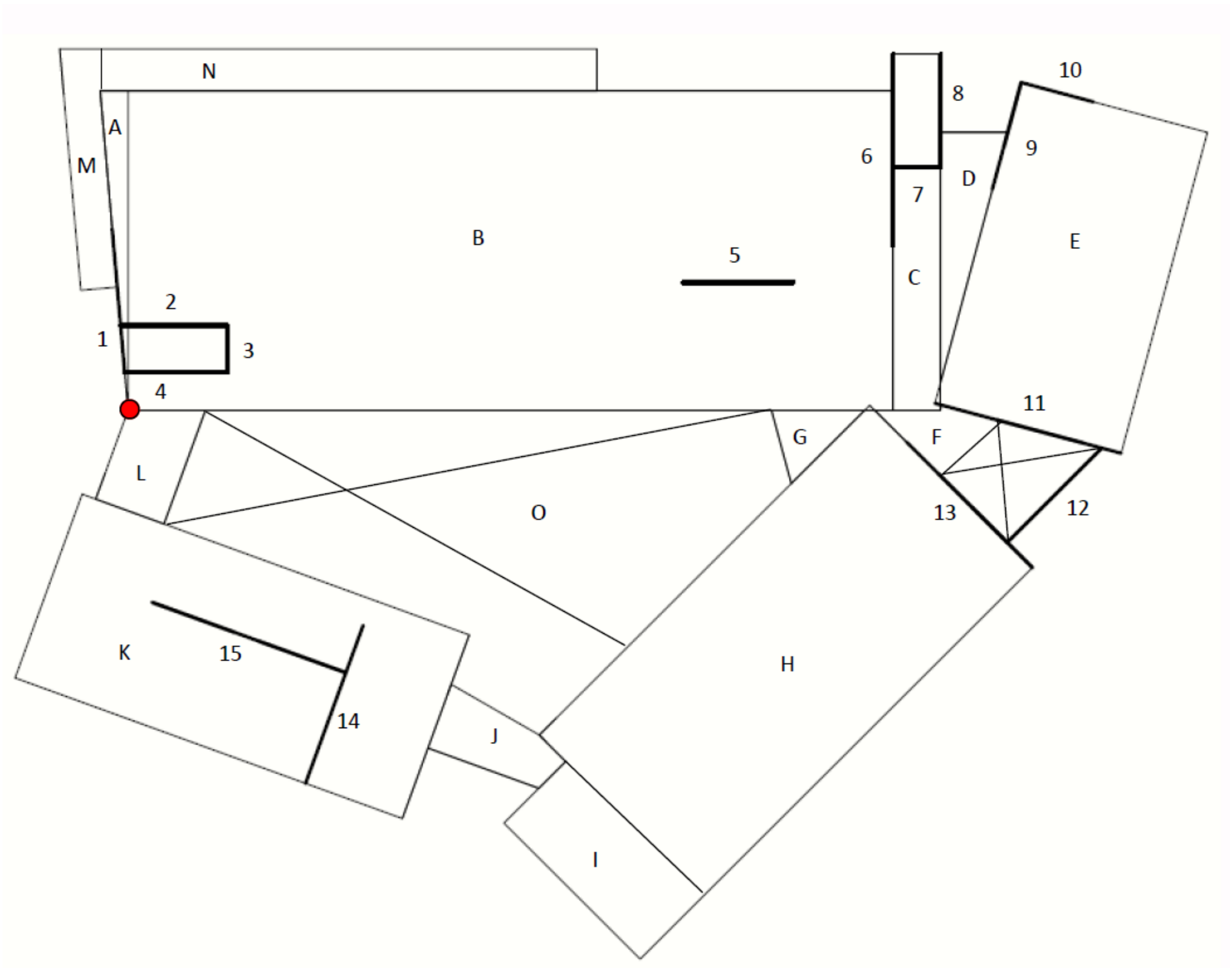


Figure 26 Floor plan showing the areas and area labels used for the calculation of the center of mass. Areas are labeled with letters, shear walls are numbered.

Level 2 Weight and Center of Mass							
Area/Item Description		Quantity	DL	SDL	Total Weight (K)	CoM X (ft)	CoM Y (ft)
Shear Walls	1	22.45 LF	875 plf	0 plf	19.64	-1.82	20.83
	2	23.24 LF	875 plf	0 plf	20.34	8.71	16.08
	3	8.83 LF	875 plf	0 plf	7.73	18.83	11.67
	4	19.47 LF	875 plf	0 plf	17.04	9.10	7.25
	5	15.24 LF	875 plf	0 plf	13.34	112.62	24.25
	6	34.50 LF	875 plf	0 plf	30.19	144.83	49.42
	7	9.00 LF	875 plf	0 plf	7.88	149.33	46.25
	8	21.33 LF	875 plf	0 plf	18.66	153.83	60.13
	9	21.00 LF	875 plf	0 plf	18.38	166.33	52.04
	10	14.17 LF	875 plf	0 plf	12.40	175.90	60.34
	11	20.58 LF	875 plf	0 plf	18.01	174.41	-4.48
	12	25.35 LF	875 plf	0 plf	22.18	175.39	-16.11
	13	25.17 LF	875 plf	0 plf	22.02	162.30	-20.95
	14	31.92 LF	875 plf	0 plf	27.93	39.04	-55.76
	15	34.00 LF	875 plf	0 plf	29.75	22.90	-43.09
Floor Areas	A	160.5 SF	127 psf	15 psf	22.79	-1.77	40.39
	B	8774.6 SF	127 psf	15 psf	1245.99	72.42	30.29
	C	416.3 SF	127 psf	15 psf	59.11	149.33	23.13
	D	299.6 SF	127 psf	15 psf	42.54	158.06	36.90
	E	2299.5 SF	127 psf	15 psf	326.53	177.91	27.19
	F	143.2 SF	127 psf	15 psf	20.33	154.18	-3.88
	G	122.0 SF	127 psf	15 psf	17.32	130.61	-5.08
	H	3855.7 SF	127 psf	15 psf	547.51	124.51	-45.71
	I	603.4 SF	127 psf	15 psf	85.68	85.34	-89.89
	J	253.3 SF	120 psf	15 psf	34.20	69.96	-63.24
	K	2892.5 SF	100 psf	35 psf	390.49	21.60	-46.66
	L	280.9 SF	120 psf	15 psf	37.92	3.83	-9.59
	M	0.0 SF	0 psf	0 psf	0.00	-7.94	51.50
	N	0.0 SF	0 psf	0 psf	0.00	41.70	64.58
	O	0.0 SF	0 psf	0 psf	0.00	58.54	-20.27
Total Weight=					3115.90 k		
Total Mass=					8.06 k-s^2/in		
Level 2 Center of Mass, X-Coordinate=						91.06 ft	
Level 2 Center of Mass, Y-Coordinate=						-0.30 ft	

Table 7 Center of mass calculation for 2nd Level. See Figure 26 for area and shear wall labels.

The center of rigidity was calculated using the individual stiffnesses of the walls. This process proved to be significantly complicated by the fact that the loads were to be applied at axes that were not parallel or perpendicular to all of the walls. When all walls lie on the same two axes as the applied loads, it can be assumed that the walls have no stiffness in out-of-plane bending/shear, and their stiffness for in-plane bending/shear can be found by applying a unit load to the wall and then using the relationship of P=KΔ.

However, the stiffness of any shear wall can be found with the following equation, which accounts for both flexural and shear deformations:

$$K = \frac{1}{\frac{h^3}{12EI_{Gi}} + \frac{1.2h}{A_i G}}$$

In this equation, h is the height of the wall (measured from the base), E is the modulus of elasticity, G is the shear modulus, Ai is the shear area in the direction under consideration of the individual wall, and I_{Gi} is the moment of the inertia in the direction under consideration of the group the wall is in.

The area was originally calculated with length of the wall times the thickness of the wall. Then, it was resolved onto the N-S Direction (also referred to as the x-direction) and the E-W Direction (also referred to as the y-direction) using sine and cosine of the angle of the wall with respect to the N-S axis (x axis).

The moments of inertia of the walls lying on the N-S or E-W axes were simply calculated with the traditional moment of inertia of a rectangle formula, and then the parallel axis theorem was used to find the moment of inertia of the wall about the centroid of the group. However, for walls at oblique angles, the following formulas had to be used for their own moment of inertias, and the parallel axis theorem was used to find the moment of inertia of the wall about the centroid of the group.

$$I_{x,i} = \frac{bh^3 \cos \propto}{12}$$

$$I_{y,i} = \frac{bh^3 \sin \propto}{12}$$

In these formulas, α is the angle between the x-axis and the wall, and a positive angle was considered to be counter-clockwise. Finally, all moments of inertia and parallel axis theorem values for the walls in a given group were added to find the moment of inertia of the group about its centroid.

Once all stiffnesses were found, the following equations were used to find the coordinates of the center of rigidity on a given level.

$$X_{COR} = \frac{\Sigma K_{y,i} x_i}{\Sigma K_{y,i}}$$

$$Y_{COR} = \frac{\Sigma K_{x,i} y_i}{\Sigma K_{x,i}}$$

A sample calculation of the center of rigidity has been included in Tables 8 and 9. Table 8 deals with the individual shear walls, whereas Table 9 combines this into group data and finds the center of rigidity.

2nd Level Wall Stiffnesses and Center of Rigidity Calculation - Wall Data											
Height (in)	Wall	t (in)	L (in)	α (deg)	X_0 (in)	Y_0 (in)	E (ksi)	v	G (ksi)	Ax (in^2)	Ay (in^2)
182	1	12	278	95	-21.84	249.96	1,912.00	0.20	796.67	3,336.00	3,336.00
	2	12	240	0	104.52	192.96	1,912.00	0.20	796.67	2,880.00	2,880.00
	3	12	108	90	225.96	140.04	1,912.00	0.20	796.67	1,296.00	1,296.00
	4	12	228	0	109.20	87.00	1,912.00	0.20	796.67	2,736.00	2,736.00
	5	12	183	0	1,351.44	291.00	1,912.00	0.20	796.67	2,196.00	2,196.00
	6	12	414	90	1,737.96	593.04	1,912.00	0.20	796.67	4,968.00	4,968.00
	7	12	108	0	1,791.96	555.00	1,912.00	0.20	796.67	1,296.00	1,296.00
	8	12	256	90	1,845.84	721.56	1,912.00	0.20	796.67	3,072.00	3,072.00
	9	12	252	75	1,995.96	624.48	1,912.00	0.20	796.67	3,024.00	3,024.00
	10	12	170	-15	2,110.80	724.08	1,912.00	0.20	796.67	2,040.00	2,040.00
	11	12	202	-15	2,092.92	-53.76	1,912.00	0.20	796.67	2,424.00	2,424.00
	12	12	288	45	2,104.68	-193.32	1,912.00	0.20	796.67	3,456.00	3,456.00
	13	12	302	-45	1,947.60	-251.40	1,912.00	0.20	796.67	3,624.00	3,624.00
	14	12	383	70	468.48	-669.12	1,912.00	0.20	796.67	4,596.00	4,596.00
	15	12	408	-20	274.80	-517.08	1,912.00	0.20	796.67	4,896.00	4,896.00

Table 8 Wall data used for center of rigidity calculation for 2nd Level.

