360 State Street

New Haven, Connecticut

FINAL THESIS REPORT

STRUCTURAL DEPTH & BREADTH STUDIES

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Executive Summary

The following document has been the work of a year-long thesis project that integrates five years of studying Architectural Engineering at the Pennsylvania State University. The final report includes a structural depth of the existing framing system of 360 State Street—a thirty-two story residential tower located in the heart of New Haven, Connecticut. Additionally, two breadth topics have been completed in conjunction with the main study. The overall intent of the presented information is to illustrate a sufficient level of understanding behind the engineering design decisions that go into large projects.

The original structure of 360 State Street consisted of a cast-in-place structural system throughout the entire residential tower. Due to local trades being unable to provide a competitive cost and schedule, the design was later changed to staggered steel trusses. It was assumed that the engineers were not able to fully investigate alternative design solutions in order to meet the deadline for complete construction documents. The intent of this report therefore; is to verify the existing design by comparing it to an alternative.

The structural depth study investigates a more traditional steel frame that potentially could have provided a more competitive cost and schedule. Two rows of columns were added to the existing gridlines as well as beams to frame the structure together. Diagonal bracing and moment connections were also added to provide stability. An analysis was then conducted to provide preliminary member sizes for the gravity and lateral systems.

The new framing system boasts a lower overall building weight while increasing the strength and rigidity of 360 State Street. Although, shortening spans did not decrease floor depths or provide an extra level, the original architectural floor plans were maintained. The lateral systems passed inspection however; more time could have been spent with alternative beam and column placement. Overall, half the design goals were met for the alternative design.

The breadth topics of the report include a building envelop study which proposes an all-glass façade and a cost & schedule comparison of the mentioned systems. The alternative façade incorporates spandrel glass panels to increase thermal performance while maintaining an interesting aesthetic. Compared to the existing system, the alternative can save up to $120,000 a year in electricity costs.

The cost comparison of the framing systems conclude that both designs are priced roughly the same—$9.5 million. Additionally, the estimated construction time of both systems is within 110 days provided a 40 hour work-week. Furthermore, the analysis of the façade design provided base numbers for basic materials however; more conclusive numbers would have to be gathered from manufacturers for a better assessment.

Overall, the intent of this report was to justify the use of staggered steel trusses in 360 State Street. Both framing systems provide a competitive design however; the trusses provide more flexibility in terms of architectural layout and ease in construction. The cost assessment additionally verified that the staggered trusses provide more performance and durability for the same cost of a more complicated system. Therefore, it can be concluded that the initial redesign of 360 State Street utilized the best system possible.

Introduction

As a year-long Capstone Project, the following thesis report is a culmination of five years studying Architectural Engineering at the Pennsylvania State University. The existing design of 360 State Street will be showcased as the subject of interest. The objective is to propose an alternative design solution based on a previous analysis of the building. The report is broken down into three major sections: a structural depth and two related breadth topics. Overall, the intent of the presented information is to illustrate a sufficient level of understanding behind engineering design decisions.

360 State Street is a thirty-two story building located in the heart of New Haven, Connecticut. It was chosen on principle for its unique integration of architectural design with engineering ingenuity. The following document will focus on the upper twenty-six stories of the building; any changes to the residential tower will have a significant impact on the entirety of the project. The depth study will investigate the existing staggered steel truss frame and will propose an alternative, preliminary steel frame design. The breadth topics will include a building envelop study of the existing pre-cast concrete and aluminum panel façade with a comparison to an all-glass façade. In addition, a cost and schedule comparison will conclude the feasibility of the proposed changes. All design considerations will include the following goals:

- ♦ Longevity and durability of the structure.
- ♦ Viability of the alternative solutions.
- ♦ Conscious and sustainable decisions.
- ♦ Preservation of the current architectural layout.

In general, the report will illustrate an understanding of the existing design by evaluating all decisions and their effect on the overall project. Each individual study will be introduced and given an overview of the existing system. It will be followed by a description of the design process and the proposed solution. Each section will conclude with a comparison of systems and recommendations.

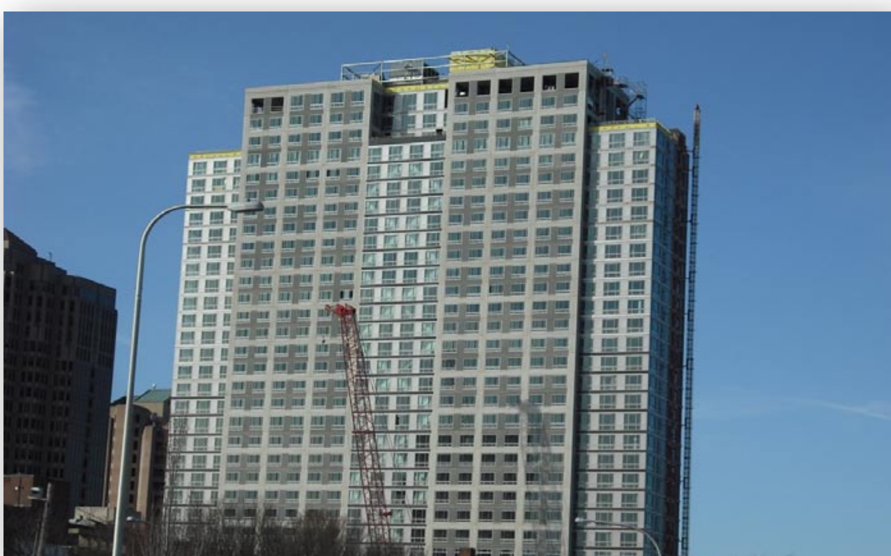


Figure 1: Most recent image of 360 State Street under construction- April 2010.

