360 State Street

New Haven, Connecticut

STRUCTURAL CONCEPTS & EXISTING CONDITIONS

STRUCTURAL TECHNICAL REPORT I

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EXECUTIVE SUMMARY

The intent of this report is to establish the structural conditions of 360 State Street located in New Haven, Connecticut. As a new landmark for the city, it consists of street level retail, four stories of parking, and five-hundred rentable apartment units. Overall, the building reaches 32 stories and is a mix of public and private spaces.

360 State Street is the first of its kind in the city of New Haven. The designer/developer wanted to make a statement about the convenience and romance of an urban lifestyle. The design of the building couples sustainable resources and tactics with location and architectural allure. The objective of this report however, is to focus on the conditions designed for the structure. A study of the wind, seismic, and gravity systems was undertaken to determine if the assumptions made by the designer were reasonable.

Beginning with a wind analysis, this report examines the building's loading using ASCE 7 as the standard. The State of Connecticut requires the use of IBC however; this alternate source has the same basic information. Wind velocities were analyzed against the two faces of the building. The lateral systems in place were determined to be sufficient although the rigidity of the structure was found to be low. This may require future attention. At the same time, the seismic analysis of the building revealed the same conclusions using ASCE 7. The structural engineer did not specifically design for seismic loads however; his or her calculations were conservative compared to the results found in this report. The base shear for the original design was found to be 45% higher than the one analyzed in this report. Furthermore, the building weight calculated was found to be 45% lower than the original design. However, this may have been the result of not taking into account superimposed dead loads in the overall weight.

Snow loads were briefly studied and found not to be a significant burden on the structure. More focus was placed on the spot checks of the gravity systems in place. An examination of a roof beam and column concluded that sufficient strength was available in the members however, it brought into question if other factors were included in the original design. According to the spot checks, varying magnitudes of additional available strength were found between the members. The punching shear between a column and slab was also examined. The strength of concrete varied between members such that requirements for additional shear reinforcing were investigated but were found unnecessary. Lastly, a typical slab panel was checked for adequate reinforcing.

To conclude the report, it was found that sufficient strength is available in 360 State Street's structure. For further analysis, a study of the foundation systems and soil composition is recommended. Additionally, it was suggested to conduct a vibration analysis due to a nearby train station. Continuing investigation could also reveal the magnitude of safety factors included in the design of the original structure.

ARCHITECTURAL INTRODUCTION

360 State Street is an innovative building project by the firm Becker + Becker Associates. Located in downtown New Haven, Connecticut, it is situated on the corner of Chapel and State Street just two blocks east of the historic town green. It is also located across the street from an Amtrak train station which services lines to New York and Boston. 360 State Street is a thirty-two story residential tower with four levels of parking and street-level retail. The designer of the building is also the owner who plans to rent the apartment units to students attending Yale University and locals attracted by an urban lifestyle. Becker + Becker also hopes to attract a grocery store to the retail space.

Figure 1: Corner of Chapel & State Street View of 360

Previously, the corner of Chapel and State Street consisted of an abandoned building with an adjacent parking lot that consumed an acre and half of land. With its redevelopment, 360 State Street now envelops the site with the exception of a small plaza in the northwest corner. The building begins one level below grade; this area functions as the loading dock for the retail space. The primary entrances are located at grade. A parking garage extends from the second to the fifth floor with a ramp that swirls around the elevator core. On the sixth level, the residential tower emerges. Its area shrinks to roughly a third of the building's footprint and is centered on the site. The sixth level contains all the amenities which include a fitness center, library, and lounge. The lower roof also doubles as a terrace for 360's residents. It consists of a landscaped garden, an outdoor pool, and a patio. The residential tower extends from the seventh to the thirty-first floor. The units include studio, one, two, and three bedroom apartments. At the very top of the building is a mechanical room which houses 360's cooling towers.

Figure 2: Interior View of Apartment Unit

Overall, 360 State Street tops off at 338'-7" coming in as the second tallest building in New Haven, Connecticut. It is clad with architectural pre-cast concrete panels, masonry, and glazing to match the city's historic character. Ornamentation also decorates the façade on the lower levels. Sustainable features such as the use of recycled building materials, rooftop gardens, and geothermal walls have pushed the building towards LEED certification. The building embodies the designer's vision of a sustainable, urban lifestyle. 360 State Street is a milestone to the city's renaissance and hopes to prove a generous stimulus to the local economy.

Foundations

The foundation of 360 State Street is a reinforced concrete mat slab located 17' – 3" below grade. The slab varies between 36" to 68" in thickness and is reinforced with #11 bars. Its strength is 4,000 psi. The depth of the slab is dependent on the programmed area's function; for example, the tower crane requires a 68" slab to safely support its weight. A mat slab was chosen as the primary support because it can distribute heavy column loads across the entire building's area. It was also chosen based on New Haven's geology and the building's proximity to the water. The site is composed of a high water table, coarse gravels, construction debris, and pockets of fine-grained sands. With these conditions, a shallow foundation with a mat slab was found to be most favorable to resist hydraulic uplift.

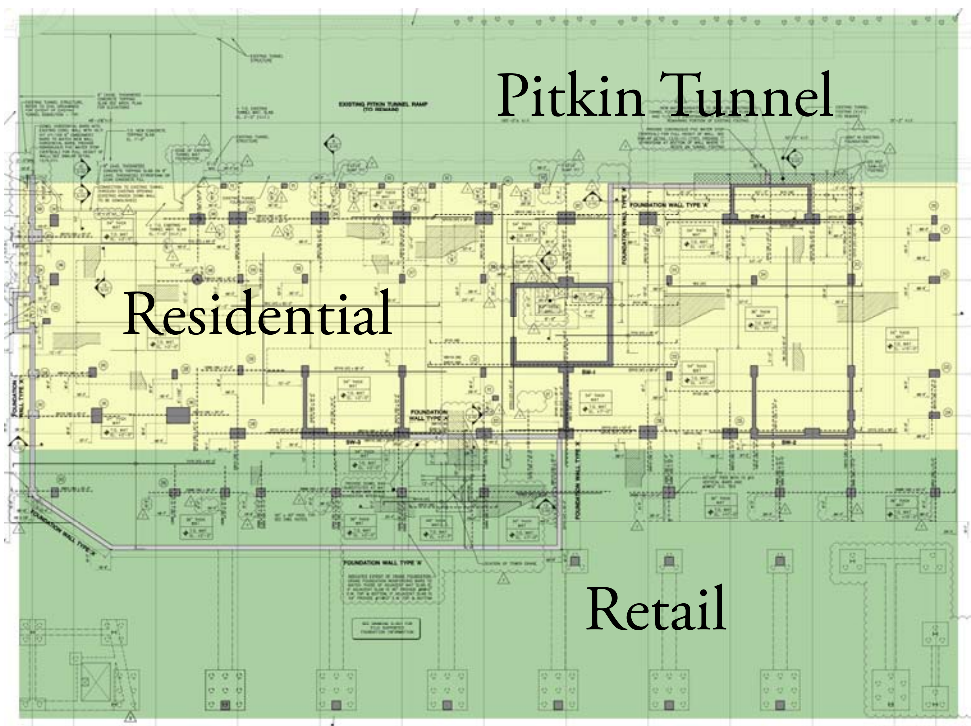


Figure 3: Building Footprint; Roughly each third has a different function.

Adjacent to the site is the Pitkin Tunnel; it previously served as an underground roadway with high-security access to New Haven's City Hall, Federal Courthouse, and electric utilities. Since the tunnel will be reused as the access ramp into the building, the foundations will be underpinned to the existing structure. Additional foundation supports consist of 14" ϕ pressure injected concrete footings that bear onto 10" ϕ drilled concrete mini-piles. The footings have a capacity of 100 tons and the piles have a capacity of 75 tons. In addition, the footings are centered under the columns of the above structure. The foundation walls are located around the perimeter of the residential tower's footprint, which is roughly a third of the overall area. The walls are 16" thick reinforced with #4 and #8 bars and consist of 8000 psi concrete. A frost wall is additionally provided along the exterior perimeter of the retail space. Its depth is 3' – 6" below grade and varies between 14" and 28" in thickness. This wall is also reinforced with #4 bars and ties into the 12" slab-on-grade. A series of 40" x 40" concrete piers support the slab and distribute its load to the footings.

Floor Systems

360 has a variety of concrete floor systems distributed throughout the building. At ground level, there is a 12" slab-on-grade that is reinforced with #5 bars and has a strength of 6,000 psi. This slab covers two-thirds of the building's footprint. The original ramp into the Pitkin Tunnel is replaced with a 5" slab-on-grade that is reinforced with 6 x 6 welded wire fabric. Between the second and fifth floor, three different slabs are used for each third of the building. The center portion that eventually rises into the residential tower consists of a 10" cast-in-place concrete slab. The top and bottom reinforcement is made up of #4 and #6 bars. This area supports the elevator lobby and unit storage rooms. Above the Pitkin Tunnel, a 7" post-tensioned concrete slab supports the tenant parking. The last third of the footprint is above the retail space. It also contains tenant parking and is composed of an 8" cast-in-place slab reinforced with #5 bars. Each system is a two-way flat plate slab that is supported by a series of post-tensioned concrete beams and columns.

The base of 360 State Street was chosen to be made of concrete primarily for its open air parking garage. Previous projects in the city had proven disastrous and costly when concrete was not the principal building material. A large arena dubbed the New Haven Coliseum had several stories of parking available to its patrons that was composed of steel framing. Originally built in 1968, the structure was continuously under repair during its lifetime. The salt in the air from the nearby Long Island Sound coupled with the salts deposited by cars during the winter time corroded the steel. The structure was eventually condemned in 2002 and demolished in 2007. To minimize the costs and labor of maintenance, concrete was the ideal material for this portion of 360 State Street.

The intermediate floor between the concrete base and the residential tower has a 12" cast-in-place slab. The lower roof, also known as the terrace, is composed of a 2" 18 gage galvanized composite floor deck with 3 ¼" concrete. Between the seventh floor and the main roof, an 8" hollow-core pre-cast plank system dominates. Each segment is reinforced with #4 and #5 bars and has a strength of 5,000 psi. The planks are laid out in sections roughly 24' x 8' and are supported by the staggered steel trusses. This is a common type of construction for multi-story apartment buildings for several reasons. The overall cost of labor and materials is lower compared to pouring a slab; less concrete is used and the planks are prefabricated. Additionally, the

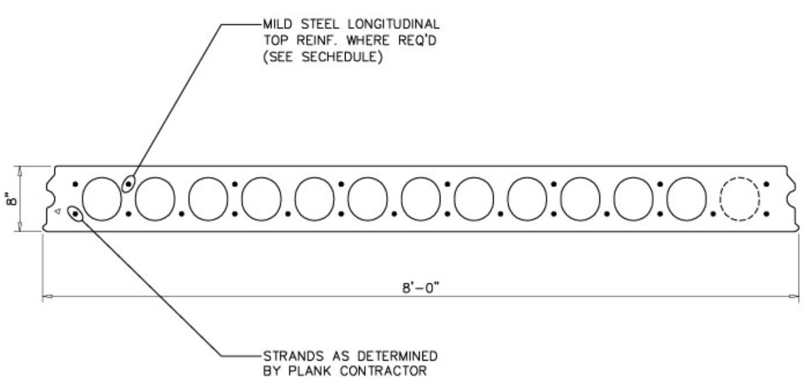


Figure 4: Section of a typical 8" Hollow Core Concrete Plank

hollow cores are filled with stationary air and act as natural insulators between the floors. These factors have lent themselves to making the apartment units more affordable and sustainable for the new tenants. Overall, 360 State Street's floor systems help to carry the live and dead loads of the structure. Additionally, they provide the lateral support necessary to handle wind and seismic loads.