2nd Level Wall Stiffnesses and Center of Rigidity Calculation - Group Data										
Group	$X_{COM,Gi}$ (in)	$Y_{COM,Gi}$ (in)	$I_{y,0,i}$ (in^4)	$I_{y,i,PAT}$ (in^4)	I_y (in^4)	$I_{x,0,i}$ (in^4)	$I_{x,i,PAT}$ (in^4)	I_x (in^4)	K_x (k/in)	K_y (k/in)
1	79.99	176.53	163,202.35	34,594,614.22	92,147,823.68	21,321,749.65	17,985,885.37	65,070,278.58	1,057.39	11,556.85
1			13,824,000.00	1,732,443.34		34,560.00	777,111.12		10,199.95	0.00
1			34,560.00	27,612,810.81		1,259,712.00	1,725,980.11		0.00	4,638.92
1			11,852,352.00	2,333,840.97		32,832.00	21,932,448.33		9,704.06	0.00
2	1,351.44	291.00	6,128,487.00	0.00	6,128,487.00	26,352.00	0.00	26,352.00	5,962.64	0.00
3	1,780.95	630.05	26,352.00	9,183,240.14	23,575,584.69	70,957,944.00	6,804,405.62	127,580,478.11	0.00	17,469.96
3			1,259,712.00	156,989.03		15,552.00	7,299,478.20		4,490.86	0.00
3			15,552.00	12,933,739.53		16,777,216.00	25,725,882.29		0.00	10,953.08
4	2,042.22	664.60	1,071,998.27	6,472,038.55	21,721,773.60	14,931,009.73	4,868,255.96	27,344,847.84	2,759.67	9,665.36
4			4,583,891.40	9,593,845.38		329,108.60	7,216,473.55		6,612.87	1,890.98
5	2,041.78	-179.87	7,690,271.36	6,338,509.59	85,563,267.98	552,136.64	38,551,758.22	83,986,066.48	8,322.56	2,272.24
5			11,943,936.00	13,671,640.30		11,943,936.00	625,030.02		8,676.69	8,672.35
5			13,771,804.00	32,147,106.73		13,771,804.00	18,541,401.60		9,086.70	11,957.48
6	368.58	-590.70	6,572,032.33	45,868,752.85	155,471,448.42	49,609,854.67	28,265,918.41	112,354,527.48	5,678.94	15,194.18
6			59,972,495.72	43,058,167.51		7,944,816.28	26,533,938.12		16,319.43	6,022.23
								Sum=	88,871.77	100,293.63
								X_{COR}=	1293.48 in	
								Y_{COR}=	-35.92 in	

Table 9 Group data used for center of rigidity calculation for 2nd Level.

Typically, this stiffness data could also be used to replicate the wall shears found in ETABS using a proportional distribution of direct and torsion-induced shear according to the following equations.

$$V_{Direct,i} = \frac{K_i}{\Sigma K_i} V$$

$$V_{Torsion,i} = \frac{eK_i d_i}{J} V$$

Where e is the eccentricity with respect to the center of mass (seismic) or center of pressure (wind) at which the story shear (V) will be applied and d is the distance to the line of resistance where wall "i" is located. J can be found by summing the product of the stiffness of a wall and its d-value squared. This calculation was attempted, but did not yield forces similar to those found in ETABS. It seems likely this is due to the complexity of the building. However, since the centers of rigidity of the building were able to be replicated within a reasonable margin of error, it seems as though the model can be considered accurate.

Upon verifying the model was approximately accurate, modal information was gathered and seismic forces were calculated using the Modal Response Spectrum Analysis (MRSA) method. The modal information used in these calculations can be found in Tables 10 and 11. It was expected that the semi-rigid diaphragm model would have periods that are slightly higher than the rigid diaphragm model, as the semi-rigid diaphragm is a slightly less stiff structure. As can be seen in the tables, this is the case. It is also of interest that the second and third modes of the semi-rigid diaphragm are both Y-translational modes (whereas typically the third mode would be a Z-rotational mode). Both modes were animated in the ETABS model to determine the cause of this, and it was found that the plan-southwest wing (the Office Wing) of the structure moves separately from the plan-north and plan-west wings. This indicates the link sections are insufficient to cause the building to behave as a rigid structure, further verifying the need for the semi-rigid diaphragm model as a check on the forces in these links.

Rigid Diaphragm Model - Modal Information							
Mode	T(s)	UX%	UY%	RX%	RY%	RZ%	$C_{S,MAX}$
1	0.7997	42.03	5.54	7.52	60.36	25.73	0.0300
2	0.5697	6.76	65.46	89.30	9.04	0.53	0.0421
3	0.4040	25.13	1.10	1.65	29.45	47.89	0.0594
4	0.1831	0.00	0.23	0.00	0.00	0.03	0.1311
5	0.1748	6.80	6.16	0.52	0.49	5.48	0.1373
6	0.1446	6.88	11.28	0.72	0.37	0.12	0.1660
7	0.1243	4.89	0.94	0.04	0.18	11.72	0.1930
8	0.0935	1.29	2.47	0.08	0.05	1.49	0.2567
9	0.0803	0.29	3.60	0.11	0.01	0.57	0.2987
10	0.0766	1.35	0.02	0.00	0.02	0.45	0.3133
11	0.0724	2.33	0.02	0.00	0.02	2.86	0.3315
12	0.0690	0.29	0.00	0.00	0.00	0.06	0.3477
Totals	N/A	98.05	96.81	99.95	99.99	96.94	N/A
$C_{m,X}=SQRT(\Sigma(C_{m,i}*UX\%)^2)=$							0.0281
$C_{m,Y}=SQRT(\Sigma(C_{m,i}*UY\%)^2)=$							0.0367

Note: "X" Direction corresponds to the N-S Direction, "Y" Direction corresponds to E-W Direction, and "Z" Direction corresponds to the Vertical Direction

Table 10 Modal periods and mass participation factors for the Rigid Diaphragm model.

Semi-Rigid Diaphragm Model - Modal Information							
Mode	T(s)	UX%	UY%	RX%	RY%	RZ%	$C_{S,MAX}$
1	0.8383	39.08	7.37	10.70	56.16	23.31	0.0286
2	0.6243	11.25	40.96	61.33	15.65	1.49	0.0384
3	0.5601	0.85	16.86	18.74	1.20	2.57	0.0428
4	0.4423	18.72	4.85	6.11	23.69	43.90	0.0543
5	0.3067	0.30	6.28	1.88	0.32	2.23	0.0782
6	0.2858	0.45	0.12	0.02	0.13	0.76	0.0840
7	0.2749	0.87	0.28	0.08	0.46	0.09	0.0873
8	0.2299	1.12	2.68	0.31	0.38	2.06	0.1044
9	0.2042	0.33	0.14	0.04	0.02	1.34	0.1175
10	0.1981	12.33	0.42	0.00	1.20	0.63	0.1212
11	0.1727	2.30	0.32	0.00	0.34	5.28	0.1389
12	0.1651	0.10	0.27	0.00	0.00	0.00	0.1454
13	0.1638	0.00	9.86	0.44	0.01	0.40	0.1465
14	0.1544	0.16	1.13	0.04	0.04	5.20	0.1554
15	0.1497	2.16	0.69	0.03	0.13	0.01	0.1604
16	0.1469	0.02	0.09	0.00	0.00	0.08	0.1634
17	0.1409	0.37	0.07	0.00	0.01	0.25	0.1704
18	0.1310	1.41	0.13	0.00	0.11	0.32	0.1832
Totals	N/A	91.83	92.50	99.72	99.84	89.90	N/A
$C_{m,X}$=SQRT(Σ($C_{m,i}$*UX%)^2)=							0.0224
$C_{m,Y}$=SQRT(Σ($C_{m,i}$*UY%)^2)=							0.0236

Note: "X" Direction corresponds to the N-S Direction, "Y" Direction corresponds to E-W Direction, and "Z" Direction corresponds to the Vertical Direction

Table 11 Modal periods and mass participation factors for the Semi-Rigid Diaphragm model.

Comparison of Results and Shear Wall Capacities

Upon completing the models and verifying their accuracy, maximum shear, moment and drift values were pulled from ETABS for both the rigid and semi-rigid diaphragm models. As these forces could not be replicated by hand calculation, it was decided to instead verify the capacities of the walls to ensure they could carry the forces applied. The hand calculations related to these capacity checks can be found in Appendix C.

Shear capacity was found with the equation for the shear capacity of a structural wall resisting seismic forces. This is equation 21-7 in ACI 318-08, and is shown below.

$$V_n = A_{cv}(\propto_c \lambda(f_c')^{1/2} + \rho_t f_y)$$

The value of αc is dependent on the height-to-length ratio of the wall under consideration, and rho-transverse is found with the equation below.

$$\rho_t = \frac{A_{v,horiz}}{t_w s_{vert}}$$

All shear walls are provided with basic reinforcing of #5 rebar at 18" on-center in each face, each way. However, they also often contain boundary elements. These have little effect on shear, and therefore were disregarded in the calculation of rho-transverse. A summary of the values used to calculate the shear capacity of each wall as well as the highest shear demand found in ETABS for each wall can be found in Table 12.

Shear Wall Shear Capacities and Maximum Design Shears										
Wall	t_w (in)	L (in)	h_w (in)	A_s (in^2)	s (in)	α_c	ρ_t	ϕV_n (k)	$V_{u,rigid}$ (k)	$V_{u,semi}$ (k)
1	12	200	1131	0.62	18	2	0.00287	735.3271	224.46	294.65
2	12	240	1131	0.62	18	2	0.00287	882.3925	103.23	132.34
3	12	108	1131	0.62	18	2	0.00287	397.0766	42.95	65.56
4	12	228	1131	0.62	18	2	0.00287	838.2729	82.74	156.1
5	12	218	1131	0.62	18	2	0.00287	801.5066	92.71	83.92
6	12	414	1131	0.62	18	2	0.00287	1522.127	343.06	387.3
7	12	108	1131	0.62	18	2	0.00287	397.0766	53.12	81.07
8	12	256	1131	0.62	18	2	0.00287	941.2187	137.6	143.6
9	12	252	1029	0.62	18	2	0.00287	926.5122	234.47	284.19
10	12	170	1029	0.62	18	2	0.00287	625.0281	63.78	76.38
11	12	202	1131	0.62	18	2	0.00287	742.6804	169.55	314.36
12	12	288	1131	0.62	18	2	0.00287	1058.871	213.62	315.72
13	12	302	1131	0.62	18	2	0.00287	1110.344	100.65	164.1
14	12	383	686	0.62	18	2.58	0.00287	1587.663	310.73	87.63
15	12	408	686	0.62	18	2.36	0.00287	1619.205	237.69	153.36

Table 12 Shear capacity calculation for each shear wall and maximum shear demand as found in ETABS for both rigid and semi-rigid diaphragm models.

As can be seen in the table, the shear capacities of each wall far exceed the demand. This may be due to the fact that every earthquake load calculated for the building in this technical report was lower in magnitude than the design loads used in the original calculations. Lastly, it appeared based on the calculations in this technical report that the moment capacity of the walls was much more critical than the shear capacity, and therefore the walls may have been designed for this.

The moment capacities of the shear wall groups were calculated by hand using a simplified procedure recommended in "Reinforced Concrete Mechanics & Design" by James K. Wight and James G. MacGregor. These calculations can be found in Appendix C. The required moments were taken from ETABS for each wall group. M_2 corresponds to the moment about the "2" axis for the wall group (shown in red in Figure 24) and M_3 corresponds to the moment about the "3" axis for the wall group (shown in blue in Figure 24). The required moments and the calculated capacities are summarized in Table 13.

Shear Wall Group Moment Capacities and Maximum Design Moments							
Moment		Wall Group Moments (k-ft)					
		Group 1	Group 2	Group 3	Group 4	Group 5	Group 6
Rigid	(+)M_{u2}	7,188.52	0.00	3,692.82	3,395.05	6,642.36	5,845.30
	(-)M_{u2}	-9,786.77	0.00	-3,058.20	-2,414.78	-6,245.99	-5,233.40
	(+)M_{u3}	12,172.82	1,317.11	13,877.09	924.55	7,387.51	5,954.13
	(-)M_{u3}	-15,584.32	-1,404.06	-18,985.02	-4,461.07	-9,260.64	-11,501.80
Semi-Rigid	(+)M_{u2}	3,215.48	0.00	-1,814.82	3,942.75	7,228.58	1,367.49
	(-)M_{u2}	-14,629.03	0.00	5,448.58	-1,347.69	-1,573.00	-337.44
	(+)M_{u3}	9,973.94	1,471.36	11,115.09	-1,405.68	4,346.47	-3,780.93
	(-)M_{u3}	-21,302.29	-1,934.24	-31,718.11	-8,137.52	-13,140.73	-17,342.70
Hand Calc.s	(+)ϕM_{n2}	13,013.00	0.00	5,122.00	6,634.00	13,838.00	17,617.00
	(-)ϕM_{n2}	-18,464.00	0.00	-7,837.00	-4,923.00	-15,240.00	-22,763.00
	(+)ϕM_{n3}	13,220.00	2,457.00	14,809.00	6,643.00	15,582.00	12,898.00
	(-)ϕM_{n3}	-19,781.00	-2,457.00	-35,064.00	-4,475.00	-16,394.00	-22,950.00

Table 13 Moment capacities for each shear wall group and maximum moment demand as found in ETABS for both rigid and semi-rigid diaphragm models.

All moment capacities are sufficient for Rigid Diaphragm moment demands, but the capacity of Group 1 and Group 4 is not high enough for the Semi-Rigid moment demands. This was assumed to be due to the fact that the simplified procedure used to calculate capacity does not account for the boundary elements, which would greatly increase the moment capacity of these wall groups.