Overview of 360 State Street

360 State Street is an innovative building project developed by Becker + Becker Associates. Located in downtown New Haven, Connecticut, the building is situated on the corner of Chapel and State Street just two blocks east of the historic town green. As the newest addition to the city's skyline, the project consists of thirty-two stories of retail, parking, and residential living space.

Figure 2: Site Map with building footprint highlighted

Architecture

Designed by the owner, 360 State Street features a precast concrete and aluminum panel façade with glazing and ornamentation on the lower levels. The large windows capture the views of New Haven harbor, Yale campus, and the surrounding hills. The base of the building includes a large retail area with four floors of an open-air parking garage. The slender tower that begins on the sixth floor contains 500 apartment units varying from one to three bedrooms. The overall design showcases a landscaped garden terrace and an outdoor pool and patio. Sustainable features include recycled building materials and geothermal walls. The designer's goal is to achieve LEED® Silver certification and encourage an urban lifestyle in the city of New Haven.

Structural Systems

The building is a mixture of reinforced concrete and steel framing. The floor to floor heights vary between the levels however; the foundation begins roughly 17' below grade. The soil composition of the site is a mixture of construction debris, coarse gravels, and pockets of sand. Additionally, the water table is moderately high. These conditions have resulted in a shallow foundation that consists of a mat slab ranging in thickness from 36" to 68." The slab bears onto soil and pressure injected footings supported by a series of mini-piles. The foundation is also underpinned to the adjacent Pitkin Tunnel which will be refinished as the access ramp into 360 State Street.

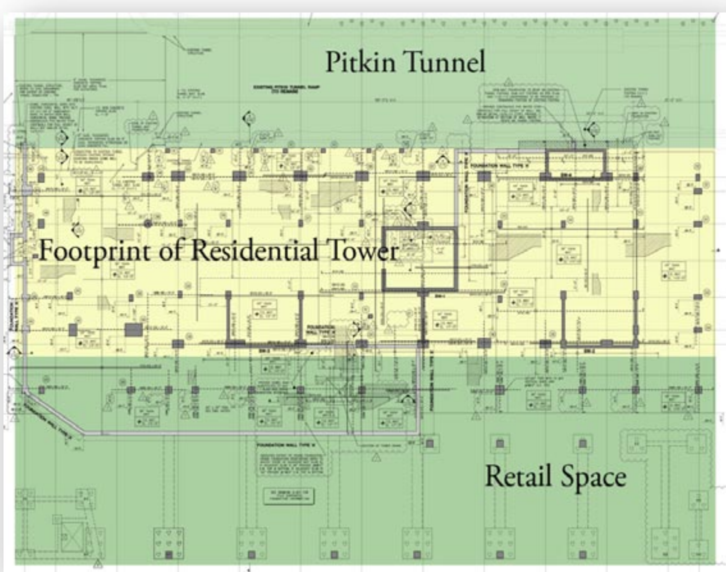


Figure 3: Foundation Plan with building functions highlighted

The base of the building is composed of high-strength cast-in-place concrete. Beams and columns support flat plate and post-tensioned floor slabs in the retail and parking garage. The upper portion of the building consists of a more uniform framing system. A series of staggered steel trusses span across the width of the building and are framed together by spandrel beams. The floor system is composed of 8" hollow core planks that have a unique architectural finish. The staggered trusses also double as the lateral support in the East-West direction of the building while X-braces support the North-South direction.

Mechanical

The main mechanical system of the building consists of a 400kW fuel cell located on the first floor. It supplies the hot and cold water for the building. Each apartment unit is also supplied with its own heat pump and air conditioning unit. Several mechanical rooms are scattered throughout the building to house fans and control panels. The rooftop mechanical room additionally houses 360's two cooling towers.

Lighting/Electrical

The utilities for the building are run through the cellar of 360 State Street up to the first floor which contains two main electrical rooms. The building is powered by five switchboards that range from 1,600 to 3,000 amps. Four are connected to the city's utility service grid at 120/280V and the fifth is connected at 277/480V. A 600kW emergency diesel generator is also located on the first floor.

360 State Street's lighting needs vary throughout the building. The overall ambient light is created by a combination of fluorescents, halogens, and LED's. The fixtures are mounted in many different positions depending on the desired aesthetic and function. For example, in the retail space and rental office, a 2x2 center basket troffer is provided. It is semi-recessed and provides enough light to efficiently work or shop. Examples of other fixtures include compact fluorescent wall-washers in the parking garage and halogen cylinder pendants in the lobby.