Gravity Systems

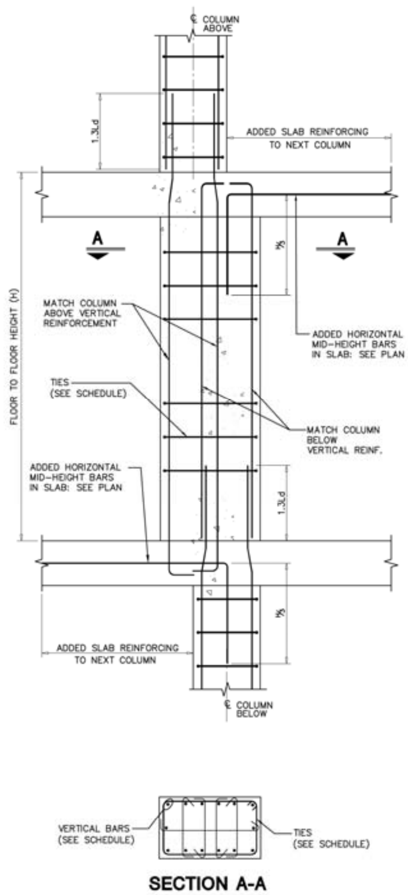


Figure 5: Typical Concrete Column.

There are two unique portions to the building's design. The first six stories are the base of the building which fill the entire acre and half of the site. Reinforced concrete is the primary structure. With the exception of the cellar and first floor, each level is fairly similar in plan. Supporting the floor systems are post-tensioned concrete beams that range in size from 24" or 34" x 32". All are reinforced with # 8 bars and share a strength of 6,000 psi. These beams tie into columns that begin as 36" or 44" x 20" and grow to 80" or 91" x 20" as they approach the foundation. The columns are spaced roughly 24' east to west. Within the center portion of the building, the spacing is 14' north to south however; columns along the exterior are spaced at about 50' to provide room for maneuvering and parking cars.

The second half of the building is comprised of the slender residential tower. Steel shapes define the structure. The columns are primarily found along the exterior perimeter with the exception of those that help support the elevator core. The columns range from W14x500 to W14x68. Steel beams are also found along the exterior with the exception of those around the elevators. Their shapes range from W18x71 to W10x60. Unlike most buildings, 360 uses a system of staggered trusses for its framing. There are nine overall which span 60' across the short length of the building. Staggered trusses were chosen to help maximize the rentable space on each floor. The walls housing the trusses are thicker but they provide adequate sound isolation between the residential units. Additionally, the use of trusses allows for more freedom in the design of interior spaces because they alternate their position between floors and bays. Each individual truss is floor to floor height and is composed of W-shapes and HSS's.

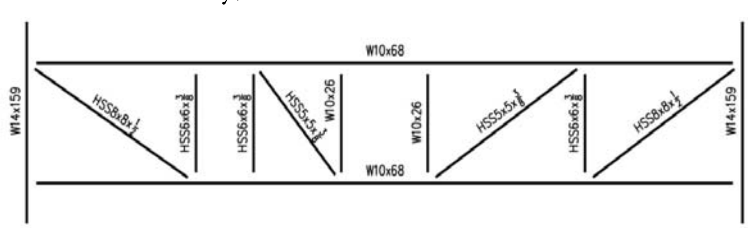


Figure 6: Example of a single Staggered Truss.

Lateral Systems

Although the lateral systems are the most minimal of the entire building, they are among the most important. The beams and columns create 360 State Street's skeleton however; the floor slabs, shear walls, and cross-bracings hold the structure together. Ultimately, the lateral systems help distribute wind and seismic forces across the entire frame.

Four main shear walls are located in the concrete base. Each is composed of 8,000 psi concrete that is 12" thick and is reinforced with #11 vertical bars and #5 horizontal bars. The shear wall designated #1 encases the elevator core and has an additional 4" of thickness. Two of these walls span mainly in the East/West direction. The other two span in both directions to provide additional stability. None of these walls continue past the fifth floor however; steel cross-bracings continue through the residential tower. The braces consist of HSS10x10x3/8 that zigzag along the North/South face of the building. The staggered trusses previously mentioned also helps support in the East/West direction.

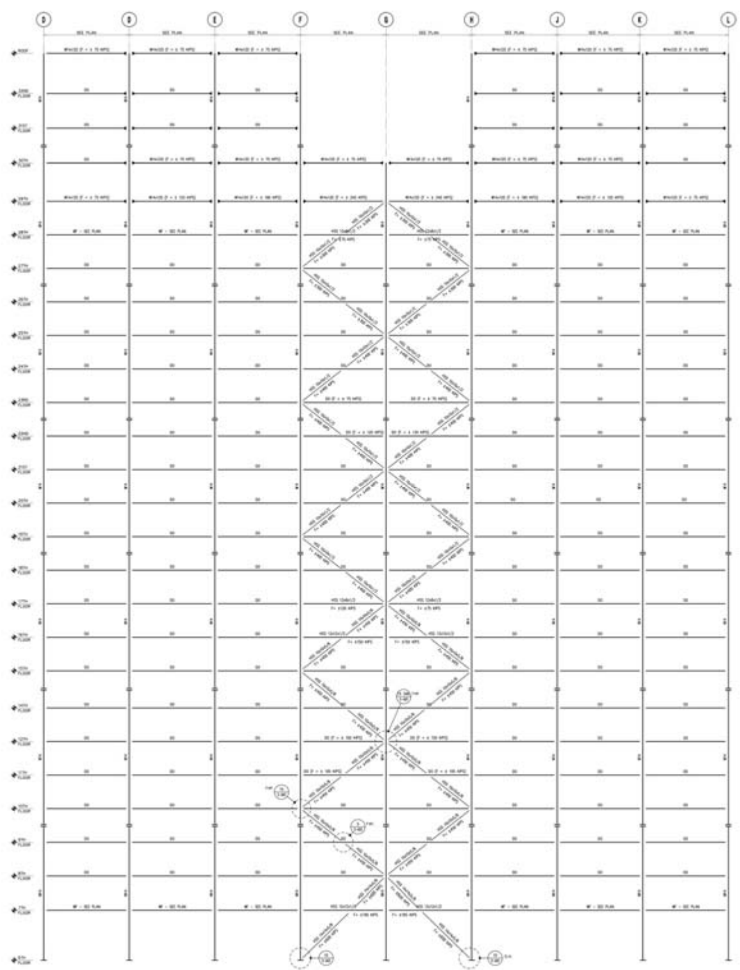


Figure 7: Elevation of North/South Steel Cross-Bracing.

Roof Systems

The main roof is composed of the same 8” hollow core pre-cast planks that support the lower levels of the residential tower. Additionally, a waterproof membrane, 12” R40 rigid insulation, ½” DensDeck prime cover board, and EPDM roofing membrane are layered on top. A pre-cast parapet wall runs along the perimeter of the roof at a height of 3’ – 6”. Flashing and another waterproof membrane tie the construction together. Supporting the parapet wall are moment frames that consist of W12x53’s.

The lower roof is supported by a 2” 18 gage galvanized composite floor deck with a 3 ¼” thick concrete slab. This level is used as the terrace and includes a green roof and a landscaped garden. A drainage mat, filter fabric, and waterproofing are layered on top of the concrete slab to create the base for the landscaping. Soil and vegetation are placed according to the landscaper’s design however; stone pavers are used everywhere else to give the terrace its unique finish.

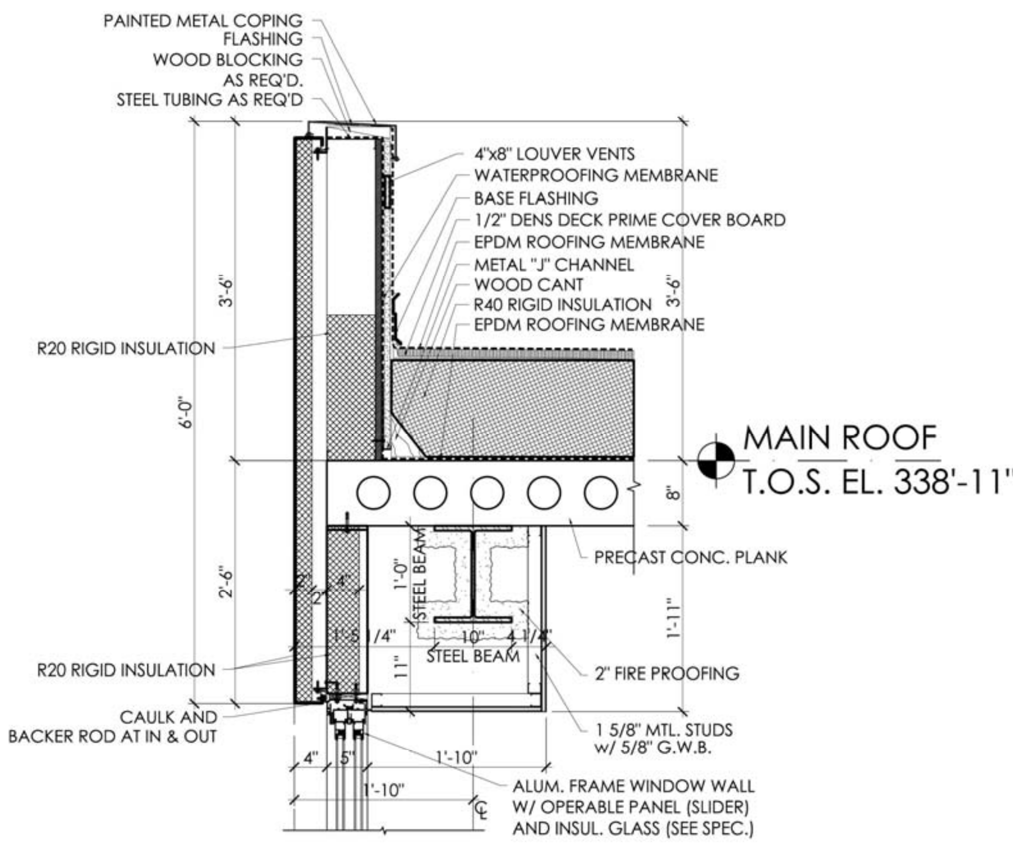


Figure 8: Detail of Main Roof Construction

The following data is provided to illustrate the general design criteria used for 360 State Street. Additional information can be found throughout the report and in the Appendices.

Codes & Design Standards

Applied to Original Design
2005 Connecticut State Building Code consisting of the 2003 International Building Code as modified by the 2005 Connecticut Supplement∵
American Institute of Steel Construction *Specification for Structural Steel Buildings – Allowable Stress Design and Plastic Design* 01 June 1989 (AISC)∵
American Concrete Institute *Building Code Requirements for Structural Concrete* ACI 318-02 (ACI)∵
American Concrete Institute *Building Code Requirements for Masonry Structures* ACI 530-99 (ACI 530)
American Iron and Steel Institute *Specification for the Design of Cold-Formed Steel Structural Members* 1996 (AISI)

∵Substituted for Analysis
American Society for Civil Engineers *Minimum Design Loads for Buildings and Other Structures* ASCE-7-05
American Institute of Steel Construction *Steel Construction Manual, Thirteenth Edition* April 2007 (AISC)
American Concrete Institute *Building Code Requirements for Structural Concrete and Commentary* ACI 318-08 (ACI)

Table 1: Codes & Standards used for Original & Analyzed Design.

Note: Thesis Design Analysis was conducted using Load and Resistance Factor Design (LRFD).

Material Strength Requirements

Material	Strength Requirement
Structural Steel:	
All Rolled Shapes	ASTM A572 (A992), Grade 50
Connection Materials	ASTM A36
Metal Deck	ASTM A611 or A653 w/ ASTM A653 G60 Galv.
Cast-In-Place Concrete:	
Foundations	4 ksi NWC
Slabs-On-Grade	4 ksi NWC
Formed Slabs	5 ksi NWC
Columns and Walls	8 ksi NWC (Foundation to 6th Floor)
Reinforcement	ASTM A615, Grade 60 Except all #11 Bars are Grade 75
Light Gage Framing	ASTM A653, Grade 50

Table 2: Material Strength Requirements as per drawing S001.