The last major check performed on the building was for relative displacements (that is, displacements of one level with respect to the level below it) and the subsequent story drifts. It was attempted to replicate the maximum displacement of the AHU Mechanical Room Roof in both x- and y-directions using the story shears experienced by each wall for the controlling load combination divided by the stiffness of the wall. This calculation can be found in Appendix C. It was found that the values could not be replicated because (similarly to the attempt to replicate shears in the walls) the calculated stiffnesses of the walls are sufficiently similar to the stiffness used by ETABS to be used in a ratio, but are not sufficiently similar to the stiffness used by ETABS to be used in further calculation.

Relative displacements and drifts as found in ETABS for both the rigid and semi-rigid diaphragm models are summarized in Table 14. These drifts were compared to the typical allowable drift value of L/50 (2% of the story height, per ASCE 7-05 Table 12.12-1). Also, all drifts resulting from modal analysis must also be modified by a factor of Cd/I. For this building, Cd is 5 and I is 1.25. This factor is incorporated into the drifts listed in Table 14.

All of the rigid diaphragm drifts were shown to be more than sufficient. However, several of the semi-rigid diaphragm drifts exceeded this serviceability limit. It is of note that this is not an indication of failure, as it is likely that the behavior of the structure is much closer to that of a rigid diaphragm than a semi-rigid due to the thickness of the slabs. The semi-rigid was largely considered solely for the purposes of determining diaphragm forces at the link. Both the shear and moment demands on the lateral system are similar for rigid and semi-rigid models, thus indicating that it is unlikely that the structural behavior of the models is not that different from a strength perspective.

Maximum Relative Story Displacements and Drifts					
Level		Direction	$\Delta_{MAX,rel}$ (in)	Drift	L/?
Rigid Diaphragm	2nd	X	0.0318	0.000699	1431.27
		Y	0.0359	0.000789	1267.27
	3rd	X	0.0570	0.001356	737.46
		Y	0.0647	0.001540	649.35
	4th	X	0.0768	0.001828	547.05
		Y	0.0892	0.002124	470.81
	5th	X	0.0906	0.002156	463.82
		Y	0.1079	0.002568	389.41
	Penthouse	X	0.1034	0.002364	423.01
		Y	0.1227	0.002804	356.63
	Atrium Roof	X	0.1197	0.005568	179.60
		Y	0.0705	0.003280	304.88
	Chiller Roof	X	0.0601	0.002932	341.06
		Y	0.0580	0.002828	353.61
	AHU Mech. Room Roof	X	0.1355	0.005312	188.25
		Y	0.0719	0.002820	354.61
Semi-Rigid Diaphragm	2nd	X	0.1533	0.003370	296.73
		Y	0.0621	0.001364	732.88
	3rd	X	0.1443	0.003436	291.04
		Y	0.0773	0.001840	543.48
	4th	X	0.2418	0.005756	173.73
		Y	0.1070	0.002548	392.46
	5th	X	0.8588	0.020448	48.90
		Y	0.1374	0.003272	305.62
	Penthouse	X	1.6186	0.036996	27.03
		Y	0.7502	0.017148	58.32
	Atrium Roof	X	14.5089	0.674832	1.48
		Y	5.5905	0.260024	3.85
	Chiller Roof	X	5.1708	0.252236	3.96
		Y	5.5621	0.271324	3.69
	AHU Mech. Room Roof	X	14.2442	0.558596	1.79
		Y	0.6513	0.025540	39.15

Table 14 Maximum relative story displacements and subsequent drifts, as found from ETABS.

To finalize the structural analysis of the building, the force in the link element indicated as problematic were checked in the semi-rigid diaphragm model. These forces are defined at mid-depth of the shell element and are labeled as shown in Figure 27, which was taken from the CSI Analysis Reference Manual. They are forces per linear length of element. In all cases, F_{11} was checked, as it proved to be the largest force. These are summarized in Table 15.

The forces determined in ETABS were compared to the allowable shear in a concrete diaphragm resisting seismic loads, which can be calculated as:

$$\varphi V_n = \varphi A_{cv}(2\lambda\ f_c'^{\frac{1}{2}} + \rho_t f_y)$$

This was calculated for a 1 inch wide strip of the 12" thick slab and was also included in Table 15.

It can be seen in the table that these forces are extremely low, and therefore the link is more than adequate to carry the required loads.

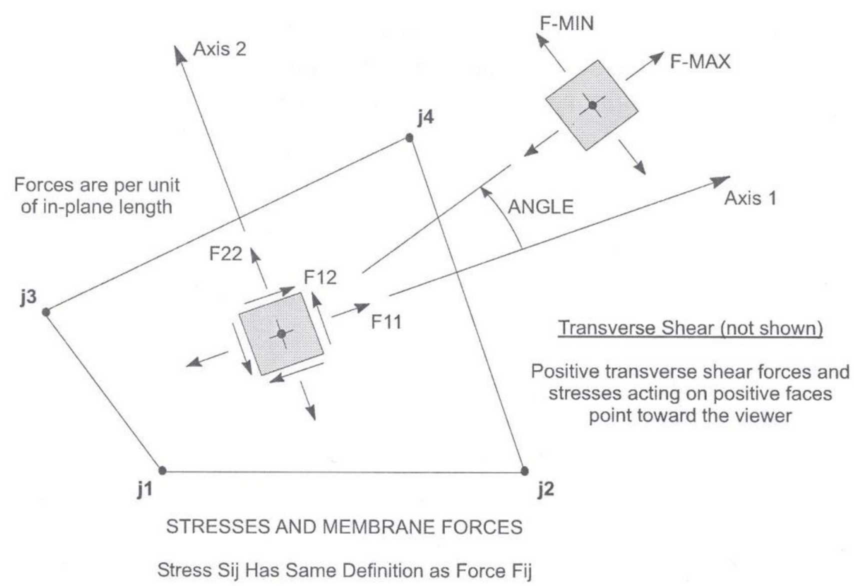


Figure 27 Diagram from the CSI Analysis Reference Manual displaying how ETABS reports shell forces.

Semi-Rigid Diaphragm Model - Link Forces					
Level	Link Forces (k/in) due to Indicated Load Case				Allowable Force (k/in)
	EX	EY	EMX	EMY	
2nd	0.3105	0.2259	0.0094	0.0243	8.38
3rd	0.7654	0.3004	0.0141	0.0293	
4th	1.3478	0.1027	0.0460	0.0618	
5th	0.3127	0.6767	0.0270	0.5505	

Table 15 Link forces due to the indicated load cases, as taken from ETABS.

Conclusions

Upon thorough analysis, the lateral system of the University Sciences Building (USB) was found to be sufficient to carry the forces it is likely to experience. This conclusion is based upon both hand calculations and finite element computer model analyses which were conducted for this technical report. The wind forces were found using the Main Wind Force Resisting System method, and the seismic forces were found first with the Equivalent Lateral Force method and then (when the finite element models, constructed in ETABS, had been found to be sufficiently accurate) with the Modal Response Spectrum Analysis method. It was found that seismic loads controlled by approximately 50%, but both wind and seismic load cases were considered in the lateral model (resulting in a total of 114 load combinations, included in Appendices A and B).

Two models were built to fully encompass the structural behavior of the building. One was a rigid diaphragm model, in which the shear walls were first individually assigned to piers for ease of reporting shear forces, and then assigned in groups to piers for ease of reporting moments in the shear wall groups. The other was a semi-rigid diaphragm model, and was built with identical pier labeling as the rigid diaphragm model. A semi-rigid model was made necessary by the concern that the link section on the plan-south side may be subjected to extremely high diaphragm forces as it serves as the only connection between shear walls 13 and 14, and it has a significantly reduced cross-section.

Upon completion of the models, shear and moment demands for both models were compared to calculated capacities, as it was not possible to replicate the forces using the traditional lateral force distribution methods. It was found that the rigid vs. semi-rigid diaphragm assignment has a significant effect on the forces and moments in the walls, even though the walls were found to be largely adequate for the demand for both modeling assumptions. Wherever the calculated capacity was exceeded by the demand, it seemed likely that the simplifications made to calculate the capacities were at fault rather than the design of the walls themselves.

Dirft values were also compared to the industry standards for allowable horizontal drifts. It was found that the rigid diaphragm drifts are well below the required values, whereas the semi-rigid diaphragm drifts are at times alarmingly large. However, since the thick concrete slabs used in the USB are more likely to behave as rigid diaphragms, and the semi-rigid diaphragm was largely only modeled in order to report forces in the diaphragms, this was determined to be a negligible result.

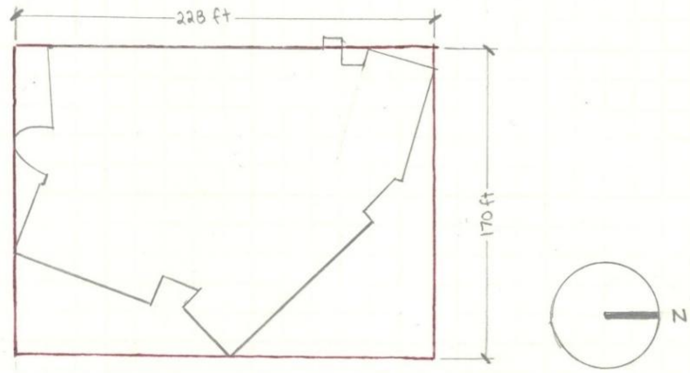
Finally, the semi-rigid diaphragm model was used to check forces in the link that was noted as a concern. These were found to be far below the capacity of the slab acting as a diaphragm, and therefore the link section was determined to be adequate to carry the diaphragm forces to which it may be subjected.

Appendices

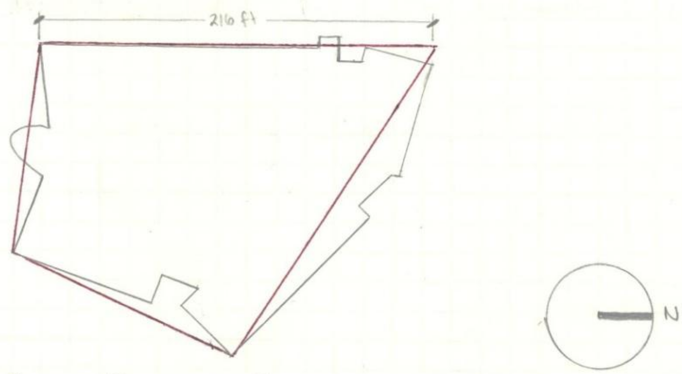
Appendix A: Wind Load Calculations

WIND ANALYSIS | TECH 1 | PG 1 OF 4

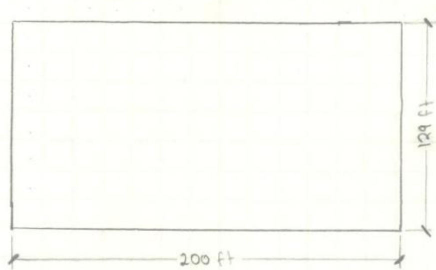
SIMPLIFYING ASSUMPTIONS



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 →

$$\left.\begin{aligned} q_z &= 0.00256\, K_z K_{zt} K_d V^2 I \\ q_h &= 0.00256\, K_h K_{zt} K_d V^2 I \end{aligned}\right\rangle$$ 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.