Figure 4: Rendering of 360 State Street

Construction

Ground breaking began in September of 2008 and will continue through October of 2010 with tenants slowly moving in over the summer. The projected footprint of 360 State Street is roughly half the area of the block. With little room for flexibility, field offices and building materials were stored offsite. The schedule predicted one level of concrete would be completed every week while one level of steel will be erected every two to three days. The assembly of the façade would follow a few weeks after each floor is put in place. Once the residential tower is complete, the second phase of the project will begin with the placement double-tee beams for the open-air parking garage and retail space.

Structural Depth Study

360 State Street was originally designed as a cast-in-place (flat plate) concrete system by DeSimone Consulting Engineers out of New York City. After an initial estimate, the design was found to be impractical. Local trades and concrete construction firms in New Haven were not able to provide a competitive cost and schedule. In comparison to New York whose firms can place one floor per two or three days, Connecticut firms found it very challenging to complete even one floor per week. The structural engineers thus redesigned the framing with a series of staggered steel trusses that local firms were able to bid more competitively. In order to keep on schedule, the redesign had a total of eight weeks to reach full construction document status. It is assumed that the structural engineers were not able to fully investigate other structural systems within that period of time. It is therefore the intent of this study to explore an alternative solution to the staggered steel trusses.

Previously concluded from the initial building analysis of 360 State Street, the staggered steel trusses were found to have a significant impact on the overall building design. The trusses not only support the gravity loads, they also support the lateral loads in the East – West direction of the building. In order to conclude an investigation of the existing design, the depth study will explore an alternative gravity system for the upper twenty-six stories of the building as well as an accompanying lateral system. To ensure feasibility, a more traditional steel frame design will be considered. The design goals will include:

- Increasing strength and rigidity with additional structural elements.
- Minimizing floor depth by shortening span lengths.
- Decreasing overall building weight.
- Optimizing the lateral system and foundation.

The new design will attempt to capitalize on the shortfalls of the original design. Heavy gravity loads will be reduced and more evenly distributed throughout the structure; floor-to-floor heights will become more consistent; and story drift due to lateral forces will be minimized. Most importantly, the design will consider the quality of materials to ensure a durable structure that can withstand use and time. Ultimately, the objective is to recommend an alternative, preliminary design that matches the owner's design goals.

To demonstrate an understanding of the engineering design decisions, the depth study will first explore the existing gravity and lateral systems by evaluating the framing elements. Load paths will be explored to determine the impact on the foundations and advantages will be compared to disadvantages. A discussion of the design process will follow regarding the alternative design. Then the new gravity and lateral systems will be introduced and examined. The study will conclude with a recommendation regarding the best solution for 360 State Street.

Existing Gravity System

The residential tower of 360 State Street is roughly one-third the area of the building's footprint. It begins on the sixth level and rises twenty-fix stories with floor-to-floor heights varying between 9' – 4" and 10' – 4". Nine evenly spaced steel frames span across the East – West direction of the building and contain an alternating number of story-high trusses. Each frame is tied together by spandrel beams and includes additional W-shapes around the stairwell and elevator openings.

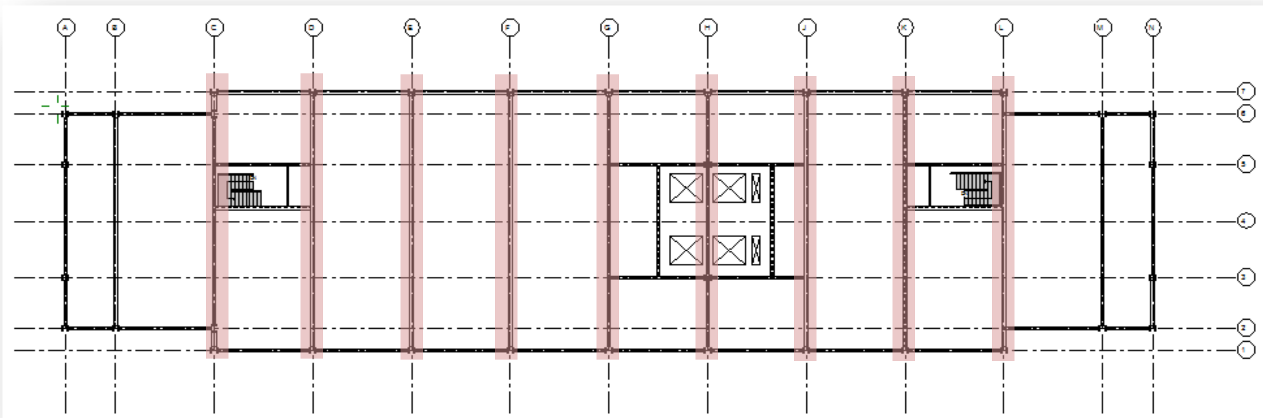


Figure 5: Typical Framing Plan for Floors 7 – 29

A typical truss is composed of W10x top and bottom chords. Spanning 62' – 0", the chords are pinned to exterior columns that are also W14's. Each column has a length of roughly two-stories; their splices occur mid-height on the above and below stores from a truss. The interior members of the trusses consist mostly of hollow structural steel members that are pinned as diagonals or are placed vertically. A Vierendeel panel is also located at the center of the truss; it frames the opening for the interior corridor. Each truss is spaced the width of a typical apartment unit so that it can be easily framed within the interior walls. The location of the trusses alternate with the column lines such that the long axis is always situated intermediately between levels as seen in Figure 7.