Deflection Criteria

Construction[1]	Live	Snow or Wind[f]	D + L[g]
Roof Members[e]:			
Supporting Plaster Ceiling	$\ell/360$	$\ell/360$	$\ell/240$
Supporting Non-Plaster Ceiling	$\ell/240$	$\ell/240$	$\ell/180$
Not Supporting Ceiling	$\ell/180$	$\ell/180$	$\ell/120$
Floor Members	$\ell/360$	-	$\ell/240$
Exterior Walls and Interior Partitions:			
With Brittle Finishes	-	$\ell/240$	-
With Flexible Finishes	-	$\ell/120$	-

Table 3: Deflection Limitations outlined by IBC 2003.

e. The above deflections do not ensure against ponding.
f. The wind is permitted to be taken as 0.7 times the "component and cladding" loads for the purpose of determining deflection limits herein.
g. For steel structural members, the dead load shall be taken as zero.

[1] Table 1604.3 Deflection Limits, 2003 International Building Code Portion of the 2005 Connecticut State Building Code

Dead & Live Loads

Level	Load Type	Design Dead Load (psf)	Design Super-Imposed Dead Load (psf)	Design Live Load (psf)	Live Load per ASCE 7 - 05(psf)
Foundation	Loading Dock	Varies on Mat Slab Thickness	40	100	-
Grade	Public	150	40	100	100
2nd to 5th	Parking	125	22	40	40
6th Terrace	Amenities	150	25	100	100
	Terrace Typ.	200	160	100	100
	Terrace Planters	200	400	100	100
	Large Tree Planters	250	620	100	-
7th	Residential	61	20	40	40
	Public	61	20	100	100
8th to 31st	Residential	61	20	40	40
	Public	61	20	100	100
Mechanical/Roof	Mechanical	61	20	40	-

Table 4: Dead & Live Load Schedule.

Note: According to Section 1606.1 in the International Building Code 2003, dead loads considered for design shall be the actual weight of materials and construction.

Occupancy/Function	psf	Occupancy/Function	psf
Corridor	100	Public Space	100
Storage (Light)	125	Lobby	100
Office	50	Terrace (Private, Public)	60, 100
Residential	40	Parking (Passenger Cars)	40

Table 5: Uniformly Distributed Live Loads from ASCE 7 Table 4-1.

Discussion of Wind Loading

According to the 2005 Connecticut State Building Code and the 2003 International Building Code, the wind loading on 360 State Street must be calculated using Section 6 of ASCE 7. Since this is the governing code which all others point to, ASCE 7 was also used for this analysis.

Wind Load Design Criteria	
Basic Wind Speed (3 s Gust)	110 mph
Wind Importance Factor	$I_w = 1.0$
Wind Exposure	B
Internal Pressure Coefficient (Enclosed Building)	GC_{pi} = + 0.18 windward = - 0.18 leeward

Table 6: Wind Design Criteria as outlined by drawing S001

Beginning with the design criteria outlined by the structural engineer's notes, the velocity pressures were determined by a series of factors including the exposure coefficients, K_z. The velocity pressures increase across the story height on the windward side however; the opposite side of the building experiences a uniform pressure. A quick summary of results can be seen in Table 7. The forces for each face were calculated for the full length of the building. High lateral forces were found in the East-to-West direction. These numbers may have been the deciding factor for the use of staggered steel trusses throughout the residential tower. In addition to the convenience they provide for a floor layout, the trusses allow more stability in the overall structure.

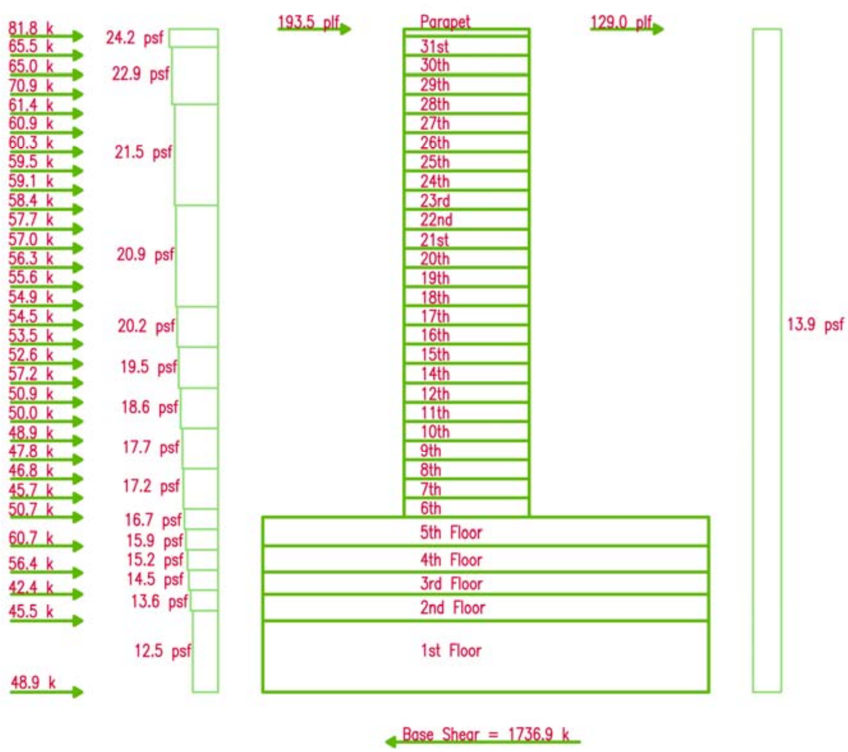


Figure 9: Wind Loading Diagram in East/West Direction.

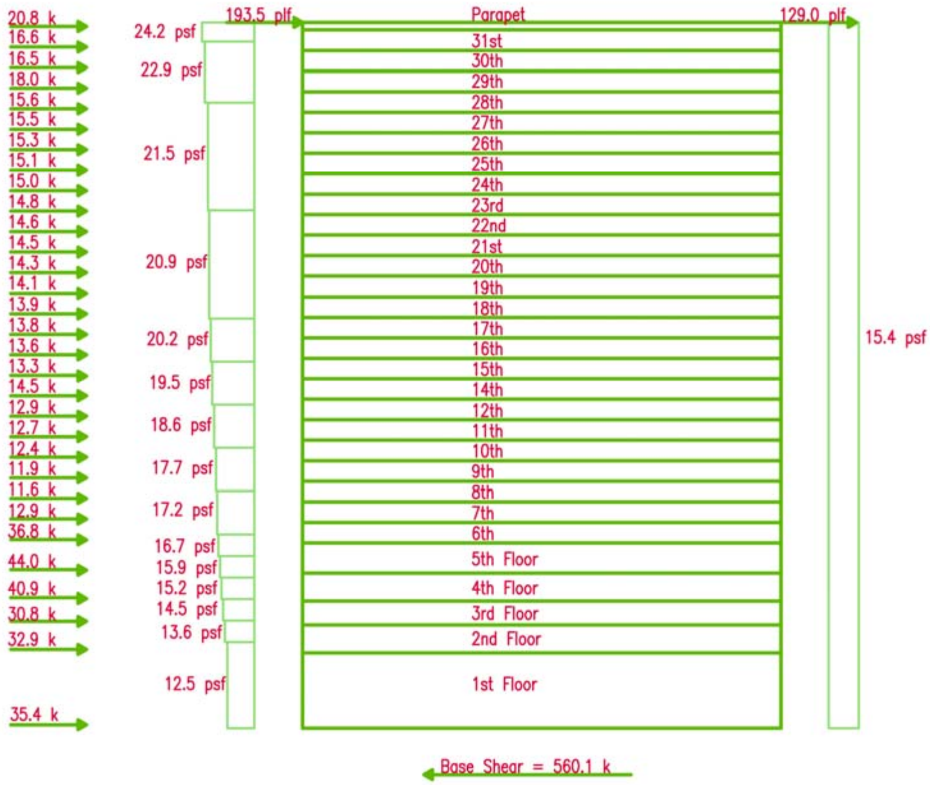


Figure 10: Wind Loading Diagram in North/South Direction.

While going through Section 6 of ASCE 7, it was required to define the building as rigid or flexible. To do this, the building's natural frequency must be determined. Generally, any frequency above 1 Hz is considered rigid however; upon calculating 360's frequency using Eq C6-15, it was found to be 0.237 Hz. Swaying in a building is a discomfort to its occupants. It may also cause structural damage and eventual collapse. Recalculating the frequency using Eq C6-16 and taking into account the concrete shear walls, the natural frequency was found to be 1.267 Hz. Though 360 has included staggered steel trusses in its frame, it might be advantageous to look into further solutions to add rigidity to the structure.

For future analysis, wind pressures on the roof should be considered. While 360's top two residential floors step back and a mechanical room is situated on the roof, potential uplift problems could be present. Furthermore, the State of Connecticut does not have many tall buildings such as 360 State Street that are not accounted for in its building code. It would be beneficial to look into other codes or design guides to determine lateral loading for taller buildings.

Location	Floor	Height Above Ground, z (ft)	Floor Height (ft)	K_z	q_z (psf)	Forces (k)		Shear (k)	
						E/W	N/S	E/W	N/S
Windward	Roof Parapet	338.583	3.5	1.4	36.86	-	-	-	-
	32	326.917	12	1.38	36.34	81.8	20.8	81.8	20.8
	31	317.25	9.67	1.37	36.07	65.5	16.6	147.3	37.4
	30	307.583	9.67	1.36	35.81	65.0	16.5	212.3	53.9
	29	296.917	10.67	1.345	35.41	70.9	18.0	283.2	71.9
	28	287.583	9.34	1.33	35.02	61.4	15.6	344.6	87.5
	27	278.25	9.34	1.32	34.76	60.9	15.5	405.5	103
	26	268.917	9.34	1.306	34.39	60.3	15.3	465.8	118.3
	25	259.583	9.34	1.29	33.97	59.5	15.1	525.3	133.4
	24	250.25	9.34	1.28	33.70	59.1	15.0	584.4	148.4
	23	240.917	9.34	1.265	33.31	58.4	14.8	642.8	163.2
	22	231.583	9.34	1.25	32.91	57.7	14.6	700.5	177.8
	21	222.25	9.34	1.235	32.52	57.0	14.5	757.5	192.3
	20	212.917	9.34	1.22	32.12	56.3	14.3	813.8	206.6
	19	203.583	9.34	1.205	31.73	55.6	14.1	869.4	220.7
	18	194.25	9.34	1.19	31.33	54.9	13.9	924.3	234.6
	17	187.917	9.34	1.18	31.07	54.5	13.8	978.8	248.4
	16	175.583	9.34	1.16	30.54	53.5	13.6	1032.3	262
	15	166.25	9.34	1.14	30.02	52.6	13.3	1084.9	275.3
	14	155.917	10.34	1.12	29.49	57.2	14.5	1142.2	289.8
	12	146.583	9.34	1.103	29.04	50.9	12.9	1193.1	302.7
	11	137.25	9.34	1.083	28.52	50.0	12.7	1243.1	315.4
	10	127.917	9.34	1.059	27.88	48.9	12.4	1291.9	327.8
	9	118.583	9.34	1.036	27.28	47.8	12.1	1339.8	339.9
	8	109.25	9.34	1.013	26.67	46.8	11.9	1386.5	351.8
	7	99.917	9.34	0.99	26.07	45.7	11.6	1432.2	363.4
	6	86.03	10.83	0.948	24.96	50.7	12.9	1482.9	376.3
	5	72.417	13.67	0.899	23.67	60.7	44.0	1543.7	420.3
	4	58.917	13.5	0.846	22.28	56.4	40.9	1600.1	461.2
	3	48.25	10.67	0.805	21.20	42.4	30.8	1642.5	492
	2	35.583	12.67	0.726	19.12	45.5	32.9	1688.0	524.9
	1	2.5	14.125	0.7	18.43	48.9	35.4	1736.9	560.3
Leeward	All				36.30				

Table 7: Summary of Wind Pressures & Forces.