$g_Q = g_v = 3.4$

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

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

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

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

NORTH-SOUTH

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

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

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

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

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

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

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

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

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

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

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

$$= \boxed{0.846}$$

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)

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

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

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

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

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

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

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

$$= \boxed{0.828}$$

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

Wind Load Combinations Used in Modelling

ASCE 7 Chapter 2 Combination #3			ASCE 7 Chapter 2 Combination #4			ASCE 7 Chapter 2 Combination #6		
	Case 1	1.2D+1.6Lr+0.8WX		Case 1	1.2D+0.5Lr+L+1.6WX		Case 1	0.9D+1.6WX
		1.2D+1.6Lr+0.8WY			1.2D+0.5Lr+L+1.6WY			0.9D+1.6WY
		1.2D+1.6Lr-0.8WX			1.2D+0.5Lr+L-1.6WX			0.9D-1.6WX
		1.2D+1.6Lr-0.8WY			1.2D+0.5Lr+L-1.6WY			0.9D-1.6WY
	Case 2	1.2D+1.6Lr+0.6WX+0.6WMX		Case 2	1.2D+0.5Lr+L+1.2WX+1.2WMX		Case 2	0.9D+1.2WX+1.2WMX
		1.2D+1.6Lr+0.6WY+0.6WMY			1.2D+0.5Lr+L+1.2WY+1.2WMY			0.9D+1.2WY+1.2WMY
		1.2D+1.6Lr+0.6WX-0.6WMX			1.2D+0.5Lr+L+1.2WX-1.2WMX			0.9D+1.2WX-1.2WMX
		1.2D+1.6Lr+0.6WY-0.6WMY			1.2D+0.5Lr+L+1.2WY-1.2WMY			0.9D+1.2WY-1.2WMY
		1.2D+1.6Lr-0.6WX+0.6WMX			1.2D+0.5Lr+L-1.2WX+1.2WMX			0.9D-1.2WX+1.2WMX
		1.2D+1.6Lr-0.6WY+0.6WMY			1.2D+0.5Lr+L-1.2WY+1.2WMY			0.9D-1.2WY+1.2WMY
		1.2D+1.6Lr-0.6WX-0.6WMX			1.2D+0.5Lr+L-1.2WX-1.2WMX			0.9D-1.2WX-1.2WMX
		1.2D+1.6Lr-0.6WY-0.6WMY			1.2D+0.5Lr+L-1.2WY-1.2WMY			0.9D-1.2WY-1.2WMY
	Case 3	1.2D+1.6Lr+0.6WX+0.6WY		Case 3	1.2D+0.5Lr+L+1.2WX+1.2WY		Case 3	0.9D+1.2WX+1.2WY
		1.2D+1.6Lr-0.6WX-0.6WY			1.2D+0.5Lr+L-1.2WX-1.2WY			0.9D-1.2WX-1.2WY
	Case 4	1.2D+1.6Lr+0.45WX+0.45WY+0.45WMX+0.45WMY		Case 4	1.2D+0.5Lr+L+0.9WX+0.9WY+0.9WMX+0.9WMY		Case 4	0.9D+0.9WX+0.9WY+0.9WMX+0.9WMY
		1.2D+1.6Lr+0.45WX+0.45WY+0.45WMX-0.45WMY			1.2D+0.5Lr+L+0.9WX+0.9WY+0.9WMX-0.9WMY			0.9D+0.9WX+0.9WY+0.9WMX-0.9WMY
		1.2D+1.6Lr+0.45WX+0.45WY-0.45WMX+0.45WMY			1.2D+0.5Lr+L+0.9WX+0.9WY-0.9WMX+0.9WMY			0.9D+0.9WX+0.9WY-0.9WMX+0.9WMY
		1.2D+1.6Lr+0.45WX+0.45WY-0.45WMX-0.45WMY			1.2D+0.5Lr+L+0.9WX+0.9WY-0.9WMX-0.9WMY			0.9D+0.9WX+0.9WY-0.9WMX-0.9WMY
		1.2D+1.6Lr+0.45WX-0.45WY+0.45WMX+0.45WMY			1.2D+0.5Lr+L+0.9WX-0.9WY+0.9WMX+0.9WMY			0.9D+0.9WX-0.9WY+0.9WMX+0.9WMY
		1.2D+1.6Lr+0.45WX-0.45WY+0.45WMX-0.45WMY			1.2D+0.5Lr+L+0.9WX-0.9WY+0.9WMX-0.9WMY			0.9D+0.9WX-0.9WY+0.9WMX-0.9WMY
		1.2D+1.6Lr+0.45WX-0.45WY-0.45WMX+0.45WMY			1.2D+0.5Lr+L+0.9WX-0.9WY-0.9WMX+0.9WMY			0.9D+0.9WX-0.9WY-0.9WMX+0.9WMY
		1.2D+1.6Lr+0.45WX-0.45WY-0.45WMX-0.45WMY			1.2D+0.5Lr+L+0.9WX-0.9WY-0.9WMX-0.9WMY			0.9D+0.9WX-0.9WY-0.9WMX-0.9WMY
		1.2D+1.6Lr-0.45WX+0.45WY+0.45WMX+0.45WMY			1.2D+0.5Lr+L-0.9WX+0.9WY+0.9WMX+0.9WMY			0.9D-0.9WX+0.9WY+0.9WMX+0.9WMY
		1.2D+1.6Lr-0.45WX+0.45WY+0.45WMX-0.45WMY			1.2D+0.5Lr+L-0.9WX+0.9WY+0.9WMX-0.9WMY			0.9D-0.9WX+0.9WY+0.9WMX-0.9WMY
		1.2D+1.6Lr-0.45WX+0.45WY-0.45WMX+0.45WMY			1.2D+0.5Lr+L-0.9WX+0.9WY-0.9WMX+0.9WMY			0.9D-0.9WX+0.9WY-0.9WMX+0.9WMY
		1.2D+1.6Lr-0.45WX+0.45WY-0.45WMX-0.45WMY			1.2D+0.5Lr+L-0.9WX+0.9WY-0.9WMX-0.9WMY			0.9D-0.9WX+0.9WY-0.9WMX-0.9WMY
		1.2D+1.6Lr-0.45WX-0.45WY+0.45WMX+0.45WMY			1.2D+0.5Lr+L-0.9WX-0.9WY+0.9WMX+0.9WMY			0.9D-0.9WX-0.9WY+0.9WMX+0.9WMY
		1.2D+1.6Lr-0.45WX-0.45WY+0.45WMX-0.45WMY			1.2D+0.5Lr+L-0.9WX-0.9WY+0.9WMX-0.9WMY			0.9D-0.9WX-0.9WY+0.9WMX-0.9WMY
		1.2D+1.6Lr-0.45WX-0.45WY-0.45WMX+0.45WMY			1.2D+0.5Lr+L-0.9WX-0.9WY-0.9WMX+0.9WMY			0.9D-0.9WX-0.9WY-0.9WMX+0.9WMY
		1.2D+1.6Lr-0.45WX-0.45WY-0.45WMX-0.45WMY			1.2D+0.5Lr+L-0.9WX-0.9WY-0.9WMX-0.9WMY			0.9D-0.9WX-0.9WY-0.9WMX-0.9WMY

Appendix B: 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"
$T_a = 0.02(94.25)^{0.75} = 0.604$ s $C_t = 0.02$, $x = 0.75$

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

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

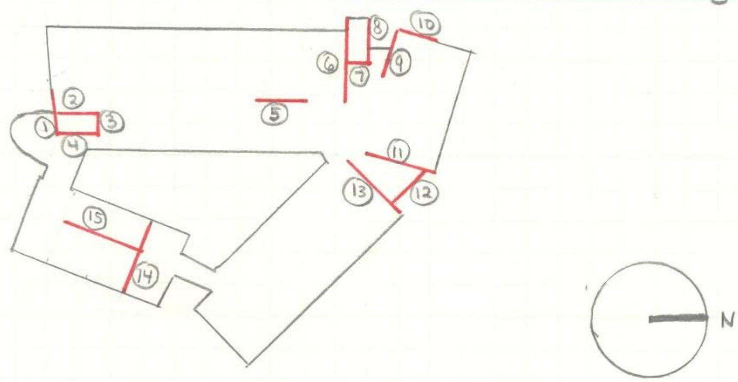
A_B → AREA OF BASE OF STRUCTURE
A_i → WEB AREA OF SHEAR WALL "i" IN FT²
D_i → LENGTH OF SHEAR WALL "i" IN FT
h_i → HEIGHT OF SHEAR WALL "i" IN FT

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)

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

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

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

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

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

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

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

$V_{E-W} = 0.359\,(30,482) = 1,095\text{ K}$
↳ 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
$k = 1 + \dfrac{T - 0.5}{2}$ $(1 \leq k \leq 2)$

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

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

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

Rigid Diaphragm Model - Seismic Response Coefficient (C_S)						
	N-S Direction			E-W Direction		
	Basic	Specific	ETABS*	Basic	Specific	ETABS*
C_t	0.02	N/A	N/A	0.02	N/A	N/A
x	0.75	N/A	N/A	0.75	N/A	N/A
A_B (ft^2)	N/A	25,460	N/A	N/A	25,460	N/A
C_W	N/A	0.15	N/A	N/A	0.21	N/A
h_n (ft)	94.25					
T_a (s)	0.6050	0.4583	N/A	0.6050	0.3932	N/A
C_U	1.7					
C_UT_a	1.0285	0.7792	N/A	1.0285	0.6684	N/A
$C_{S,CALC}$	0.0747					
$C_{S,MAX}$	0.0233	0.0308	0.0281	0.0233	0.0359	0.0367
$C_{S,MIN}$	0.01					
C_S	0.0233	0.0308	0.0281	0.0233	0.0359	0.0367

* Note: Calculated based on mass participation factors and modal periods. See "Rigid Diaphragm Model - Modal Information" table for values used in this calculation.

Semi-Rigid Diaphragm Model - Seismic Response Coefficient (C_S)						
	N-S Direction			E-W Direction		
	Basic	Specific	ETABS*	Basic	Specific	ETABS*
C_t	0.02	N/A	N/A	0.02	N/A	N/A
x	0.75	N/A	N/A	0.75	N/A	N/A
A_B (ft^2)	N/A	25,460	N/A	N/A	25,460	N/A
C_W	N/A	0.15	N/A	N/A	0.21	N/A
h_n (ft)	94.25					
T_a (s)	0.6050	0.4583	N/A	0.6050	0.3932	N/A
C_U	1.7					
C_UT_a	1.0285	0.7792	N/A	1.0285	0.6684	N/A
$C_{S,CALC}$	0.0747					
$C_{S,MAX}$	0.0233	0.0308	0.0224	0.0233	0.0359	0.0236
$C_{S,MIN}$	0.01					
C_S	0.0233	0.0308	0.0224	0.0233	0.0359	0.0236

* Note: Calculated based on mass participation factors and modal periods. See "Semi-Rigid Diaphragm Model - Modal Information" table for values used in this calculation.

Sesimic Load Combinations Used in Modelling			
ASCE 7 Chapter 2 Combination #5	1.206D+L+0.2Lr+1.0EX	ASCE 7 Chapter 2 Combination #7	0.894D+1.0EX
	1.206D+L+0.2Lr+1.0EY		0.894D+1.0EY
	1.206D+L+0.2Lr-1.0EX		0.894D-1.0EX
	1.206D+L+0.2Lr-1.0EY		0.894D-1.0EY
	1.206D+L+0.2Lr+1.0EX+1.0EMX		0.894D+1.0EX+1.0EMX
	1.206D+L+0.2Lr+1.0EY+1.0EMY		0.894D+1.0EY+1.0EMY
	1.206D+L+0.2Lr+1.0EX-1.0EMX		0.894D+1.0EX-1.0EMX
	1.206D+L+0.2Lr+1.0EY-1.0EMY		0.894D+1.0EY-1.0EMY
	1.206D+L+0.2Lr-1.0EX+1.0EMX		0.894D-1.0EX+1.0EMX
	1.206D+L+0.2Lr-1.0EY+1.0EMY		0.894D-1.0EY+1.0EMY
	1.206D+L+0.2Lr-1.0EX-1.0EMX		0.894D-1.0EX-1.0EMX
	1.206D+L+0.2Lr-1.0EY-1.0EMY		0.894D-1.0EY-1.0EMY

Appendix C: Shear Wall Capacity Checks

SHEAR CAPACITY | TECH 3 | pg 1 OF 1

FOR WALLS RESISTING SEISMIC FORCES,

$$V_n = A_{cv}\left(\alpha_c \lambda \sqrt{f'_c} + \rho_t f_y\right) \qquad \text{ACI 318-08 § 21.9.4.1}$$

$$\rho_t = \frac{A_{v,horiz}}{h\, s_{horiz}}$$

$\alpha_c = 2.0$ FOR $h_w/\ell_w \geq 2.0$

$\alpha_c = 3.0$ FOR $h_w/\ell_w \leq 1.5$

USE INTERPOLATION FOR $1.5 < h_w/\ell_w < 2$

A_{cv} = SHEAR AREA OF CONCRETE

CHECK SHEAR WALL 1

$h_w = 1131$ in

$\ell_w = 200$ in

$\frac{h_w}{\ell_w} = 5.65 > 2.0 \therefore \alpha_c = 2.0$

REINFORCING PROVIDED → #5 @ 18" O.C, EACH FACE

$$\therefore \rho_t = \frac{2\,(0.31\text{ in}^2)}{12\text{in}\,(18\text{in})} = 0.00287$$