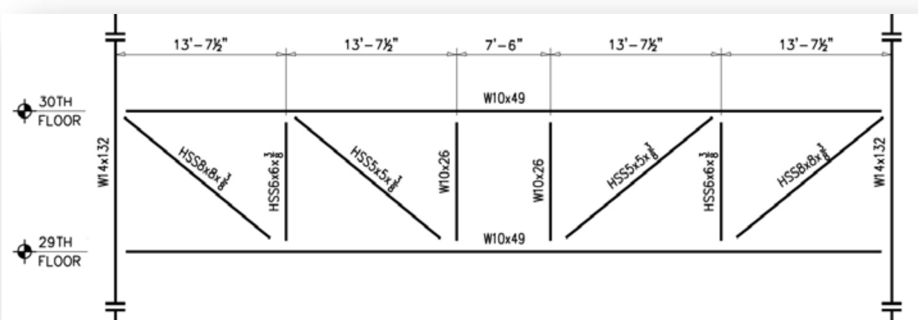


Figure 6: Detail of Typical Truss

The overall staggered steel truss system also includes a floor system of 8" hollow core planks topped with concrete and a unique architectural finish. Tensioned steel strands are located within the precast concrete to help carry loads. The planks come in sections of 4' x 24' and they span between the trusses bearing onto the top or bottom of an adjacent chord. Rebar is additionally placed within the tubular voids and are grouted to connect the sections. Altogether, the floor system behaves as a rigid diaphragm to distribute loads across the frames.

Existing Lateral System

The residential tower has two distinct frames that compose the upper level lateral system. In the short dimension of the building, staggered steel trusses combine to create a unique frame. The trusses are composed of W-shapes and hollow structural sections. Each spans 62' – 0" between the exterior columns and alternate between levels. Altogether there are nine frames that begin on the sixth floor and terminate at the roof level.

The second frame present in the tower is a system of X-braces on the North and South exterior of the building. The braces span between three columns and are composed of 14 x 14 and 10 x 10 hollow structural sections. Each diagonal member is roughly two stories in height and intersects the intermediate spandrel beam. The X-braces are included in a moment frame composed of the spandrel beams and columns on the level above the twenty-eighth floor.

Additionally, the base of 360 State Street is laterally supported by four unique shear walls. Each is composed of 8,000 psi strength concrete and is heavily reinforced with bar sizes ranging from #5's to #11's in both vertical and horizontal directions. One shear wall encloses the elevator core and another encloses a stairwell. The remaining two have three sides in order to leave room for parking. In general, the shear walls begin below grade and top off at the sixth floor. They will be ignored in this report as well as the concrete base of the building.