Discussion of Seismic Loading

The State of Connecticut is not known for seismic activities though it is always good practice to design a building to withstand such movements. 360 State Street was primarily designed to resist movement from wind forces. Using Chapters 11 and 12 in ASCE 7 for this analysis, the base shear of the building was found to be roughly 1250 kips. This force is based on a total building weight of 93,993 kips. Compared to the information provided on the structural drawings, S001, the shear of 1822 kips was found based on a weight of 136,992 kips. The structural engineer's original estimate is more conservative. The building weight calculation includes slabs, columns, curtain walls, and shear walls however; superimposed dead loads were not accounted for. This may be the expected cause for the analysis results to vary by 31%.

Seismic Design Criteria			
$I_e = 1.0$	$T_s = 0.499$	$S_{ms} = 0.455$	x = 0.75
$S_s = 0.290$	R = 3	$S_{m1} = 0.204$	$H_n = 326.9$
$S_1 = 0.085$	$T_a = 1.54$	$S_{ds} = 0.303$	$C_s = 0.0133$
Soil Class D	Ct = 0.02	$S_{d1} = 0.136$	K = 1.52
Category C	$F_a = 1.568$	$F_v = 2.4$	**V = 1250 k**

Table 8: Seismic Design Criteria used for Calculations

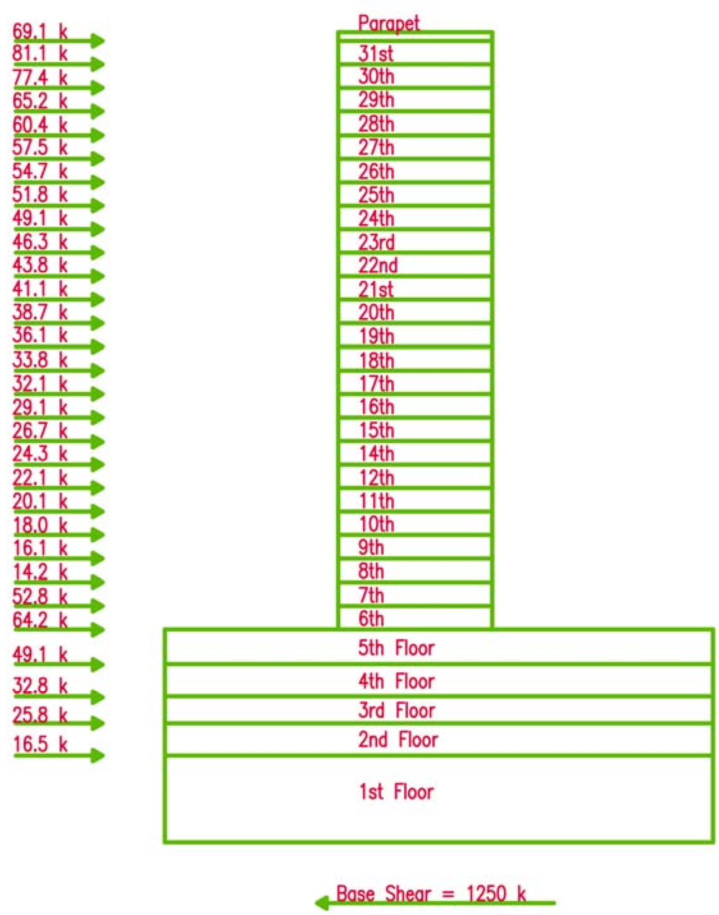


Figure 11: East/West Seismic Loading Diagram

Discussion of Snow Loading

Snow loading for 360 State Street was analyzed using chapter 7 of ASCE 7. The main roof of the building will probably see the least snow accumulation compared to the lower roof. It is fully exposed and has access to the mechanical room. This roof also has the smallest area and is right above heated living spaces. The lower roof, as known as the terraced level, is partially sheltered due to the residential tower. It has the largest area and is featured above the open air parking garage. While the main roof does not support many functions, the terrace supports recreational activities and the landscaped garden. A snow load of 21 psf should not add too much burden.

A calculation for snow drift was also completed and can be seen in Appendix D. Since the building is partially exposed and the heights of the roofs vary significantly, drift may not be a major issue. Any snow blown off the roof could miss the lower levels entirely however; the parapet walls running along the perimeter of the building could cause problems with snow accumulation.

Although New Haven is located in the snowy Northeast, it is also located on the coast which tends to be much warmer during the winter time. Typically the city will see more rain than snow but further analysis may include ponding instabilities and wet-snow loads.

Snow Load Design Criteria	
Flat Roof Snow Load	$P_g = 30$ psf
Snow Exposure Factor	$C_e = 1.0$
Snow Load Importance Factor	$I_s = 1.0$
Thermal Factor	$C_r = 1.0$
$P_f = 0.7C_eC_rI_sP_g$ (Eq 7-1 ASCE)	**$P_f = 21$ psf**

Table 9: Snow Design Criteria as outlined on drawing S001

Discussion of Gravity System Spot Checks

After completing an initiate analysis of the building's loadings, a variety of spot checks were concluded on 360 State Street focusing on the gravity systems. The first spot check looked at the adequacy of a W10x45 on the thirty-second floor. This beam supports the 8" hollow core plank roof construction and ties into an exterior column and a staggered truss. The analysis was conducted using 1.2D + 1.6L for the loading and accounted for superimposed dead loads. The beam was found to have adequate strength and could carry 25% more if necessary. The controlling factor in the beam's design is estimated to be governed by deflection. Since the beam does not support a ceiling, $\ell/120$ was used to establish a deflection limit of 2.37". The deflection of a D + L load was found to be 1.61". Although the beam passes, additional strength may be necessary for potential snow loads.

A steel column was also checked on the thirty-second floor. A W14x99 was analyzed to see if it can carry the roof load. The majority of columns are fairly short within 360 spanning typically a single level. This provides more stability in a column as an axial load is applied. The W14 was found to be more than sufficient for its loading by 98%. The reasoning for this figure to appear so high may have resulted in something being excluded. This column may be supporting more than just the roof construction. For example, it may tie into a staggered truss and carry additional loading from the system.

Moving lower into the structure, the punching shear was calculated for column 217 which is located on the ground floor near the center of the building. The column has a strength of 8,000 psi while the floor slab that it passes through only has a strength of 6,000 psi. Since the slab was designed as a flat plate system, the analysis was conducted to ensure the column's loading was not overbearing on the slab. Both pass with no additional reinforcing required for shear however; top bars across the column are present in details on the original structural drawings. It can be estimated that special shear requirements were necessary due to the function of the space. For example, the column in question is located in the retail space and the surrounding slab may be expected to carry additional stationary loads. The magnitude of these loads was probably estimated to be those of a grocery store which the owner hopes to move into the space.

In a nearby panel, the same slab was also checked for sufficient reinforcing. Using the Direct Design Method, the distribution of moments was determined across a beam spanning between two columns. The amount of reinforcing calculated was very close to the amount specified in the structural drawings. The designed slab has an additional bar within the set span of distribution and has a smaller spacing. This configuration is slightly more conservative and may include decisions that were based on design experience of such structural systems.

For more information, see Appendix F for calculations.

CONCLUSIONS

360 State Street is designed with a wide range of structural systems. Beginning with the foundations, mat slabs coupled with footings and piles support the entirety of the building. Concrete columns and beams complete the base with shear walls providing lateral stability. Steel shapes gives rise to the residential tower. Staggered trusses and cross-bracing complete the skeleton of the building. Last but not least, a variety of floor systems support the interior functions of the building. All in all, these structural systems were analyzed to determine if sufficient strength was achieved in the original design.

Several loading types were taken into consideration. The affects of wind on the structure were the dominating lateral forces that contributed to the design of shear walls and diagonal bracing. Seismic forces also contributed to the design of lateral systems although the building is not situated in a seismically activity location. In addition to calculating the overall building weight, snow loads were considered and gravity system spot checks were conducted. Overall, 360 State Street was designed by its structural engineer with sufficient strength provided in each member.

For further analysis, it would be advantageous to look into why some of the results varied in terms of additional available strength. Perhaps some loads were not calculated accurately or some were left out. It is not evident if safety factors were included or if some of the design decisions were based on previous experience with similar structures. Furthermore, analysis on soil composition should be conducted to decide if an alternate solution was available for deeper foundations. This could increase the amount of rentable space available to the owner. Considering the building's location next to a train station, a study in vibrations could reveal further requirements of strength in the structure.