FOR NWC, $\lambda = 1.0$

$$V_n = 12\text{in}\,(200\text{ in})\left[2\,(1.0)\sqrt{4500\text{ psi}} + 0.00287\,(60{,}000\text{ psi})\right]$$

$V_n = 735.3$ K

$\phi V_n = 0.75\,(735.3\text{ k}) = 551.46$ K

MAX SHEAR IN WALL 1, RIGID DIAPHRAGM → $V_{u,RD} = 224.46$ K

$\phi V_n > V_u$ OK ✓

MAX SHEAR IN WALL 1, SEMI-RIGID DIAPHRAGM → $V_{u,S\text{-}RD} = 294.65$ K

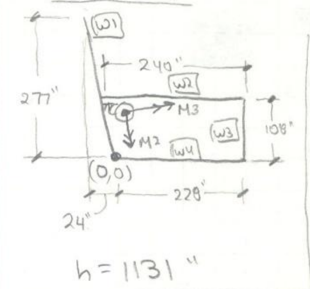
$\phi V_n > V_u$ OK ✓

OTHER SHEAR WALL CAPACITIES CHECKED IN SPREADSHEET

MOMENT CHECKS | TECH 3 | Pg 1 OF 6

WALLS BEHAVE AS GROUPS WHEN RESISTING MOMENTS

GROUP 1



FIND GROUP CENTROID

$$\bar{X} = \frac{\sum_{i=1}^{4} L_i x_i}{\sum_{i=1}^{4} L_i} = \frac{278''(-12'') + 240''(110'') + 228''(114'') + 108''(228'')}{278 + 240 + 228 + 108} = 86.3 \text{ in}$$

$$\bar{Y} = \frac{\sum_{i=1}^{4} L_i y_i}{\sum_{i=1}^{4} L_i} = \frac{278''(138.5'') + 240''(108'') + 228''(0'') + 108''(54'')}{278 + 240 + 228 + 108} = 79.0 \text{ in}$$

FIND $+\phi T_n$

$$\Sigma M_c = T_n = T_3(240'') = A_{s3} F_y (240'')$$

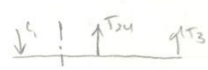
$$\phi T_n = 0.9\left(\frac{0.62 \text{ in}^2}{18''}\right)(1131 \text{ in})(60 \text{ ksi})(240'')$$

$$= 504{,}878 \text{ k-in} = \boxed{42{,}073 \text{ k-ft}}$$

FIND $-\phi T_n$

$$-\phi T_n = +\phi T_n = \boxed{-42{,}073 \text{ k-ft}}$$

FIND $+\phi M_{n2}$

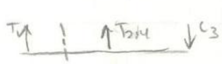


$$\Sigma M_c = M_{n2} = T_{2,4}(124) + T_3(240'') = A_{s2,4} F_y (124'') + A_{s3} F_y (240'')$$

$$\phi M_{n2} = 0.9\left(\frac{0.62 \text{ in}^2}{18''}\right)(240 + 228'')(60 \text{ ksi})(124'') + 0.9\left(\frac{0.62 \text{ in}^2}{18''}\right)(108'')(60 \text{ ksi})(240'')$$

$$= 156{,}150 \text{ k-in} = \boxed{13{,}013 \text{ k-ft}}$$

FIND $-\phi M_{n2}$



$$T_1 + T_2 + T_4 = \left(\frac{0.62 \text{ in}^2}{18 \text{ in}}\right)(60 \text{ ksi})(278 + 240 + 228) = 575 + 496 + 470 = 1541$$

$$a = \frac{T_1}{0.85 f'_c b} = \frac{1541 \text{ k}}{0.85(4.5 \text{ ksi})(108'')} = 3.73'' < t_{wall} = 12$$

$\therefore T_{2,4}$ OK ✓

$$-\phi M_{n2} = 0.9[575 \text{ k}(240'') + (496 \text{ k} + 470 \text{ k})(112 \text{ in})] = 221{,}573 \text{ k-in} = \boxed{-18{,}464 \text{ k-ft}}$$

MOMENT CHECKS | TECH 3 | pg 2 OF 6

FIND $+\phi M_{n3}$

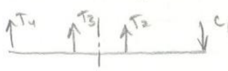
$$T_1 + T_2 + T_3 = 575 + 496 + \overbrace{\left(\tfrac{0.62\,in^2}{18\,in}\right)(60\,ksi)(108)}^{223} = 1294\ k$$

$$a = \frac{T_1 + T_2 + T_3}{0.85 f'_c\, b} = \frac{1294\ k}{0.85\,(4.5\,ksi)(278\,in)} = 1.22\ in < t_{WALL}$$

$\therefore T_3$ OK ✓

$$\phi M_{n3} = 0.9\left[(496\,k)(108'') + (575\,k)(192.5) + 223(54'')\right]$$
$$= 158{,}668\ k\text{-}in = \boxed{13{,}220\ k\text{-}ft}$$

FIND $-\phi M_{n3}$



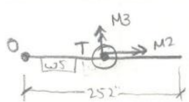
$$T_4 + T_3 + T_2 = 470 + 223 + 496 = 1189\ k$$

$$a = \frac{T_4 + T_3 + T_2}{0.85 f'_c\, b} = \frac{1189\ k}{0.85\,(4.5\,ksi)(12\,in)} = 26\ in < 169\ in$$

$\therefore T_2$ OK ✓

$$-\phi M_{n3} = 0.9\left[470k(277\,in) + 223\,k(223\,in) + 496\,k(169\,in)\right]$$
$$= 237{,}369\ k\text{-}in = \boxed{19{,}781\ k\text{-}ft}$$

GROUP 2



CENTROID IS AT THE CENTER

WALL CANNOT RESIST TORSION OR M2

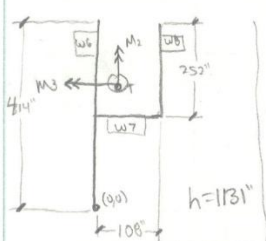
$h = 1131\ in$

FIND $+\phi M_3 / -\phi M_{n3}$

$$T = \left(\tfrac{0.62\,in^2}{18\,in}\right)(126\,in)(60\,ksi) = 260\ K$$

$$\phi M_{n3} = 0.9\,(260\,k)(126\,in) = 29{,}484\ k\text{-}in = \boxed{2{,}457\ k\text{-}ft}$$

GROUP 3

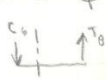


FIND GROUP CENTROID

$$\bar{X} = \frac{\sum_{i=1}^{3} L_i x_i}{\sum_{i=1}^{3} L_i} = \frac{414''(0'') + 108''(54'') + 252''(108'')}{414 + 108 + 252} = 42.7\ in$$

$$\bar{Y} = \frac{\sum_{i=1}^{3} L_i y_i}{\sum_{i=1}^{3} L_i} = \frac{414''(207'') + 108''(162'') + 252''(288'')}{414 + 108 + 252} = 227.1\ in$$

FIND $+\phi T_n / -\phi T_n$



$$T_8 = A_{s8} F_y = \left(\tfrac{0.62\,in^2}{18\,in}\right)(1131)(60\,ksi) = 2337\ K$$

$$\phi T_n = 0.9\,(2337\,k)(108'') = 227{,}195\ k\text{-}in = \boxed{18{,}930\ k\text{-}ft}$$

MOMENT CHECK | TECH 3 | pg 3 OF 6

FIND $+\phi M_{n2}$

$$\Sigma M_c = M_n = T_7\left(\frac{108''}{2}\right) + T_8(108'') = A_{s7}F_y(54'') + A_{s8}F_y(108'')$$

$$\phi M_n = 0.9\left(\frac{0.62\ in^2}{18\ in}\right)(108'')(60\ ksi)(54'') + 0.9\left(\frac{0.62\ in^2}{18\ in}\right)(252'')(60\ ksi)(108'')$$

$$= 61{,}469.3\ k\text{-}in = \boxed{5{,}122\ k\text{-}ft}$$

FIND $-\phi M_{n2}$

$$T_6 + T_7 = \left(\frac{0.62\ in^2}{18\ in}\right)(60\ ksi)(414'' + 108'') = 856\ k + 223\ k = 1079\ in$$

$$a = \frac{T_6 + T_7}{0.85 f'_c\, b} = \frac{1079\ in}{0.85(4.5\ ksi)(252'')} = 1.12\ in < t_{WALL} \quad \therefore T_7\ OK$$

$$-\phi M_{n2} = 0.9\left[856\ k(108\ in) + 223\ k(54\ in)\right] = 94{,}041\ k\text{-}in = \boxed{-7{,}837\ k\text{-}ft}$$

FIND ϕM_{n3}

$$\Sigma M_c = M_n = T_7(252'') + T_8(333'') = A_{s7}F_y(252'') + A_{s8}F_y(301'')$$

$$\phi M_n = 0.9\left(\frac{0.62\ in^2}{18''}\right)(108'')(60\ ksi)(252'') + 0.9\left(\frac{0.62\ in^2}{18\ in}\right)(227'')(60\ ksi)(301'')$$

$$= 177{,}710\ k\text{-}in = \boxed{14{,}809\ k\text{-}ft}$$

FIND $-\phi M_{n3}$

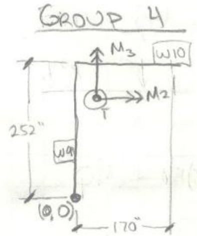
$$T_7 + T_{6,8} = A_6F_y + A_7F_y + A_8F_y = 2(252'')\left(\frac{0.62\ in^2}{18\ in}\right)(60\ ksi) + \left(\frac{0.62\ in^2}{18''}\right)(60\ ksi)(108'')$$

$$= 1042 + 223 = 1265\ k$$

$$a = \frac{T_7 + T_{6,8}}{0.85 f'_c\, b} = \frac{1265\ k}{0.85(4.5\ ksi)(24\ in)} = 13.78\ in < 162\ in \quad \therefore T_7\ OK$$

$$-\phi M_{n3} = 0.9\left[1042\ k(414\ in) + 223\ k(162\ in)\right] = 420{,}763\ k\text{-}in = \boxed{-35{,}064\ k\text{-}ft}$$

GROUP 4



$h = 1029''$

FIND GROUP CENTROID

$$\bar{X} = \frac{\sum_{i=1}^{2} L_i x_i}{\sum_{i=1}^{2} L_i} = \frac{252(0'') + 170''(85'')}{252 + 170} = 34.2\ in$$

$$\bar{Y} = \frac{\sum_{i=1}^{2} L_i y_i}{\sum_{i=1}^{2} L_i} = \frac{252''(126'') + 170''(252'')}{252 + 70} = 176.8\ in$$

FIND $\pm\phi T_n$

$$T = \left(\frac{0.62\ in^2}{18\ in}\right)(60\ ksi)(1029'') = 2127\ k$$

$$\phi T_n = 0.9(2127\ k)(34.2\ in) = 65469\ k\text{-}in = \boxed{5{,}456\ k\text{-}ft}$$

MOMENT CHECKS | TECH 3 | pg 4 of 6

FIND ϕM_{n2}

$T_{10} = \left(\frac{0.62\ in^2}{18\ in}\right)(60\ ksi)(170") = 351\ k$

$\phi M_{n2} = 0.9(351\ k)(252\ in) = 79{,}607\ k\text{-}in = \boxed{6{,}634\ k\text{-}ft}$

FIND $-\phi M_{n2}$

$T_9 = \left(\frac{0.62\ in^2}{18\ in}\right)(60\ ksi)(252\ in) = 521\ k$

$-\phi M_{n2} = 0.9(521\ k)(126\ in) = 59{,}081\ k\text{-}in = \boxed{-4{,}923\ k\text{-}ft}$

FIND ϕM_{n3}

$T_9 = 521\ k$

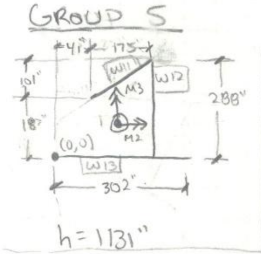
$\phi M_{n3} = 0.9(521\ k)(170\ in) = 79{,}713\ k\text{-}in = \boxed{6{,}643\ k\text{-}ft}$

FIND $-\phi M_{n3}$

$T_{10} = 351\ k$

$\phi M_{n3} = 0.9(351\ k)(170\ in) = 53{,}703\ k\text{-}in = \boxed{4{,}475\ k\text{-}ft}$

GROUP 5



FIND GROUP CENTROID

$\bar{X} = \frac{\sum_{i=1}^{3} L_i x_i}{\sum_{i=1}^{3} L_i} = \frac{302"(151") + 288"(216") + 202"(128.5")}{302 + 288 + 202} = 168.9\ in$