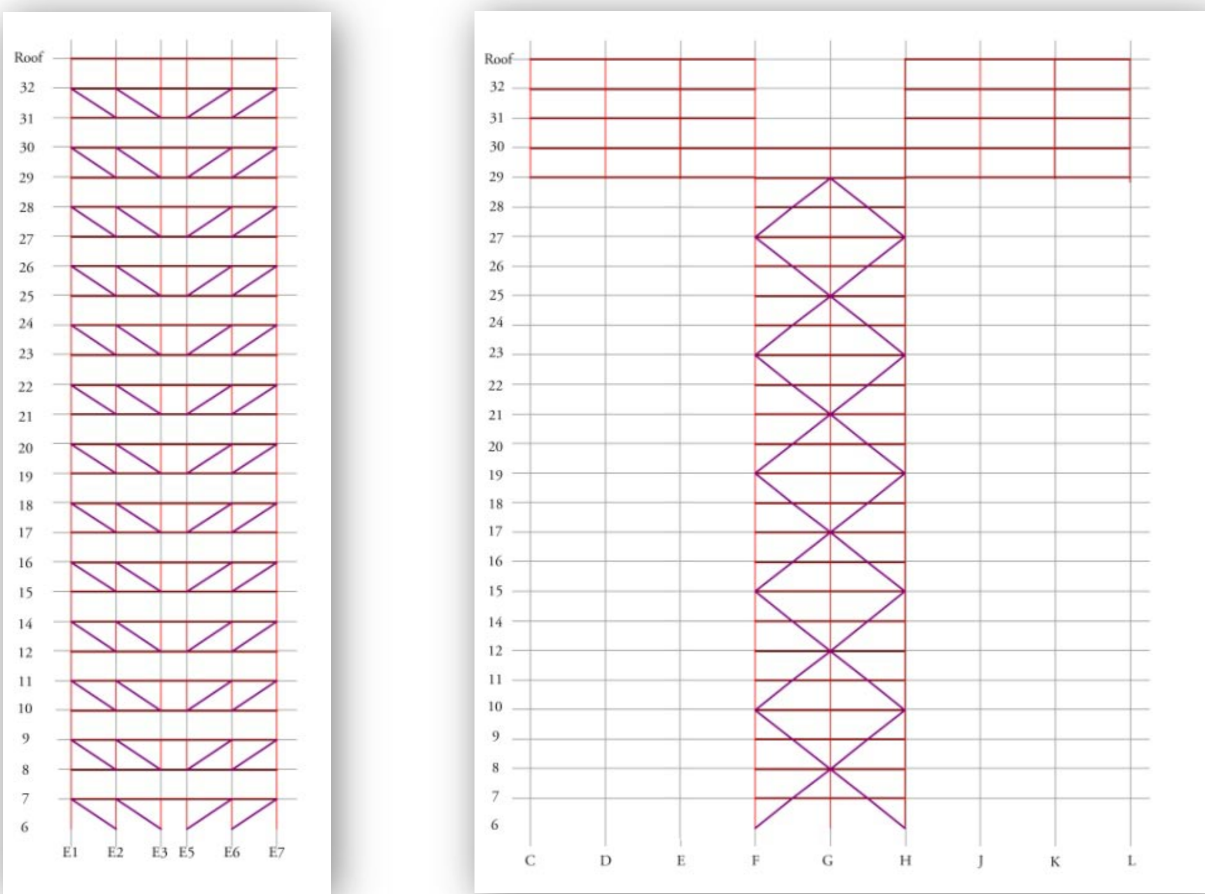


Figure 7: Section of Staggered Truss Frame (left); Section of X – brace frame (right)

Load Paths

The distribution of loads across a framing system determines the efficiency of the structure. A successful design minimizes the occurrence of loads concentrating in one area by creating load paths. In general, load follows stiffness meaning loads will be carried by rigid structural members down to the foundation. The staggered steel truss system takes advantage of this principle by utilizing a rigid diaphragm with the hollow core planks. The floor system, spanning between two trusses, will distribute its loads to the adjacent truss chords. Depending if the structural member is a top or bottom chord, the loads will distribute a little differently. A top chord member will distribute its load throughout the truss members. Some force will reach the columns and continue to the foundations while some force will carry onto the next level of planks only to be distributed again through adjacent chord member. The load will dissipate across the entire frame before reaching the foundation, eliminating high stress concentrations. A bottom chord will act in the same fashion.

With lateral loads, the distribution of forces is slightly different for staggered steel trusses. The horizontal forces put pressure on the surface of the building; these forces are then transferred into the rigid diaphragm or diagonal brace. Shear will move through the planks and dissipate into the adjacent chord members. Some force will reach the columns and continue to the foundations while some force will be carried into the next level of planks. This process will continue until all forces are dissipated throughout the entire structure and reach the foundations.

X – braces similarly distribute loads across the entire structure. The orientation of each diagonal determines the path of the force. One diagonal is typically in compression while the other is in tension. The roles reverse depending on the direction of the wind or the cyclic motion of seismic forces. When a horizontal force arrives at a diagonal, the force is carried across the brace. These structural members are typically very stiff in order to direct the path of the load. The diagonal forces will pass into the adjacent column and continue towards the foundation.

Impact on the Foundation

The foundations of 360 State Street consist of a concrete mat slab that varies in thickness from 36 to 68 inches. The thickness depends upon the required capacity of that particular area. As previously described, all the forces eventually transmit into the soil via the load paths established by the structural design. In 360 State Street, the loads funnel into the exterior columns of the trusses and into the concrete columns located in the base of the building. The locations of these columns are highlighted in Figure 8. Each column is typically 36” x 52” reinforced with (16) # 10 bars in 8,000 psi concrete.