APPENDIX A – FRAMING PLANS & ELEVATIONS

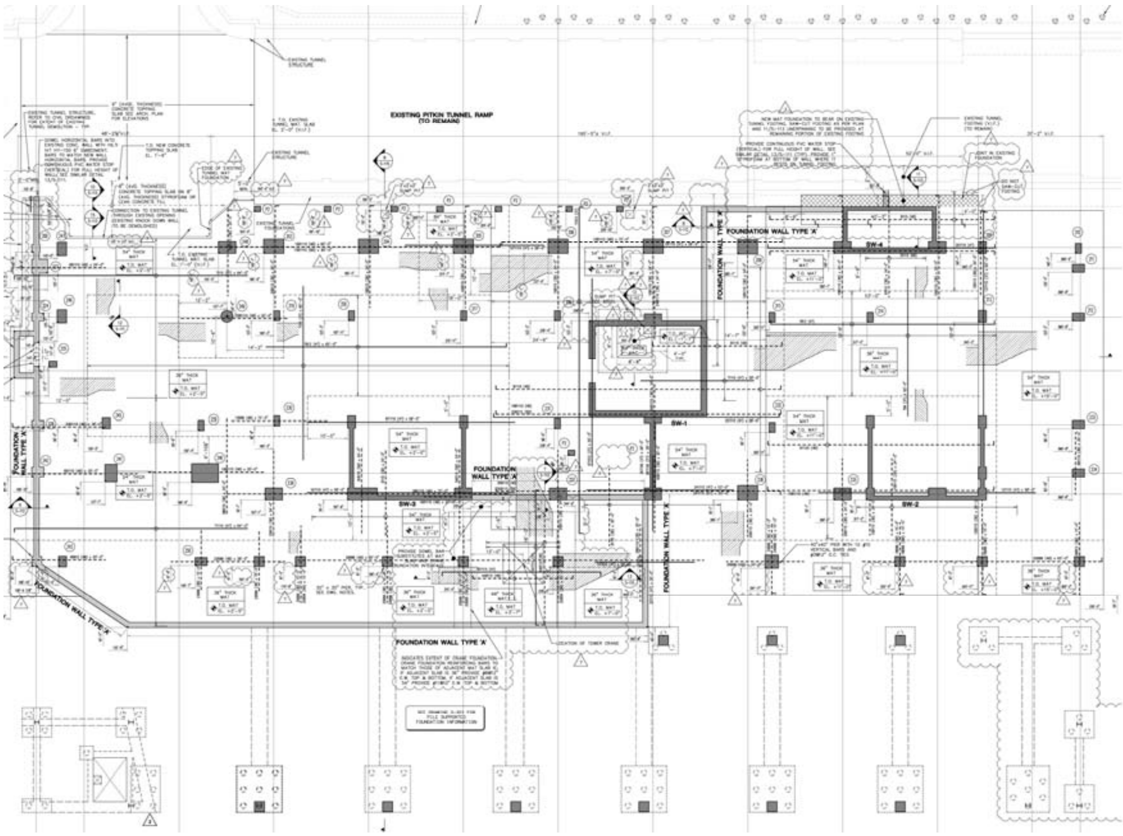


Figure A.1 Foundation Plan, Shear Walls are Shaded

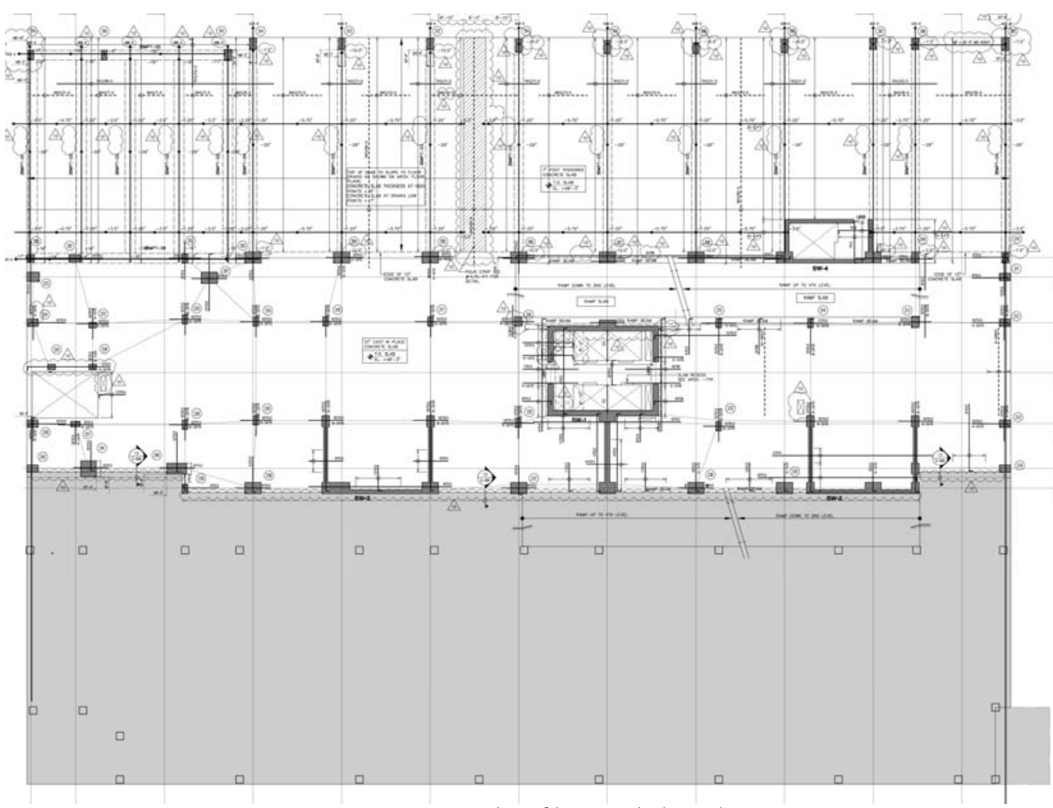


Figure A.2 Second - Fifth Typical Floor Plan

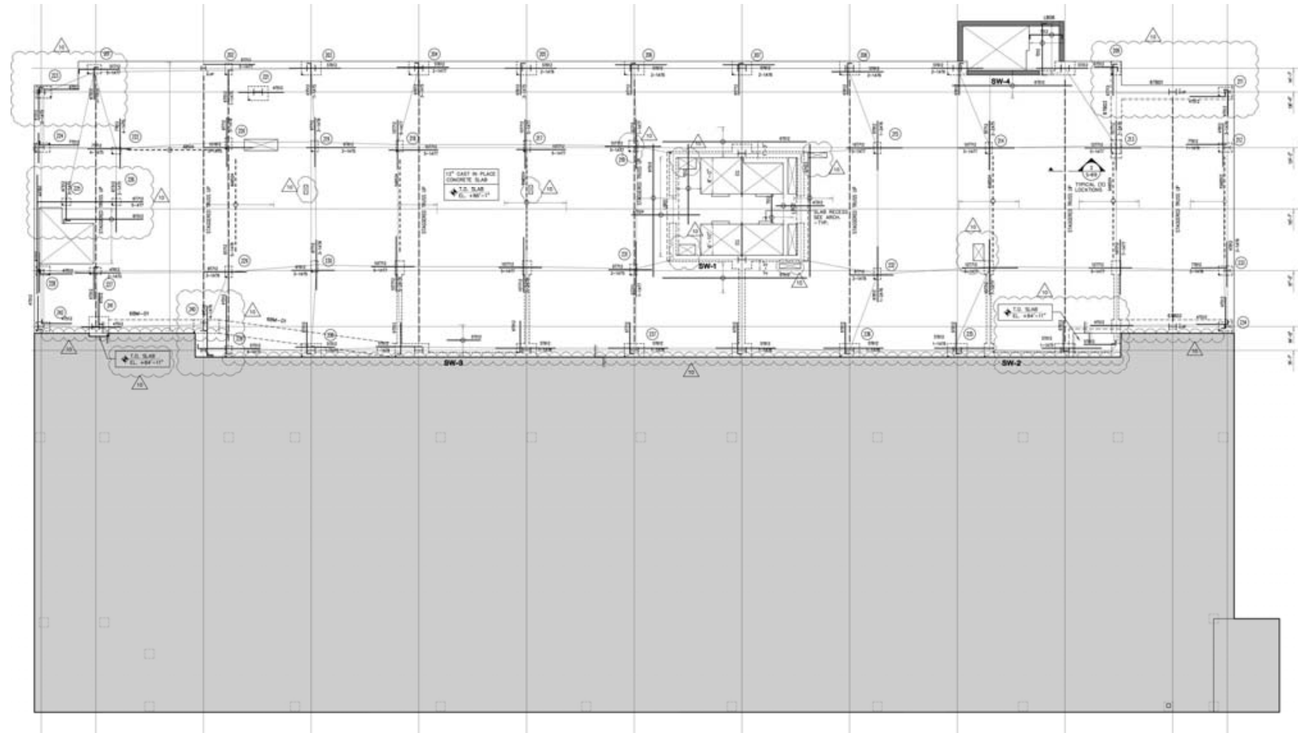


Figure A.3 Terrace & Sixth Floor Plan

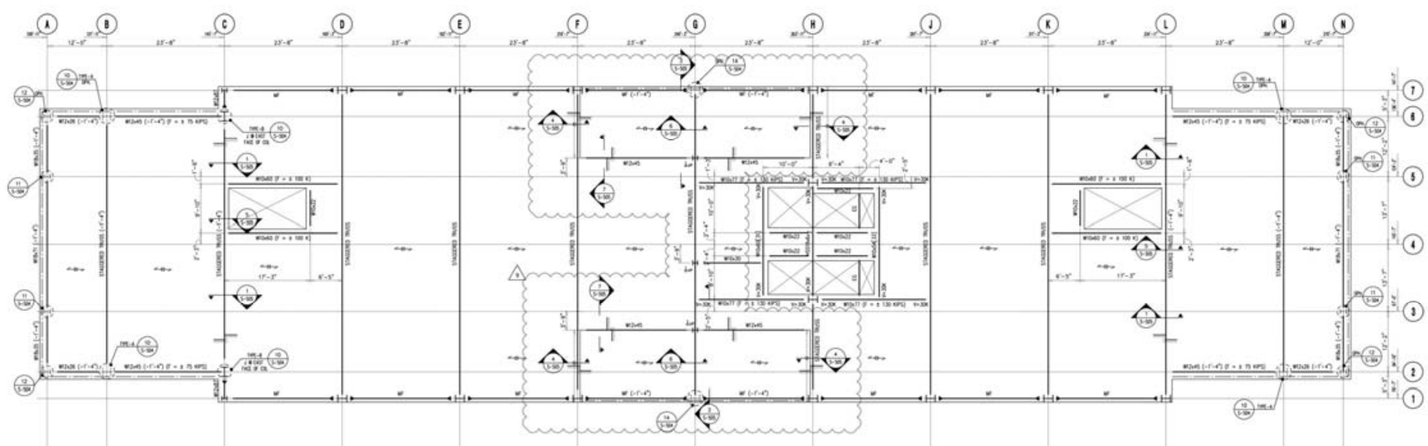


Figure A.4 Typical Floor Plan for Residential Tower

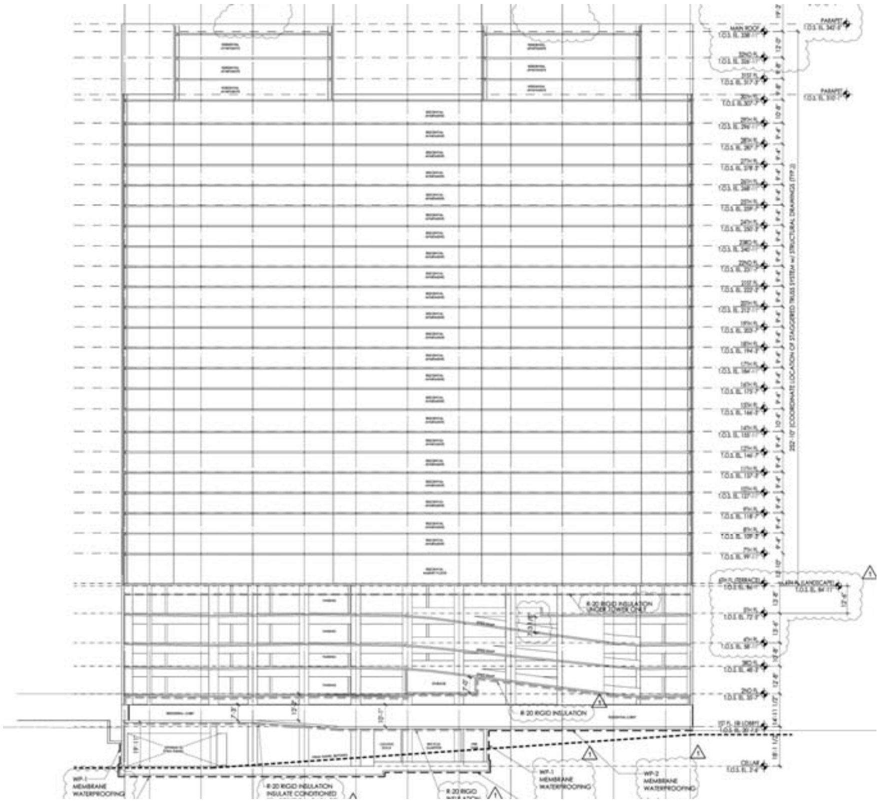


Figure A.5 North/South Building Elevation

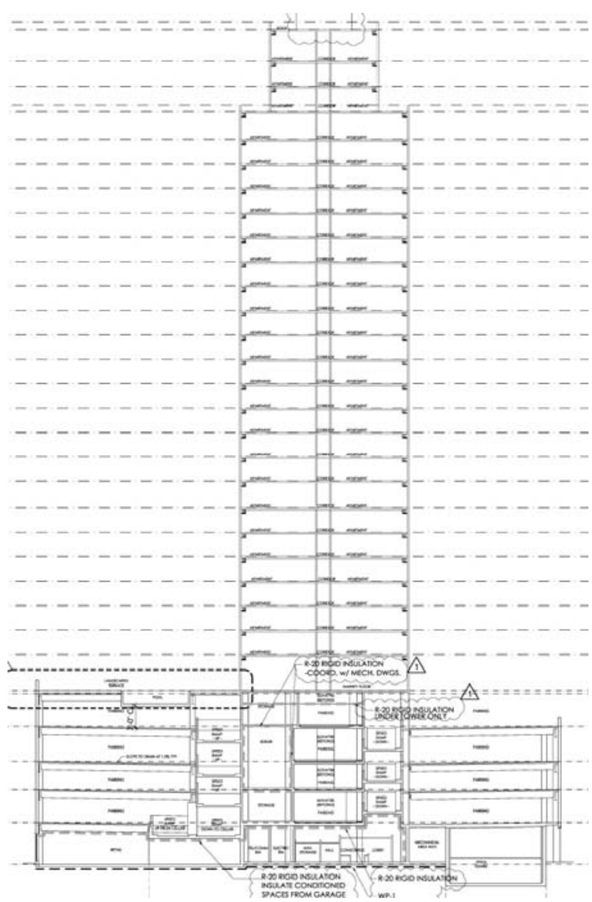


Figure A.6 East/West Building Elevation

Wind Load Design Criteria			
Basic Wind Speed (3 s Gust)	110 mph	K_d = 0.85	G = 0.85
Wind Importance Factor	I_w = 1.0	K_{zt} = 1.0	
Wind Exposure	B	Design Category	II
Internal Pressure Coefficient (Enclosed Building)	GC_{pi} = + 0.18 windward = - 0.18 leeward	Combined Net Pressure Coefficient	GC_{pn} = + 1.5 windward parapet = - 1.0 leeward parapet
p_p = 55.29 psf windward parapet -36.86 leeward parapet	F_p = 193.5 plf windward parapet -129.0 plf leeward parapet		