$\bar{Y} = \frac{\sum_{i=1}^{3} L_i y_i}{\sum_{i=1}^{3} L_i} = \frac{302(0") + 288"(144") + 202"(237")}{302 + 288 + 202} = 112.8\ in$

FIND $\pm\phi T_n$

$T_{11} = \left(\frac{0.62\ in^2}{18\ in}\right)(60\ ksi)(1131\ in) = 2337\ k$

$\phi T_n = 0.9(2337\ k)(237") = 498{,}482\ k\text{-}in = \boxed{41{,}540\ k\text{-}ft}$

FIND $+\phi M_{n2}$

$T_{11} + T_{12} = \left(\frac{0.62\ in^2}{18\ in}\right)(60\ ksi)(202\ in + 288\ in) = 417\ k + 595\ k = 1012\ k$

$a = \frac{T}{0.85 f'_c b} = \frac{1012\ k}{0.85(4.5\ ksi)(302")} = 0.88\ in < t_{wall}$ $\therefore t_{12}$ OK ✓

$\phi M_{n2} = 0.9[417\ k(237\ in) + 595\ k(144\ in)] = 166{,}058\ k\text{-}in = \boxed{13{,}838\ k\text{-}ft}$

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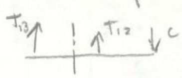
FIND $-\phi M_{n2}$



$$T_{13}+T_{12} = \overbrace{\left(\frac{0.62\,in^2}{18\,in}\right)(60\,ksi)(302\,in)}^{624\,k} + 595\,k = 1219\,k$$

$$a = \frac{T}{0.85 f'_c b} = \frac{1219\,k}{0.85(4.5\,ksi)(175\,in)} = 1.82\,in < t_{wall} \quad \therefore T_{12}\ OK$$

$$-\phi M_{n2} = 0.9\left[624\,k\,(237\,in) + 595\,k\,(93\,in)\right] = 182{,}901\ k\text{-}in = \boxed{-15{,}240\ k\text{-}ft}$$

FIND ϕM_{n3}

$$T_{11}+T_{13} = 417\,k + 624\,k = 1041\,k$$

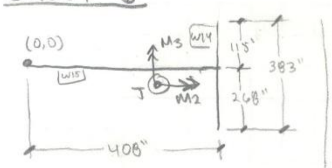
$$\phi M_{n3} = 0.9\left[417\,k\,(175\,in) + 624\,k\,(216\,in)\right] = 186{,}983\ k\text{-}in = \boxed{15{,}582\ k\text{-}ft}$$

FIND $-\phi M_{n3}$

$$T_{11}+T_{12} = 417\,k + 595\,k = 1012\,k$$

$$-\phi M_{n3} = 0.9\,(1012\,k)(216\,in) = 196{,}733\ k\text{-}in = \boxed{-16{,}394\ k\text{-}ft}$$

GROUP 6



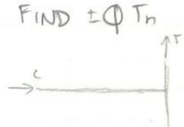
$h = 686$ in

FIND GROUP CENTROID

$$\bar{X} = \frac{\sum_{i=1}^{2} L_i x_i}{\sum_{i=1}^{2} L_i} = \frac{408''(204'') + 383''(408'')}{408 + 383} = 303\ in$$

$$\bar{Y} = \frac{\sum_{i=1}^{2} L_i y_i}{\sum_{i=1}^{2} L_i} = \frac{408''(268'') + 383''(192'')}{408'' + 383''} = 231\ in$$

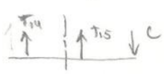
FIND $\pm\phi T_n$



$$T = \left(\frac{0.62\,in^2}{18\,in}\right)(686\,in)(60\,ksi) = 1418\,k$$

$$\phi T_n = 0.9\,(1418\,k)(115'') = 146{,}763\ k\text{-}in = \boxed{12{,}230\ k\text{-}ft}$$

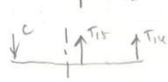
FIND ϕM_{n2}



$$T_{15} + T_{14} = \left(\frac{0.62\,in^2}{18\,in}\right)(60\,ksi)(408\,in + 268\,in) = 843\,k + 554\,k = 1397\,k$$

$$\phi M_{n2} = 0.9\left[554\,k\,(249\,in) + 843\,k\,(115\,in)\right] = 211{,}402\ k\text{-}in = \boxed{17{,}617\ k\text{-}ft}$$

FIND $-\phi M_{n2}$



$$T_{15} + T_{14} = 843\,k + \overbrace{\left(\frac{0.62\,in^2}{18\,in}\right)(60\,ksi)(115\,in)}^{238\,k} = 1081\,k$$

$$-\phi M_{n2} = 0.9\left[843\,k\,(268\,in) + 238\,k\,(326\,in)\right] = 273{,}161\ k\text{-}in = \boxed{-22{,}763\ k\text{-}ft}$$

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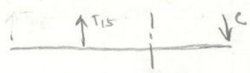
FIND ϕM_{n3}



$T_{15} = 843\ k$

$\phi M_n = 0.9\ (843\ k)(204\ in) = 154{,}775\ k\text{-}in = \boxed{12{,}898\ k\text{-}ft}$

FIND $-\phi M_{n3}$

$T_{14} = \left(\frac{0.62\ in^2}{18\ in}\right)(60\ ksi)(363\ in) = 750\ k$

$\phi M_n = 0.9(750\ k)(408\ in) = 275{,}400\ k\text{-}in = \boxed{-22{,}950\ k\text{-}ft}$

DRIFT CHECK | TECH 3 | pg 1 OF 2

USE WALL 1 TO CHECK MAX. DISPLACEMENT IN THE Y-DIRECTION
↳ THIS IS CAUSED BY $1.206D + E_Y - E_{MY} + L + 0.2L_r$

STORY FORCES IN WALL 1 DUE TO THIS LOAD (ETABS):		ELEV	WALL STIFFNESS (HAND CALC)
LEVEL 2 →	112.45 K	= 182 in	$K_{1,y}$ = 11,557 K/in
LEVEL 3 →	160.62 K	= 350 in	= 5337 K/in
LEVEL 4 →	127.43 K	= 518 in	= 3050 K/in
LEVEL 5 →	53.40 K	= 686 in	= 1897 K/in
PENTHOUSE →	40.32 K	= 861 in	= 1223 K/in
ATRIUM ROOF →	36.93 K	= 947 in	= 1002 K/in
CHILLER ROOF →	36.94 K	= 1029 in	= 836 K/in
AHU ROOF →	36.95 K	= 1131 in	= 675 K/in

$$\Delta_{max,y} = \frac{P}{K} = \frac{112.45\,K}{11557\,K/in} + \frac{160.62\,K}{5337\,K/in} + \frac{127.43\,K}{3050\,K/in} + \frac{53.40\,K}{1897\,K/in} + \frac{40.32\,K}{1223\,K/in} + \frac{36.93\,K}{1002\,K/in} + \frac{36.94\,K}{836\,K/in} + \frac{36.95\,K}{675\,K/in}$$

$\Delta_{max,y} = 0.279$ in

STIFFNESS CALCULATIONS NOT RIGHT

$\Delta_{max,y,ETABS,RIGID} = 0.594$ in (~50% DIFFERENCE)

$\Delta_{max,y,ETABS,SEMI} = 0.831$ in

$$DRIFT,\ MAX,\ RIGID = \frac{0.0719}{1131-1029} = 0.000705 \sim \frac{l}{1418} > \frac{l}{400}$$

OK ✓

USE WALL 7 TO CHECK MAX DISPLACEMENT IN THE X-DIRECTION
↳ THIS IS CAUSED BY $1.206D + E_X - E_{MX} + L + 0.2L_r$

STORY FORCES IN WALL DUE TO THIS LOAD (ETABS):		ELEV	WALL STIFFNESS (HAND CALC)
LEVEL 2 →	45.93 K	= 182 in	$K_{7,x}$ = 4491 K/in
LEVEL 3 →	42.85 K	= 350 in	= 2057 K/in
LEVEL 4 →	38.24 K	= 518 in	= 1164 K/in
LEVEL 5 →	37.85 K	= 686 in	= 717 K/in
PENTHOUSE →	37.77 K	= 861 in	= 459 K/in
ATRIUM ROOF →	16.17 K	= 947 in	= 374 K/in
CHILLER ROOF →	16.17 K	= 1029 in	= 311 K/in
AHU ROOF →	16.19 K	= 1131 in	= 251 K/in

DRIFT CHECK | TECH 3 | pg 2 OF 2

$$\Delta_{max,x} = \frac{P}{K} = \frac{45.93\,k}{4491\,k/in} + \frac{42.85\,k}{2057\,k/in} + \frac{38.24\,k}{1164\,k/in} + \frac{37.85\,k}{717\,k/in} + \frac{37.77\,k}{459\,k/in} + \frac{16.17\,k}{374\,k/in} + \frac{16.17\,k}{311\,k/in} + \frac{16.19\,k}{251\,k/in}$$

$\Delta_{max,x} = 0.359$ in ← ~30% DIFFERENT, INDICATES STIFFNESS OF WALL 7 IS SIGNIFICANTLY DIFFERENT IN HAND CALCS VS. ETABS

$\Delta_{max,x,ETABS,RIGID} = 0.518$ in

$\Delta_{max,x,ETABS,SEMI} = 0.529$ in

FROM ETABS → MAX DRIFT IN X-DIRECTION OCCURS @ CHILLER ROOF → AHU ROOF

$$\text{DRIFT}_{MAX,RIGID} = \frac{0.1355\text{ in}}{1131-1029} = 0.0013 \sim \frac{l}{753} > \frac{l}{400} \quad \text{OK}$$

Appendix D: Typical Plans

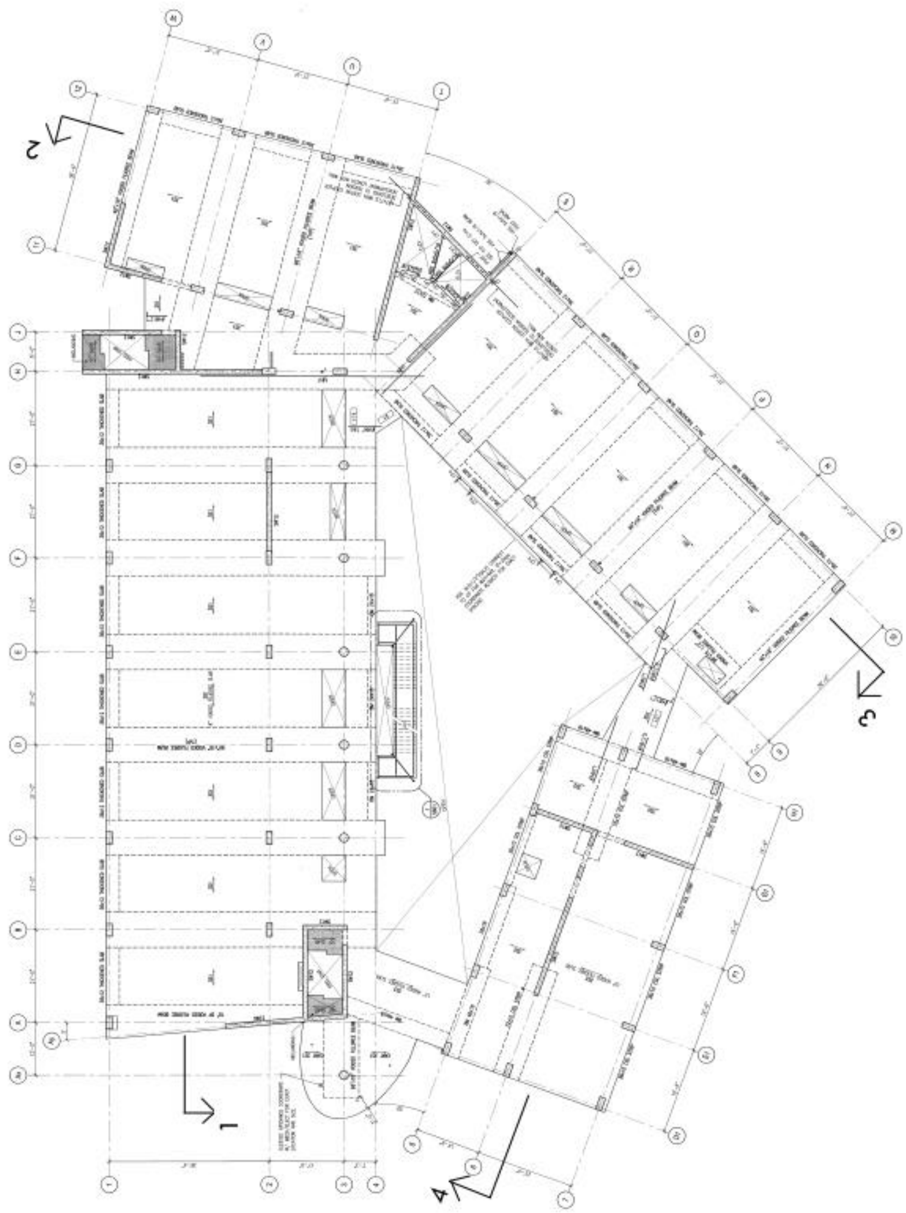


Figure D.1 Typical Floor plan, taken from S202. See following figures for sections indicated on the plan.