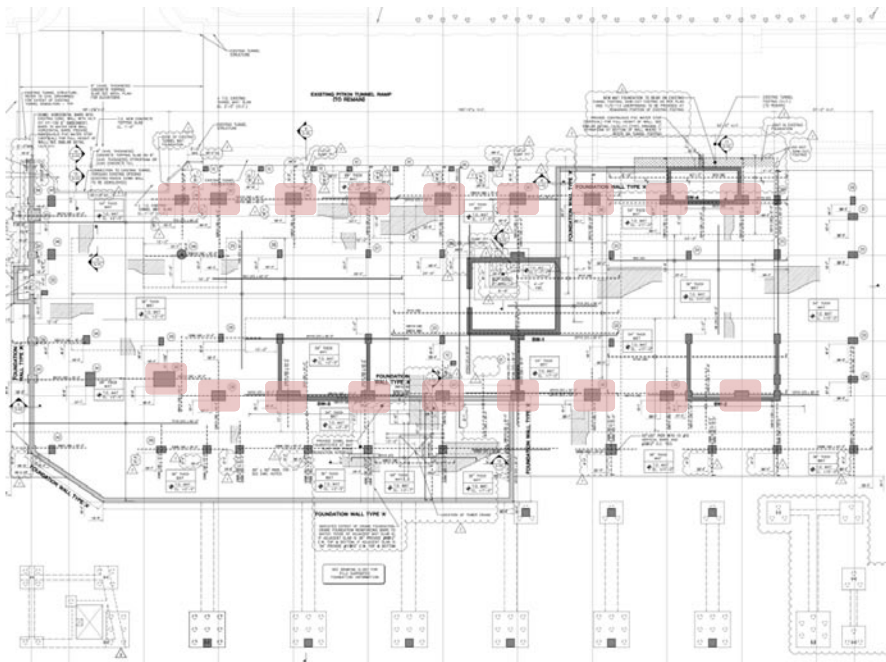


Figure 8: Foundation Plan with Columns highlighted

Although the loads are found to distribute across the residential tower, the loads concentrate in the middle third of the foundation. A quick design check was conducted to verify the bearing capacity of the foundation. According to structural drawing S001, the minimum bearing capacity is slated as 4.5 tons per square foot. The overall building weight, calculated in Appendix X, was divided by the area of the tower to find that the gravity loads are an estimated 2.81 tsf. The foundation’s capacity is more than adequate to sustain such loads however; the factor of safety was found to be only 1.6. Typical bearing capacities have a factor of safety of 4.0. Further investigation would be required to provide a more accurate design check; this is will not be completed in this report. Moreover, the foundation is deemed to be sufficient. High concentrations of load in the mat slab do not appear to be a problem within 360 State Street.

Quick Check of Bearing Capacity:

$$Qa \geq \frac{Total\ Load}{Total\ Area}\ where\ Qa = 4.5\ tsf \qquad Qa = 4.5\ tsf \geq qa = 2.81\ tsf \therefore okay$$

$$qa = \frac{\{90{,}757k\left(\frac{1lb}{2000\frac{lb}{ton}}\right)\}}{(62\ ft \times 260\ ft)} = 2.81\ tsf \qquad F.S. = \frac{4.5\ tsf}{2.81\ tsf} = 1.6$$

Design Confirmation

The design of 360 State Street was found to be sufficient in three previous technical reports. The building was analyzed for gravity and lateral loading. In this section, critical structural members will be evaluated to ensure the sufficient strength was obtained. Information regarding gravity, wind, and seismic loads can be found in Appendix D.

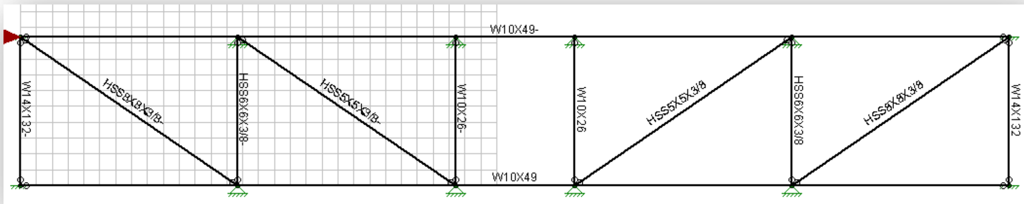


Figure 9: Typical Truss

Beginning with a typical truss, the element was modeled in RISA 3D to determine its behavior under the most critical loading case. The top and bottom chords were loaded with the 160 psf taken over a tributary area of 23' – 8" (roughly 3.8 k/ft). This value is taken from the load combination of 1.2D + 1.6L as well as the live and dead schedules found in Appendix C.

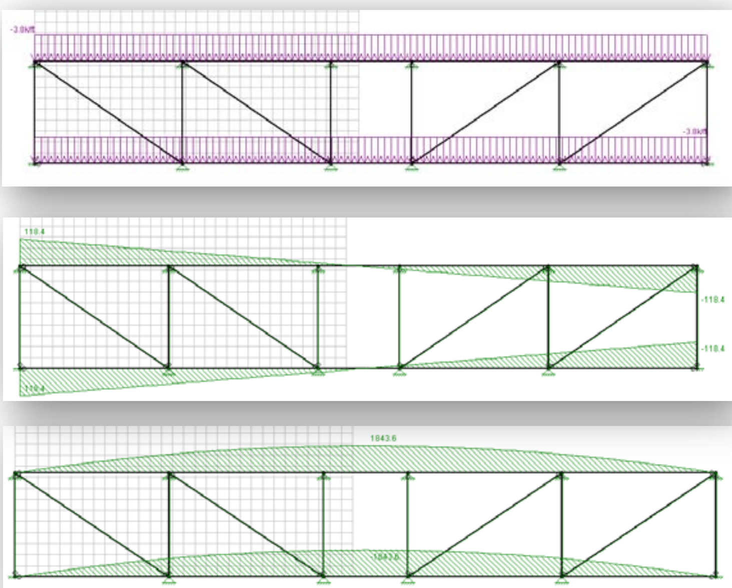


Figure 10: Truss in loading (top); Shear Diagram (middle); Moment Diagram (bottom)

It was originally assumed that the interior members of the truss would carry a significant portion of the design loads from the top to the bottom chord. In Figures X & X, the shear and moment diagrams illustrate that the truss members behave as a singular unit. The reactions were found to be negligible in the interior members. Each truss can be considered as a beam with unique sectional properties. Further investigation is necessary to determine more accurately the meaning of the resulting reactions.

The framing system also includes an 8" hollow core plank floor system that is topped with 2 inches of concrete and an architectural finish. According to the load tables provided by StresCore Inc., an 8 inch plank can carry up to 181 pounds per square foot. For a design load of 160 psf from the load combination 1.2D + 1.6L, the plank is more than sufficient to withstand 360 State Street's gravity loads. With deflection criteria already incorporated into the manufacturer's span tables, the floor system is reasonably within deflection limitations.