Table B.1 Wind Design Criteria According to ACSE 7 – 05

Location	Floor	Height Above Ground, z (ft)	Floor Height (ft)	K_z	q_z (psf)	External Pressure qGC_p (psf)	Internal Pressure q_hGC_{pi} (psf)	Net Pressure p (psf)	
								+ GC_{pi}	- GC_{pi}
Windward	Parapet	338.58	3.5	1.4	36.86	-	-	-	-
	32	326.92	12	1.38	36.33	24.71	-6.54	18.17	31.25
	31	317.25	9.67	1.37	36.07	24.53	-6.54	17.99	31.07
	30	307.58	9.67	1.36	35.81	24.35	-6.54	17.81	30.89
	29	296.92	10.67	1.345	35.41	24.08	-6.54	17.54	30.62
	28	287.58	9.34	1.33	35.02	23.81	-6.54	17.27	30.35
	27	278.25	9.34	1.32	34.76	23.63	-6.54	17.09	30.17
	26	268.92	9.34	1.306	34.39	23.38	-6.54	16.84	29.92
	25	259.58	9.34	1.29	33.97	23.10	-6.54	16.56	29.64
	24	250.25	9.34	1.28	33.70	22.92	-6.54	16.38	29.46
	23	240.92	9.34	1.265	33.31	22.65	-6.54	16.11	29.19
	22	231.58	9.34	1.25	32.91	22.38	-6.54	15.84	28.92
	21	222.25	9.34	1.235	32.52	22.11	-6.54	15.57	28.65
	20	212.92	9.34	1.22	32.12	21.84	-6.54	15.30	28.38
	19	203.58	9.34	1.205	31.73	21.57	-6.54	15.03	28.11
	18	194.25	9.34	1.19	31.33	21.31	-6.54	14.77	27.85
	17	187.92	9.34	1.18	31.07	21.13	-6.54	14.59	27.67
	16	175.58	9.34	1.16	30.54	20.77	-6.54	14.23	27.31
	15	166.25	9.34	1.14	30.02	20.41	-6.54	13.87	26.95
	14	155.92	10.34	1.12	29.49	20.05	-6.54	13.51	26.59
	12	146.58	9.34	1.103	29.04	19.75	-6.54	13.21	26.29
	11	137.25	9.34	1.083	28.51	19.39	-6.54	12.85	25.93
	10	127.92	9.34	1.059	27.88	18.96	-6.54	12.42	25.50
	9	118.58	9.34	1.036	27.28	18.55	-6.54	12.01	25.09
	8	109.25	9.34	1.013	26.67	18.14	-6.54	11.60	24.68
	7	99.92	9.34	0.99	26.07	17.73	-6.54	11.19	24.27
	6	86.03	10.83	0.948	24.96	16.97	-6.54	10.43	23.51
	5	72.42	13.67	0.899	23.67	16.10	-6.54	9.56	22.64
	4	58.92	13.5	0.846	22.27	15.15	-6.54	8.61	21.69
	3	48.25	10.67	0.805	21.20	14.41	-6.54	7.87	20.95
	2	35.58	12.67	0.726	19.12	13.00	-6.54	6.46	19.54
	1	2.50	14.125	0.7	18.43	12.53	-6.54	5.99	19.07
Leeward	All				36.30	-13.88	-6.54	-20.42	-7.34

Table B.2 Design Wind Pressures in the E-W Direction

Location	Floor	Height Above Ground, z (ft)	Floor Height (ft)	K_z	q_z (psf)	External Pressure qGC_p (psf)	Internal Pressure q_hGC_{pi} (psf)	Net Pressure p (psf)	
								+ GC_{pi}	- GC_{pi}
Windward	Parapet	338.58	3.5	1.4	36.86	-	-	-	-
	32	326.92	12	1.38	36.33	24.71	-6.54	18.17	31.25
	31	317.25	9.67	1.37	36.07	24.53	-6.54	17.99	31.07
	30	307.58	9.67	1.36	35.81	24.35	-6.54	17.81	30.89
	29	296.92	10.67	1.345	35.41	24.08	-6.54	17.54	30.62
	28	287.58	9.34	1.33	35.02	23.81	-6.54	17.27	30.35
	27	278.25	9.34	1.32	34.76	23.63	-6.54	17.09	30.17
	26	268.92	9.34	1.306	34.39	23.38	-6.54	16.84	29.92
	25	259.58	9.34	1.29	33.97	23.10	-6.54	16.56	29.64
	24	250.25	9.34	1.28	33.70	22.92	-6.54	16.38	29.46
	23	240.92	9.34	1.265	33.31	22.65	-6.54	16.11	29.19
	22	231.58	9.34	1.25	32.91	22.38	-6.54	15.84	28.92
	21	222.25	9.34	1.235	32.52	22.11	-6.54	15.57	28.65
	20	212.92	9.34	1.22	32.12	21.84	-6.54	15.30	28.38
	19	203.58	9.34	1.205	31.73	21.57	-6.54	15.03	28.11
	18	194.25	9.34	1.19	31.33	21.31	-6.54	14.77	27.85
	17	187.92	9.34	1.18	31.07	21.13	-6.54	14.59	27.67
	16	175.58	9.34	1.16	30.54	20.77	-6.54	14.23	27.31
	15	166.25	9.34	1.14	30.02	20.41	-6.54	13.87	26.95
	14	155.92	10.34	1.12	29.49	20.05	-6.54	13.51	26.59
	12	146.58	9.34	1.103	29.04	19.75	-6.54	13.21	26.29
	11	137.25	9.34	1.083	28.51	19.39	-6.54	12.85	25.93
	10	127.92	9.34	1.059	27.88	18.96	-6.54	12.42	25.50
	9	118.58	9.34	1.036	27.28	18.55	-6.54	12.01	25.09
	8	109.25	9.34	1.013	26.67	18.14	-6.54	11.60	24.68
	7	99.92	9.34	0.99	26.07	17.73	-6.54	11.19	24.27
	6	86.03	10.83	0.948	24.96	16.97	-6.54	10.43	23.51
	5	72.42	13.67	0.899	23.67	16.10	-6.54	9.56	22.64
	4	58.92	13.5	0.846	22.27	15.15	-6.54	8.61	21.69
	3	48.25	10.67	0.805	21.20	14.41	-6.54	7.87	20.95
	2	35.58	12.67	0.726	19.12	13.00	-6.54	6.46	19.54
	1	2.50	14.125	0.7	18.43	12.53	-6.54	5.99	19.07
Leeward	All				36.30	-15.43	-6.54	-21.97	-8.89

Table B.3 Design Wind Pressures in the N-S Direction

Flexibility Check				
Eq C6-16 ASCE				
$n_1 = 385c_w^{1/2}/H$			1.267	Hz
$c_w = 100/A_b$å$(H/h_i)^2[A_i/(1+0.83(h_i/D_i)^2]$ =			1.156	
in E-W direction				
SW1			å$(H/h_i)^2[A_i/(1+0.83(h_i/D_i)^2]$	
Area	78.8	ft^2	413.8	
Height	86	ft		
Length	59	ft		
SW2				
Area	30	ft^2	55.4	
Height	86	ft		
Length	30	ft		
SW3				
Area	31.1	ft^2	61.2	
Height	86	ft		
Length	31	ft		
SW4				
Area	46.5	ft^2	175.0	
Height	86	ft		
Length	47	ft		
Eq C6-15 ASCE				
$n_1 = 43.5/H^{0.9}$			0.237	Hz

Table B.4 Flexibility Calculation

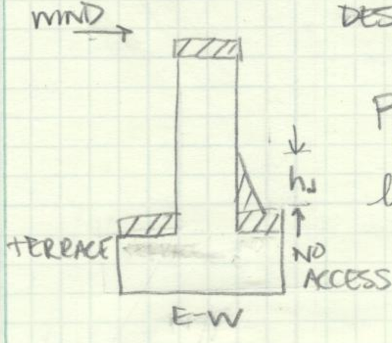
Seismic Design Criteria	
Seismic Important Factor	I_e = 1.0
Mapped Spectral Response Accel. For Short Periods	S_s = 0.290
Mapped Spectral Response Accel. For 1-Second Periods	S_1 = 0.085
Site Class	D
Design Spectral Response Accel. For Short Periods	S_{ds} = 0.303
Design Spectral Response Accel. For 1-Second Periods	S_{d1} = 0.136
Seismic Use Group	II
Seismic Design Category	C
Basic Seismic-Force-Resisting System	Structural Steel Systems Not Specifically Detailed for Seismic Resistance
Design Base Shear	V = 1,822 kips
Seismic Response Coefficient	C_s = 0.0133
Response Modification Factor	R = 3
Analysis Procedure	Equivalent Lateral Force Method

Table C.1 Seismic Design Criteria as outlined on drawing S001

Level	Story Weight w_x (kips)	Height h_x (ft)	$w_x h_x^k$	Lateral Force F_x (kips)	Story Shear V_x (kips)
32	1588	326.92	10538905	69.1	69.1
31	1951	317.25	12370514	81.1	150.1
30	1953	307.58	11814218	77.4	227.5
29	1735	296.92	9947290	65.2	292.7
28	1689	287.58	9224646	60.4	353.2
27	1689	278.25	8773464	57.5	410.7
26	1691	268.92	8339948	54.7	465.3
25	1691	259.58	7903938	51.8	517.1
24	1694	250.25	7489314	49.1	566.2
23	1694	240.92	7068901	46.3	612.5
22	1699	231.58	6676480	43.8	656.3
21	1699	222.25	6271810	41.1	697.4
20	1707	212.92	5903550	38.7	736.1
19	1707	203.58	5514684	36.1	772.2
18	1713	194.25	5153071	33.8	806.0
17	1713	187.92	4899884	32.1	838.1
16	1719	175.58	4434955	29.1	867.1
15	1719	166.25	4081630	26.7	893.9
14	1722	155.92	3708781	24.3	918.2
12	1722	146.58	3376605	22.1	940.3
11	1725	137.25	3060609	20.1	960.4
10	1725	127.92	2749920	18.0	978.4
9	1730	118.58	2457877	16.1	994.5
8	1730	109.25	2169935	14.2	1008.7
7	7354	99.92	8053330	52.8	1061.5
6	11229	86.03	9795102	64.2	1125.7
5	11171	72.42	7499800	49.1	1174.8
4	10208	58.92	5008485	32.8	1207.7
3	10889	48.25	3943700	25.8	1233.5
2	11048	35.58	2518676	16.5	1250.0

Table C.2 Seismic Lateral Force Calculation, 1st & Cellar Levels Excluded

SNOW DRIFT CALCULATION



DESIGNED ACCORDING TO ASCE 7 § 7.7.1

$P_g = 30$ psf

l_u = LENGTH OF ROOF UPWIND OF DRIFT = 75' (E-W WIND)

$\gamma = 0.13 p_g + 14 < 30$ psf MAX INTENSITY OF DRIFT LOAD

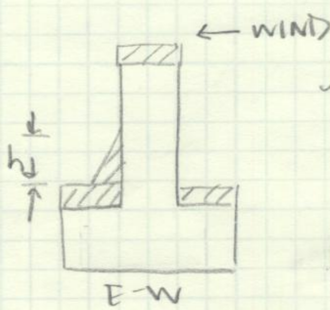
$\gamma = 0.13(30\text{psf}) + 14 = 17.9$ psf < 30 psf → OKAY

h_d = HEIGHT OF SNOW DRIFT

$h_d = 0.43 \sqrt[3]{l_u}\sqrt[4]{P_g + 10} - 1.5$

$= 0.43 \sqrt[3]{75'}\sqrt[4]{30+10} - 1.5$

$= 3.06'$



$l_u = 80'$ (W-E WIND)

$\gamma = 17.9$ psf < 30 psf → OKAY

$h_d = 0.43 \sqrt[3]{80'}\sqrt[4]{30+10} - 1.5$

$= 3.16'$

- ONLY ONE SIDE IS DESIGNED FOR ACCESS, THEREFORE DRIFT SHOULDN'T BE AN ISSUE
- THE NO-ACCESS LOWER ROOF IS PARTIALLY COVERED BY AN ADJACENT, TALLER STRUCTURE. DRIFT MIGHT NOT BE AN ISSUE.