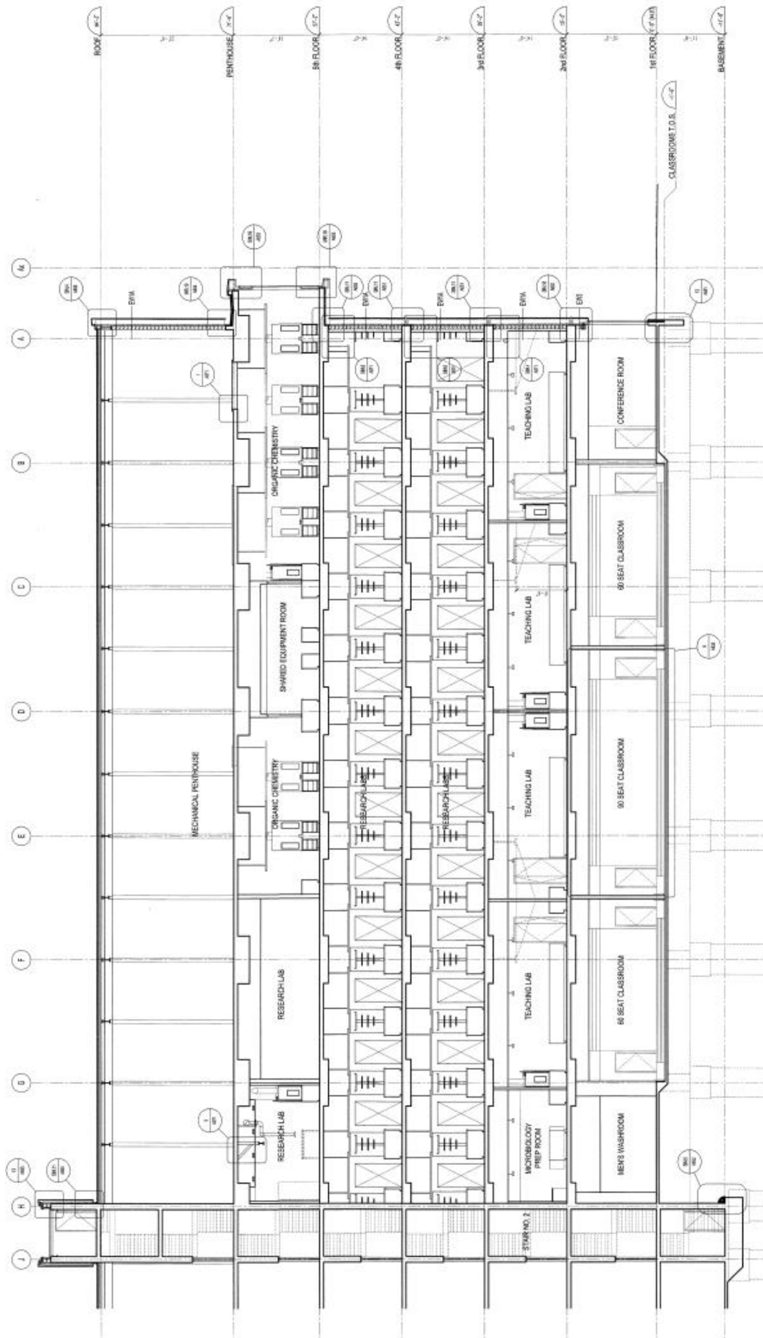


Figure D.2 Section 1 through portion of building at 0° rotation (see Figure 1), taken from 3/A401.

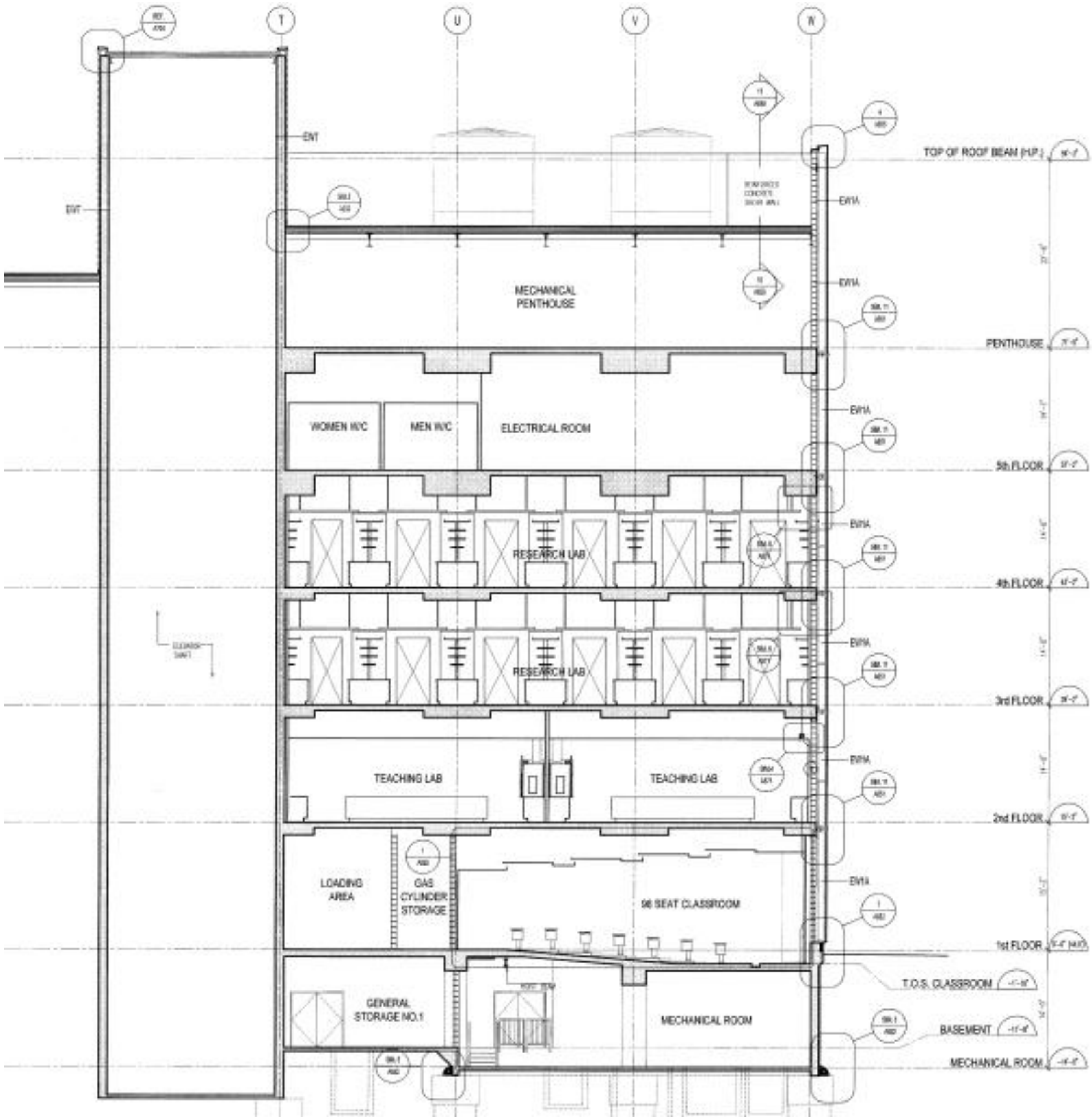


Figure D.3 Section 2 through portion of building at -15° rotation (see Figure 1), taken from 2/A402.

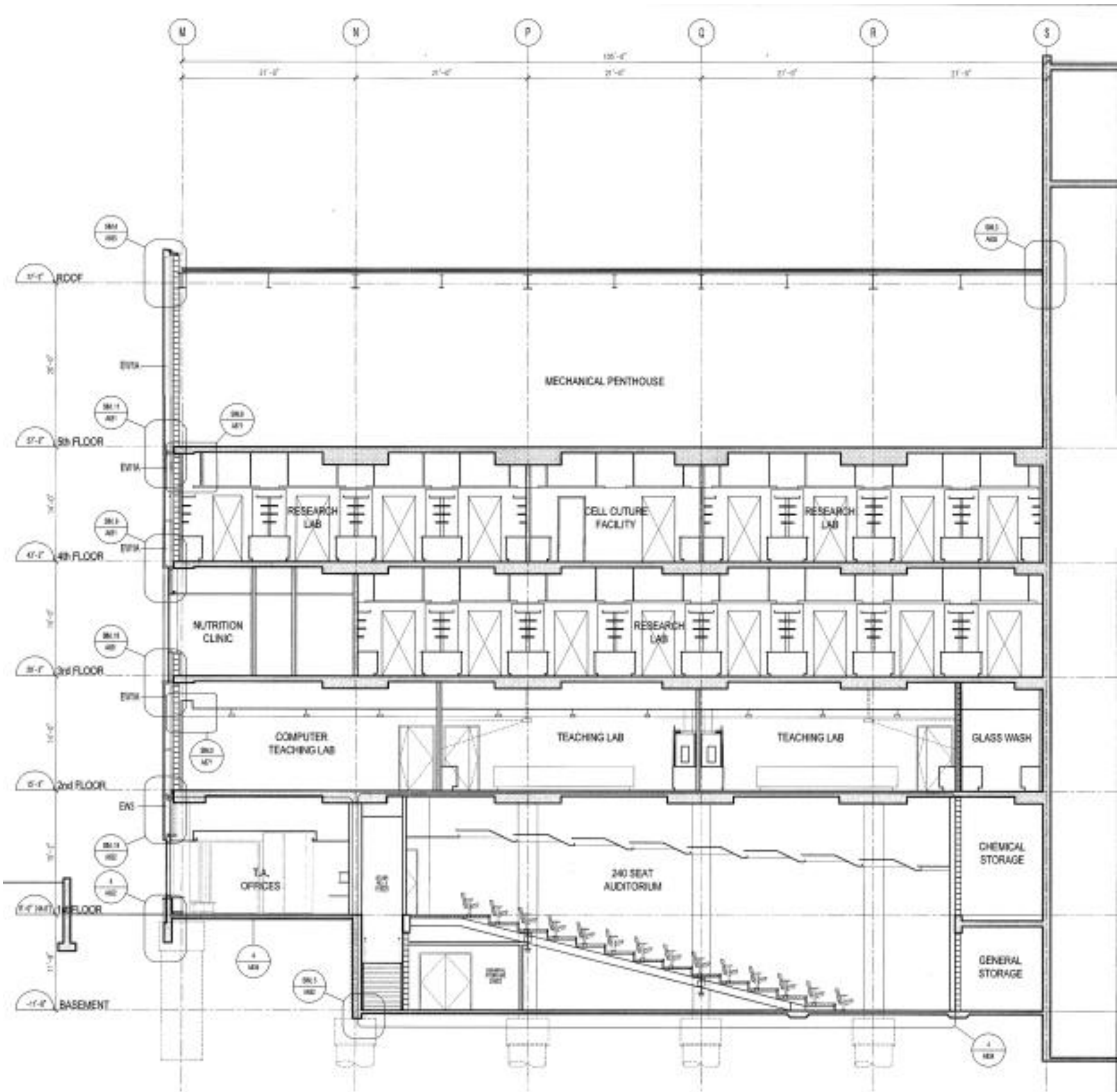


Figure D.4 Section 3 through portion of building at -45° rotation (see Figure 1), taken from 4/A402.