	SPAN IN FEET												
STRAND	18'	20'	22'	24'	26'	28'	30'	32'	34'	36'	38'	40'	42'
8SC46	116	85	62	45									
8SC47	170	129	99	76	58	43							
8SC48		182	143	112	89	70	55	43					
8SC58			185	148	120	97	78	63	51	40			
8SC68				183	149	122	100	83	68	56	45		
8SC78					179	148	123	102	85	71	59	49	40
8ST46	142	102	73										
8ST47	210	158	119	89	66								
8ST48		225	174	136	106	82	63						
8ST58			228	181	144	115	92	73					
8ST68				224	181	147	119	97	76				
8ST78				264	215	177	145	120	96	76			
	SUPERIMPOSED LOAD CAPACITY IN POUNDS PER SQUARE FOOT												

Figure 11: Hollow Core Plank Load Table

The following table evaluates a critical column line located in along gridline E. The member capacities appear significantly higher than the resulting reactions however; this could be caused by any unknown design decisions. Additionally, the member sizes could be higher in order to provide adequate stiffness for the lateral system.

Critical Column E – 1					
Floor	**ϕPn (kips)**		**Pu (kips)**	**Size**	
Roof	1360	≥	118	W14X109	√
31	1360	≥	236	W14X109	√
30	1360	≥	354	W14X109	√
29	1640	≥	472	W14X132	√
28	1640	≥	590	W14X132	√
27	1990	≥	708	W14X159	√
26	1990	≥	826	W14X159	√
25	2210	≥	944	W14X176	√
24	2210	≥	1062	W14X176	√
23	2650	≥	1180	W14X211	√
22	2650	≥	1298	W14X211	√
21	3230	≥	1416	W14X257	√
20	3230	≥	1534	W14X257	√
19	3330	≥	1652	W14X283	√
18	3330	≥	1770	W14X283	√
17	4050	≥	1888	W14X342	√
16	4050	≥	2006	W14X342	√
15	4710	≥	2124	W14X398	√
14	4710	≥	2242	W14X398	√
12	5040	≥	2360	W14X426	√
11	5040	≥	2478	W14X426	√
10	5420	≥	2596	W14X455	√
9	5420	≥	2714	W14X455	√
8	5950	≥	2832	W14X500	√
7	5950	≥	2950	W14X500	√

Figure 12: Critical Column Evaluation

The following table examines the capability of the lateral systems against wind and seismic forces. In RAM Structural Systems, one frame containing staggered steel trusses was modeled as well as one frame with the X – braces. A 1,000 kip force was applied horizontally to the top of both frames to determine the displacement of the frame as well as its relative stiffness. Furthermore, the model was used to calculate the story drift caused by seismic and wind forces.

Floor	Displace-ment Y	Displace-ment X	Stiffness Y	Stiffness X	Seismic - Story Drift Y (in)	Seismic - Story Drift X (in)	Wind - Story Drift Y (in)	Wind - Story Drift X (in)
Roof	7.578	22.162	132	45	0.06	0.79	0.01	0.15
31	6.831	20.084	146	50	0.07	0.90	0.01	0.13
30	6.024	17.812	166	56	0.06	0.76	0.01	0.11
29	5.411	16.079	185	62	0.04	0.57	0.00	0.10
28	4.888	14.525	205	69	0.04	0.49	0.00	0.09
27	4.548	13.499	220	74	0.03	0.43	0.00	0.08
26	4.260	12.611	235	79	0.03	0.38	0.00	0.08
25	3.961	11.689	252	86	0.03	0.33	0.00	0.07
24	3.660	10.763	273	93	0.02	0.29	0.00	0.06
23	3.369	9.875	297	101	0.02	0.25	0.00	0.06
22	3.082	9.000	324	111	0.02	0.22	0.00	0.05
21	2.791	8.119	358	123	0.01	0.18	0.00	0.05
20	2.509	7.267	399	138	0.01	0.15	0.00	0.04
19	2.245	6.474	445	154	0.01	0.13	0.00	0.04
18	1.986	5.701	503	175	0.01	0.11	0.00	0.03
17	1.723	4.918	580	203	0.01	0.09	0.00	0.03
16	1.478	4.194	677	238	0.01	0.07	0.00	0.02
15	1.271	3.584	787	279	0.00	0.05	0.00	0.02
14	1.078	3.021	928	331	0.00	0.04	0.00	0.02
12	0.879	2.447	1138	409	0.00	0.03	0.00	0.01
11	0.694	1.917	1441	522	0.00	0.02	0.00	0.01
10	0.538	1.477	1859	677	0.00	0.01	0.00	0.01
9	0.393	1.074	2544	931	0.00	0.01	0.00	0.00
8	0.246	0.668	4068	1498	0.00	0.01	0.00	0.00
7	0.118	0.318	8503	3143	0.00	0.00	0.00	0.00
6	0.032	0.086	31447	11614	0.00	0.00	-	-
Total:					0.48”	6.31”	0.06”	1.24”
					$\Delta_{allowable}$=7.33”	Passes	$\Delta_{allowable}$=7.61”	Passes

Figure 13: Lateral Analysis for Existing Lateral System

Advantages & Disadvantages

Staggered steel trusses and hollow core planks were developed in the 1960's. Beginning as a study for US Steel, the truss framing system was designed to achieve similar floor-to-floor heights as a flat plate concrete system (roughly 8' – 8"). This would allow owners to maximize the number of stories with a building if any height restrictions were in place. In the case of 360 State, the building tops off at thirty-two stories because the economy of scale was not in favor of the design. Had the building been taller, the floor plate would have been inefficient with less allowable area per zoning regulations compared to the gross square footage.