1/1

Figure D.1 Snow Drift Hand Calculation

APPENDIX E – BUILDING WEIGHT CALCULATION

Floor/ Function	Area (ft^2)	Design Dead Load (psf)	Floor Weight (lbs)	Floor Height (ft)	∑ Steel Column Weight (plf)	Column Weight (lbs)	Curtain Wall Length (ft)	Curtain Wall Weight (lbs)	∑ Shear Wall Length (ft)	∑ Shear Wall Thickness (ft)	Shear Wall Weight (lbs)
32nd				12	1484	17808	510	459000	-	-	-
Mech	11112	100	1111200								
31st				9.67	2683	25945	510	369877.5	-	-	-
Priv Terrace	3715	100	371500								
Residential	10365	100	1036500								
Corridor	1110	100	111000								
Elev Lobby	510	100	51000								
30th				9.67	2683	25945	510	369877.5	-	-	-
Priv Terrace	3715	100	371500								
Residential	10365	100	1036500								
Corridor	1110	100	111000								
Elev Lobby	510	100	51000								
29th				10.67	3356	35809	652	521763	-	-	-
Residential	10457	100	1045700								
Corridor	1014	100	101400								
Elev Lobby	510	100	51000								
28th				9.34	3356	31345	652	456726	-	-	-
Residential	10457	100	1045700								
Corridor	1014	100	101400								
Elev Lobby	510	100	51000								
27th				9.34	3653	34119	652	456726	-	-	-
Residential	10457	100	1045700								
Corridor	1014	100	101400								
Elev Lobby	510	100	51000								
26th				9.34	3653	34119	652	456726	-	-	-
Residential	10457	100	1045700								
Corridor	1014	100	101400								
Elev Lobby	510	100	51000								
25th				9.34	3840	35866	652	456726	-	-	-
Residential	10457	100	1045700								
Corridor	1014	100	101400								
Elev Lobby	510	100	51000								
24th				9.34	3840	35866	652	456726	-	-	-
Residential	10457	100	1045700								
Corridor	1014	100	101400								
Elev Lobby	510	100	51000								
23rd				9.34	4225	39462	652	456726	-	-	-
Residential	10457	100	1045700								
Corridor	1014	100	101400								
Elev Lobby	510	100	51000								
22nd				9.34	4225	39462	652	456726	-	-	-
Residential	10457	100	1045700								
Corridor	1014	100	101400								
Elev Lobby	510	100	51000								
21st				9.34	4731	44188	652	456726	-	-	-
Residential	10457	100	1045700								
Corridor	1014	100	101400								
Elev Lobby	510	100	51000								
20th				9.34	4731	44188	652	456726	-	-	-
Residential	10457	100	1045700								
Corridor	1014	100	101400								
Elev Lobby	510	100	51000								

19th				9.34	5633	52612	652	456726	-	-	-
Residential	10457	100	1045700								
Corridor	1014	100	101400								
Elev Lobby	510	100	51000								
18th				9.34	5633	52612	652	456726	-	-	-
Residential	10457	100	1045700								
Corridor	1014	100	101400								
Elev Lobby	510	100	51000								
17th				9.34	6282	58674	652	456726	-	-	-
Residential	10457	100	1045700								
Corridor	1014	100	101400								
Elev Lobby	510	100	51000								
16th				9.34	6282	58674	652	456726	-	-	-
Residential	10457	100	1045700								
Corridor	1014	100	101400								
Elev Lobby	510	100	51000								
15th				9.34	6898	64427	652	456726	-	-	-
Residential	10457	100	1045700								
Corridor	1014	100	101400								
Elev Lobby	510	100	51000								
14th				9.34	6898	64427	652	456726	-	-	-
Residential	10457	100	1045700								
Corridor	1014	100	101400								
Elev Lobby	510	100	51000								
12th				9.34	7206	67304	652	456726	-	-	-
Residential	10457	100	1045700								
Corridor	1014	100	101400								
Elev Lobby	510	100	51000								
11th				9.34	7206	67304	652	456726	-	-	-
Residential	10457	100	1045700								
Corridor	1014	100	101400								
Elev Lobby	510	100	51000								
10th				9.34	7525	70284	652	456726	-	-	-
Residential	10457	100	1045700								
Corridor	1014	100	101400								
Elev Lobby	510	100	51000								
9th				9.34	7525	70284	652	456726	-	-	-
Residential	10457	100	1045700								
Corridor	1014	100	101400								
Elev Lobby	510	100	51000								
8th				9.34	8020	74907	652	456726	-	-	-
Residential	10457	100	1045700								
Corridor	1014	100	101400								
Elev Lobby	510	100	51000								
7th				9.34	8020	74907	652	456726	-	-	-
Residential	10457	100	1045700								
Corridor	1014	100	101400								
ElevLobby	510	100	51000								
6th				10.83	9175	99365	652	529587	-	-	-
Residential	2976	150	446400								
Corridor	987	150	148050								
Elev Lobby	510	150	76500								
Storage	721	150	108150		Conc.						
Amenities	12325	150	1848750		Column						
Pub Terrace	20486	200	4097200		Area (ft^2)						
5th				13.67	309.6	634835	968	992442	343.3	4.3	3026928
Elev Lobby	510	125	63750								

Storage	2856	125	357000								
Mechanical	1213	125	151625								
Parking	48021	125	6002625								
4th				13.5	309.6	626940	968	980100	343.3	4.3	2989285
Elev Lobby	510	125	63750								
Storage	2856	125	357000								
Mechanical	1213	125	151625								
Parking	48021	125	6002625								
3rd				10.67	309.6	495515	968	774642	343.3	4.3	2362642
Elev Lobby	510	125	63750								
Storage	2856	125	357000								
Mechanical	1213	125	151625								
Parking	48021	125	6002625								
2nd				12.67	309.6	588395	968	919842	343.3	4.3	2805499
Elev Lobby	510	125	63750								
Storage	2856	125	357000								
Mechanical	1213	125	151625								
Parking	48021	125	6002625								
1st*			* not included in seismic calculations								
Retail	14416	150	2162400	14.1	386.7	817871	968	1023660	343.3	4.3	3122142
Corridor	2324	150	348600								
Mainten	7299	150	1094850								
Office	685	150	102750								
MEP	9303	150	1395450								
Lobby	6534	150	980100								
Cellar*	* not included in seismic calculations			17.3	386.9	1004006	968	1255980	343.3	4.3	3830713
Storage	119821	850	101847850								
			Total from Slabs			Total from Columns		Total Curtain Walls			Total Shear Walls
			63634450 lbs			3665583 lbs		15508377 lbs			11184354 lbs

Total Building Weight: 93,993 kips

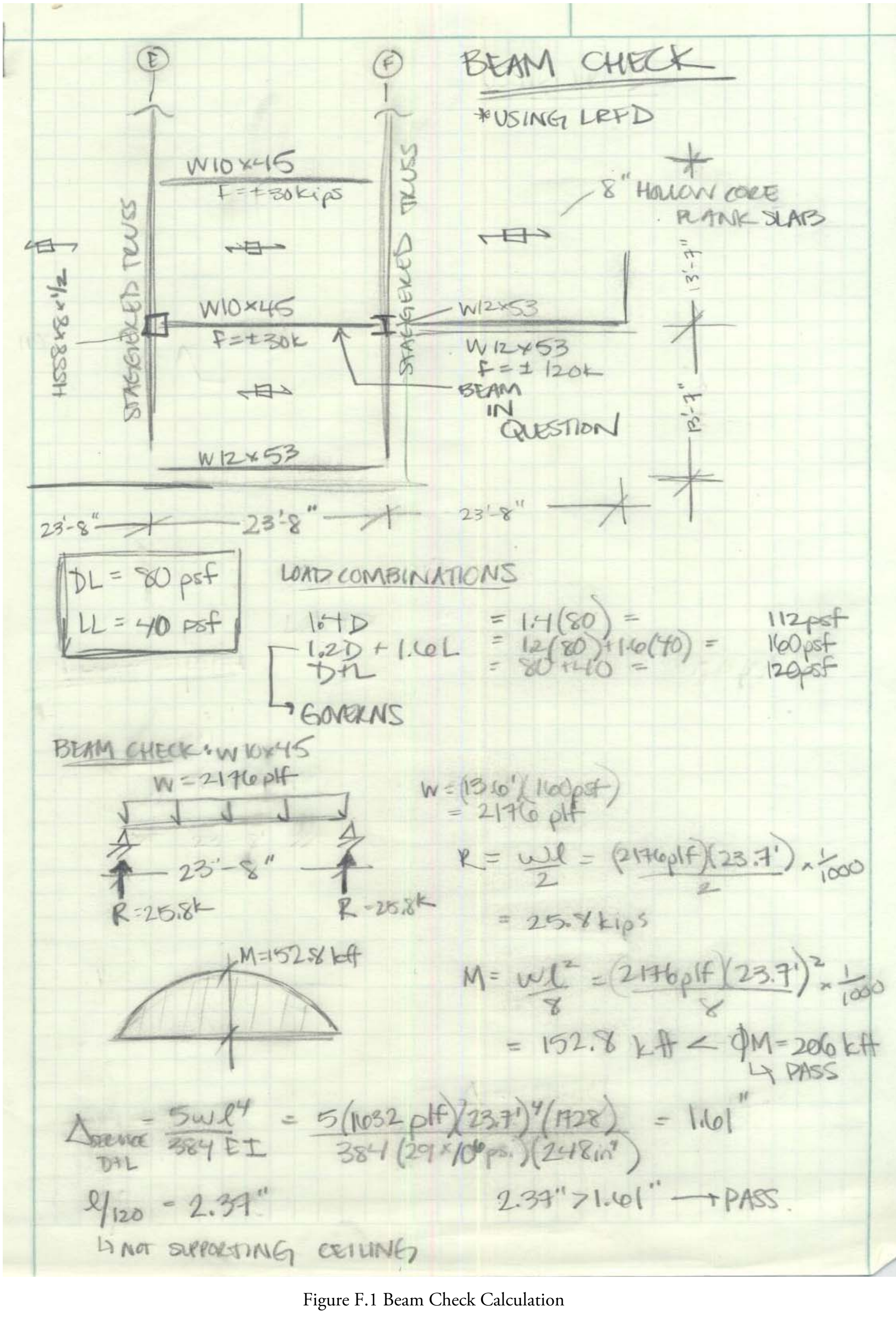


Figure F.1 Beam Check Calculation

COLUMN CHECK $\boxed{K-1}$ 32nd FLOOR

TRIB AREA = 23.7' × 8.75' = 207.4 sf

w_L = 40 psf $\qquad$ l = 12'

w_D = 80 psf

$1.2D + 1.6L = 1.2(80\,psf)(207.4\,sf) + 1.6(40\,psf)(207.4\,sf)$
$= 33.2^k = P_u$