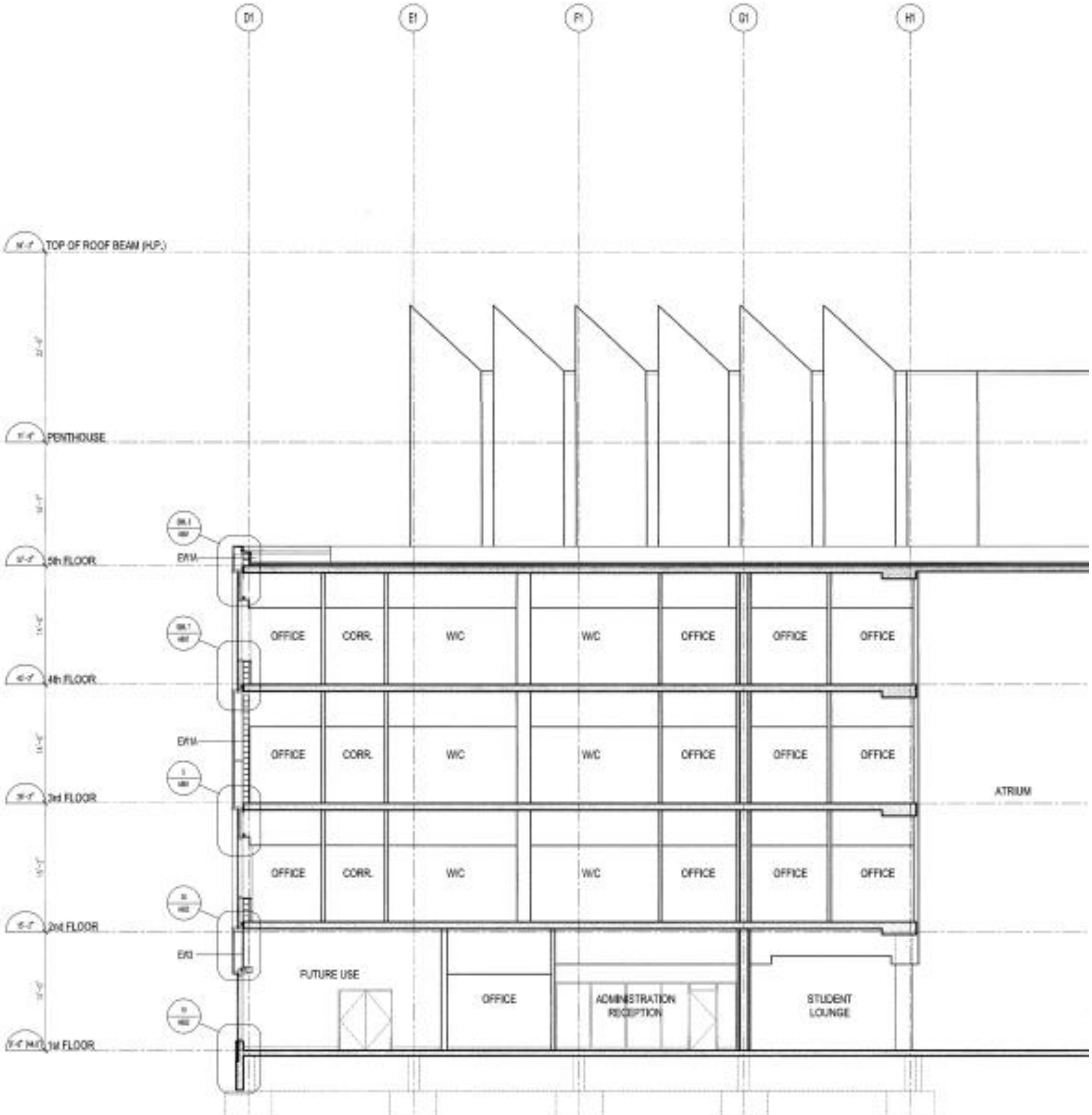


Figure D.5 Section 4 through portion of building at -20° rotation (see Figure 1), taken from 3/A403.

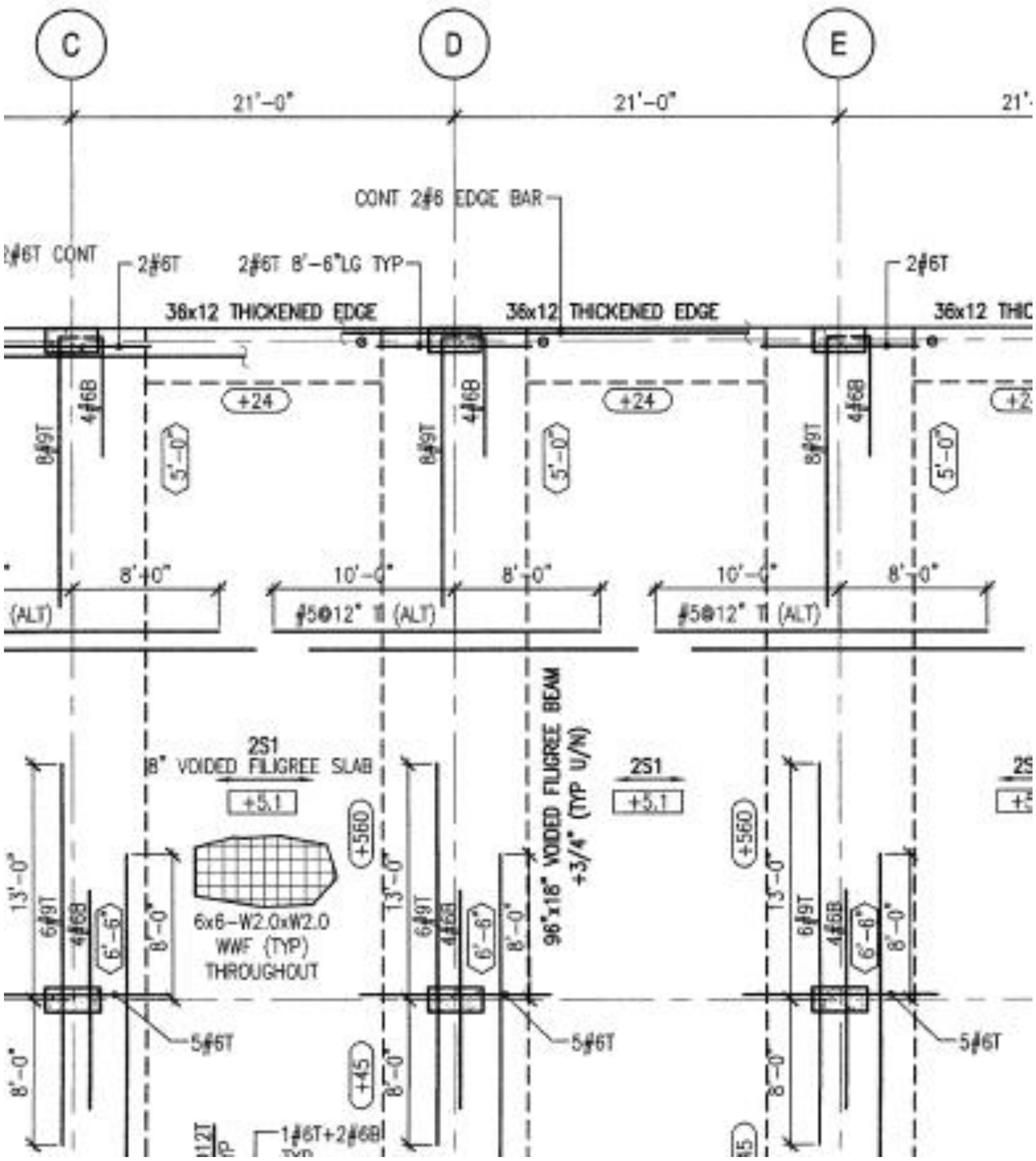


Figure D.6 Enlarged floor plan for a typical bay in the laboratory wing, taken from S202 (levels 2 through 4 are identical, and reinforcing is only displayed on level 2). Slab design moments are boxed (k-ft/ft), beam design moments are enclosed in an oval (k-ft), and the location of the first void in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

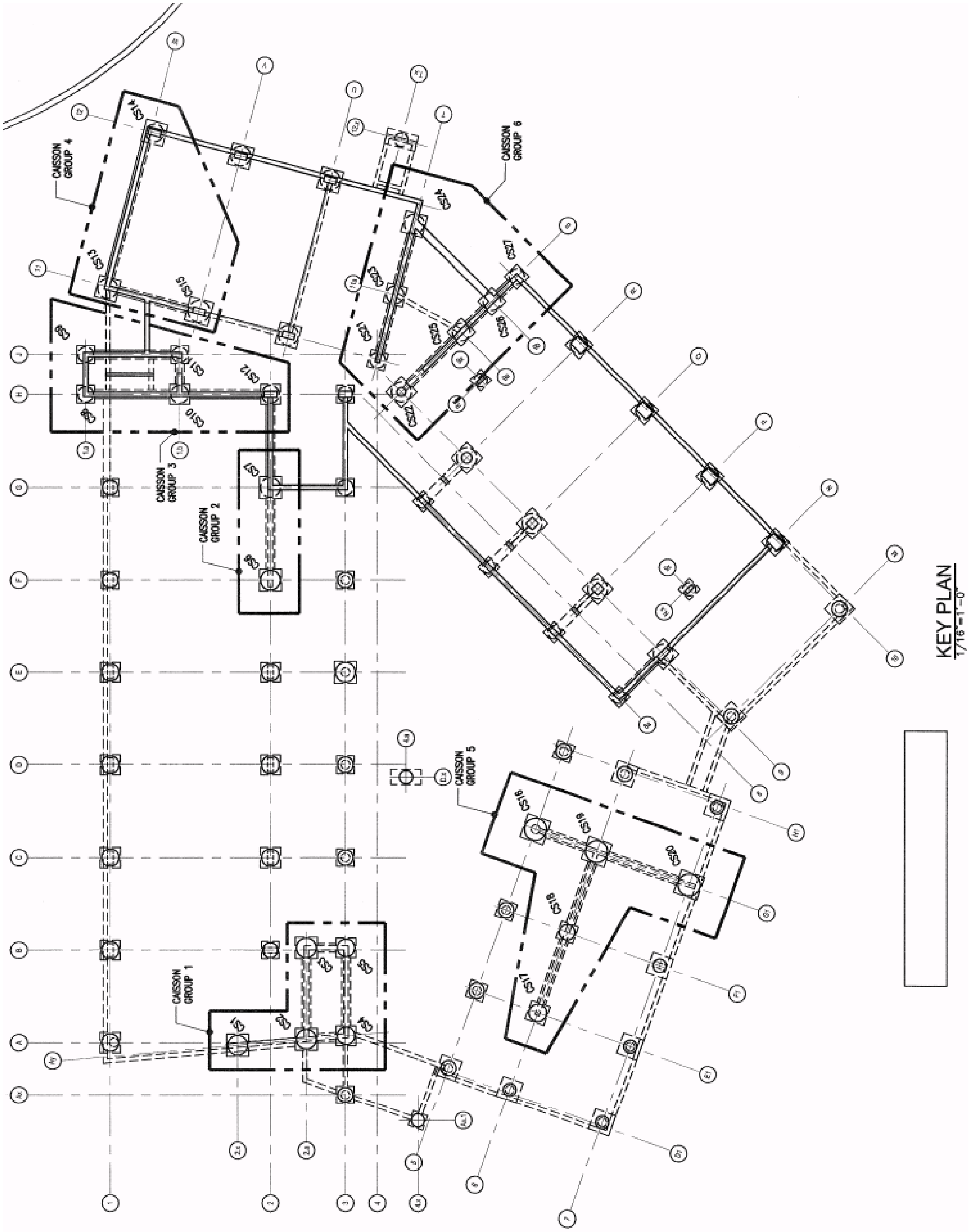


Figure D.7 Caisson groups diagram from Sheet S310.

CAISSON LOAD TABLE FOR SHEAR WALL FOUNDATION				
CAISSON GROUP	CAISSON ID	DOWN-WARD (kip)	UP-WARD (kip)	LATERAL (kip)
1	CS1	1010	70	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	760	–	50
	CS10	1055	–	45
	CS11	810	–	45
	CS12	1380	–	50
4	CS13	1635	–	40
	CS14	1140	–	50
	CS15	1265	–	35
5	CS16	550	15	85
	CS17	560	–	65
	CS18	685	–	65
	CS19	745	–	105
	CS20	965	25	100
6	CS21	820	–	35
	CS22	480	–	35
	CS23	930	–	30
	CS24	1205	–	45
	CS25	1020	–	35
	CS26	990	–	45
	CS27	955	–	45

Figure D.8 Caisson uplift values from Sheet S310.