W14x99
$A = 29.1\,in^2$ $\qquad$ $I_y = 402\,in^4$ $\qquad$ $K = 1.0$
$I_x = 1110\,in^4$ $\qquad$ $r_y = 3.71\,in$
$r_x = 6.17\,in$

$$\frac{KL}{r_x} = \frac{(1.0)(12')(12'')}{6.17\,in} = 23.3$$

$$\frac{KL}{r_y} = \frac{(1.0)(12')(12'')}{3.71\,in} = 38.8 \rightarrow \text{GOVERNS}$$

$$\frac{KL}{r} \le 4.71\sqrt{E/F_y} = 4.71\sqrt{\frac{29000}{50}\,ksi} = 113 > 38.8$$

↳ INELASTIC BEHAVIOR

$$F_e = \frac{\pi^2 E}{(KL/r)^2} = \frac{\pi^2(29000\,ksi)}{(38.8)^2} = 190.1\,ksi$$

$$F_{cr} = \left[0.658^{F_y/F_e}\right]F_y = \left[0.658^{\frac{50}{190.1\,ksi}}\right]50 = 44.8\,ksi$$

$$\phi P_n = \phi F_{cr} A_g = 0.9(44.8\,ksi)(29.1\,in^2) = 1173.3^k$$

$\phi P_n > P_u \rightarrow$ OKAY

TABLE 4-22, 4-1 ASIC

COLUMN PASSES

$\phi F_{cr} = 40.3\,ksi$

$\phi P_n = 1170^k \approx 1173.3^k$

SUFFICIENT STRENGTH IS AVAILABLE IN ORIGINAL DESIGN

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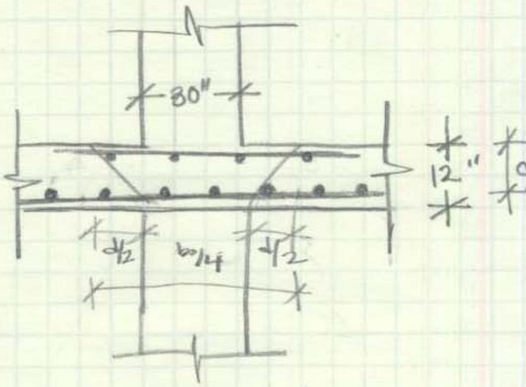
Figure F.2 Column Check Calculation

PUNCHING SHEAR CHECK

COLUMN $\boxed{217}$ INTERIOR
30" × 18" (6) #9
SLAB t = 12"
w/ #5 @ 12" O.C. BOTT
#6 @ 12" O.C. TOP



$f'_c = 8$ ksi

#5 $\phi = 0.625"$
$w_u = 388$ psf

$V_c = 4\sqrt{f'_c}\, b_o d$

$d = 12" - 0.75" - \frac{1}{2}(0.625") = 10.9375"$

$\frac{b_o}{4} = 2\left(\frac{d}{2}\right) + 30" = 10.9375" + 30" = 40.9375"$

$b_o = 40.9375"(4) = 163.75"$

$V_c = 4\sqrt{8000\text{psi}}\,(163.75")(10.9375") = 640.8^k$

$V_c' = \left(\frac{\alpha_s d}{b_o} + 2\right)\sqrt{f'_c}\, b_o d$ WHERE $\alpha_s = 40$ FOR INTERIOR COLUMNS

$V_c' = \left(\frac{40(10.9375")}{163.75"} + 2\right)\sqrt{8000\text{psi}}\,(163.75")(10.9375")$

$= 748.4^k$

$V_c \leq V_c' \longrightarrow$ OKAY.

$\phi V_c \geq V_u$

$\phi V_c = (0.75)(640.8^k) = 480.6^k$

$V_u = (0.388\text{ ksf})\Big(\underbrace{528.5\text{sf}}_{\text{TRIB AREA}} - \underbrace{\left(\frac{30" \times 18"}{12 \times 12}\right)}_{\text{- COLUMN AREA}}\Big) = 203.6^k$

$\phi V_c \geq V_u \longrightarrow$ OKAY.

NO SPECIAL OR ADDITIONAL REINFORCING REQUIRED
ORIGINAL DESIGN HAS SUFFICIENT STRENGTH

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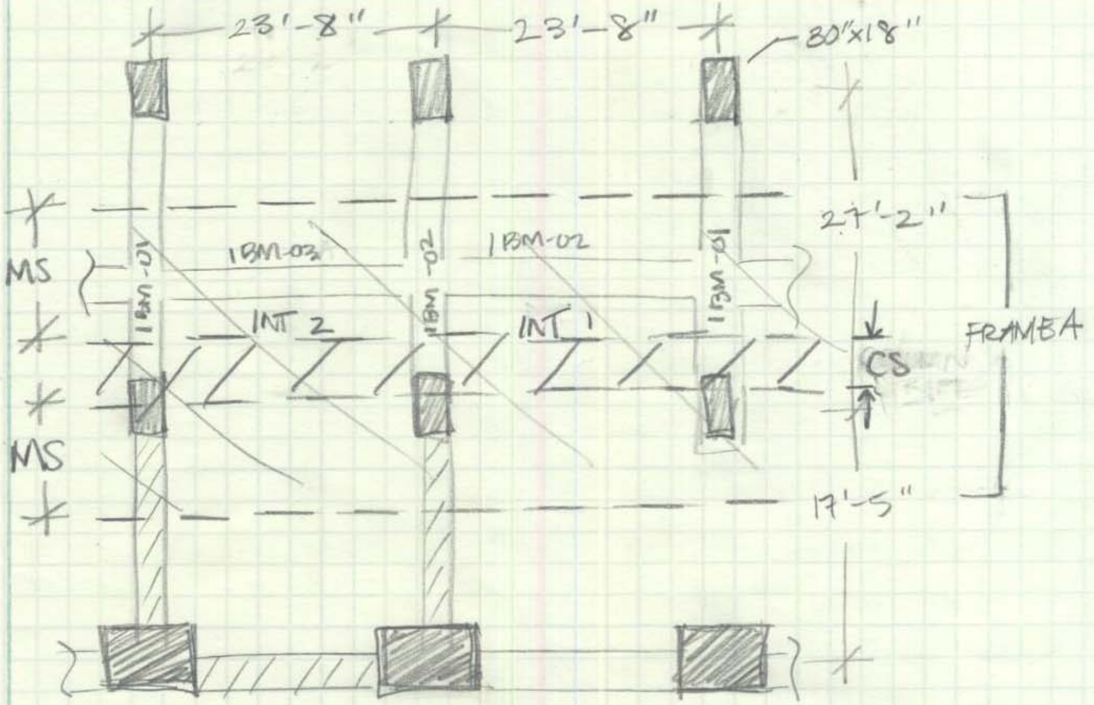
Figure F.3 Punching Shear Check Calculation

SLAB CHECK



12" CONCRETE SLAB
1ST FLOOR GRIDS E-F-G, 1-3-5

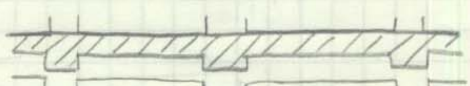
1BM-01 - 26"x40"
(7)#10, (4)#10

f'_c = 6 ksi   #5 @ 12" o.c. BOTTOM
#6 @ 12" o.c. MIDSTRIPE TOP

1BM-02 - 20"x36"
(6)#9, (6)#9

FOR SIMPLICITY ASSUMING BEAMS 1BM-03 & 1BM-02 ARE ALIGNED WITH MIDDLE COLUMNS.

1BM-03 - 20"x34"
(6)#9, (6)#9



$w_L = 100$ psf
$w_D = 190$ psf — INCLUDES SUPERIMPOSED DEAD LOADS

$w_u = 1.2w_D + 1.6w_L = 1.2(190\text{psf}) + 1.6(100\text{psf}) = 388\text{psf}$

$M_o = \frac{1}{8}(0.388\ k/ft^2)(22.3')(23.7' - \frac{18"}{12"})^2 = 533$ kft

DISTRIBUTION OF M_o

$-M_{int} = 0.65(533\text{kft}) = 347$ kft
$+M_{int} = 0.35(533\text{ kft}) = 187$ kft

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Figure F.4.1 Slab Check Calculation

+187 +187

M_{0A} | −347 INT 2 −347 | −347 INT 1 −347 | kft

RATIO OF FLEXURAL STIFFNESS

$\alpha = \frac{I_{BEAM}}{I_{SLAB}}$ FLAT PLATE $I_S = I_B \rightarrow \alpha = 1.0$.

MINIMUM THICKNESS OF SLAB WITHOUT INTERIOR BEAMS
↳ w/o DROP PANELS

$\frac{l_n}{33} = \frac{(23.6' \times 12")}{33} = 8.575" < t_{DESIGN} = 12" \rightarrow$ OKAY

INT 1 : 2

$\frac{l_2}{l_1} = \frac{22.3'}{20.7'} = 1.077$ $\alpha \frac{l_2}{l_1} = (1.0)(1.077) = 1.077 > 1.0$

ACI 318 § 13.6.4.1

1.0	1.077	2.0
75	x	45

$x = 72.69 \approx 73\%$
↳ 73% OF M^- TO CS
27% OF M^- TO MS

CS = COLUMN STRIP = 11.2'
MS = MIDDLE STRIPE = 11.2'

MOMENT DISTRIBUTION.

−347 kft
/ 73% TO CS = −253 kf
  / 85% TO BEAM = −215 kft
  \ 15% TO CS SLAB = −38 kft
\ 27% TO MS = −94 kft

+187 kft
/ 73% TO CS = 137 kft
  / 85% TO BEAM = 116 kft
  \ 15% TO CS SLAB = 21 kft
\ 27% TO MS = 50 kft

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Figure F.4.2 Slab Check Calculation

SUMMARY

FRAME A TOTAL WIDTH = 22.3', CS = 11.15', MS = 11.15'

TOTAL M	-347	+187	-347	-347	+187	-347	kft
M BEAM	-215	+116	-215	-215	+116	-215	
M_{CS} SLAB	-38	+21	-38	-38	+21	-38	
M_{MS} SLAB	-94	+50	-94	-94	+50	-94	

REINFORCEMENT DESIGN

ITEM	DESCRIPTION	INTERIOR SPAN M^+	INTERIOR SPAN M^-	
1	M_u (kft)	+21	-38	
2	CS WIDTH (b)	114"	114"	
3	EFFECTIVE DEPTH (d)	10.3125"	10.3125"	
4	$M_u(12)/b$	+2.21	-4	
5	$M_n = \frac{M_u}{\phi}$, $\phi = 0.9$	+23.3	-42.2	(kft)
6	$R = M_n/bd^2$ (lb in)	23	38	
7	$\rho_{req'd}$	0.0014	0.0014	
8	$As_{req'd} = \rho bd$	1.65 in²	1.65 in²	
9	f_y = 60ksi, $As_{min} = 0.0018bt$	2.46 in²	2.46 in²	← GOVERNS
10	# OF BARS $\frac{As}{0.31in^2}$ ASSUMING #5's	7.9 → 8	7.9 → 8	
11	MINIMUM # = WIDTH/2t	5	5	

CONCLUSION:

ANALYZED: (8) #5 BARS @ 14" O.C

DESIGNED: (9) #5 BARS @ 12" OC

↳ DESIGN EXCEEDS ANALYZED

STRENGTH SUFFICIENT

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Figure F.4.3 Slab Check Calculation