360 State Street takes advantage of the staggered truss design by using hollow core planks (other floor systems are possible with staggered steel trusses as well). The pairing of the two materials was found to maximize the strength of a building as well as increase its rigidity. The planks act as a rigid diaphragm distributing loads more effectively across a frame. This led to the analysis of tall buildings as a cantilevered beam in which the various structural members perform as a single unit. Moments across a frame would be lower in comparison however; this would not necessarily decrease member sizes or structural weight. Staggered trusses are inherently heavy due to the column sizes closer to the base. High concentrations of gravity loads from the columns would require a larger foundation to effectively carry the loads as seen with 360 State Street's thick mat slab.

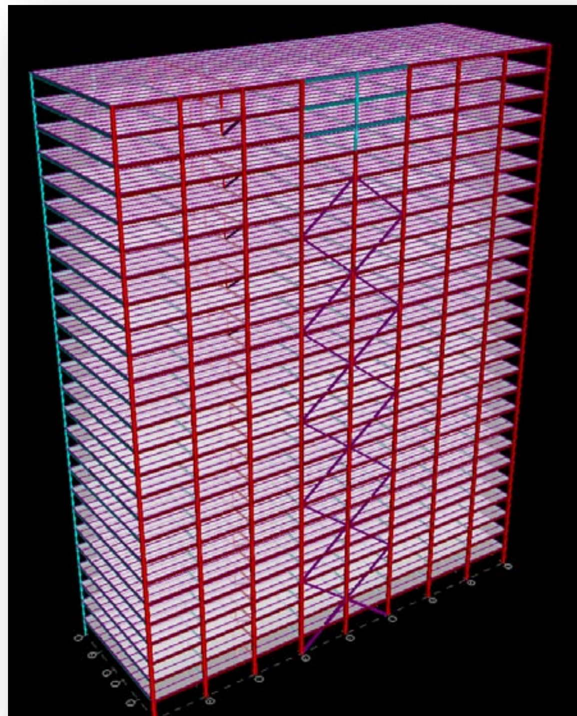


Figure 14: Existing Framing System

Hollow core planks also carry with them many advantages that benefit the overall sustainability of the building. As precast slabs with tubular voids, less material is necessary for manufacturing and less labor is required for installation. This decreases the pollution caused by equipment and limits residual waste. Additionally, this significantly decreases the construction cost for the majority of the building. Although the slabs are thicker than usual for residential projects, they are inherently fire-rated and provide an adequate amount of sound isolation. By and large, this increases the quality and durability of the structure as well as saves the owner money.

A significant advantage to staggered steel trusses is the inherent flexibility in the architectural floor plans. A single truss can span upwards of 70' – 0" eliminating the need for interior columns. Although the system never became overly popular, it was most often seen in buildings that had a doubly-loaded center corridor or repetitive floor plans such as high-rise apartment buildings like 360 State Street. In this case, the trusses span 62' – 0" with columns only running along the perimeter of the building. Less formwork was required for the foundations because fewer columns were bearing down on the structure. Fewer columns decrease the weight of the building as well as the cost of construction.

Most of the advantages of staggered steel trusses and hollow core planks are related to the constructability of 360 State Street. Although the trusses require some lead time for fabrication, they are ideal for fast-tracked projects. Steel erection is fairly quick because the trusses can be lifted directly from the truck bed. This is especially convenient if onsite storage is an issue as it was the case with 360 State Street. The hollow core planks slow down the erection process slightly because each section has to be lifted and lowered into place every two to three floors of steel. There is a benefit however, in the combination of the two systems. Once the planks are placed, other trades may begin working at or below the highest level of planks.

The combination of the two elements creates an attractive system that allows the architectural design to flourish however; they need significant attention in lateral design. Although the system can inherently carry horizontal forces in one direction, many difficulties arise to brace the other direction. In the case of 360 State Street, X – braces were placed across the columns of the trusses. The lack of interior columns limited the available locations for bracing. Within 360 State Street, the X – braces were forced to be framed across the window openings as seen in Figure 15. Aesthetically, the design can be seen as problematic. Moment connections can be considered as an alternative solution however; the time and labor placed into welding each connection is very expensive.

Figure 15: Existing X - braces

The benefits of a staggered steel truss system outweigh the disadvantages however; some improvements can be made. With a constant depth of the steel and thickness of the slab, floor-to-floor heights still vary due to connection locations. Member sizes can be optimized and forces can be more evenly distributed across the building. This could potentially decrease the overall building weight as well as the size of the foundations.

Introduction to the Alternative Structural Design

As previously mentioned, the original structure of 360 State Street consisted of a cast-in-place structural system throughout the entire residential tower. The design was later changed to staggered steel trusses on account of local trades not being able to provide a competitive cost and schedule for a concrete construction. New Haven firms have more experience with steel construction therefore; the redesign of 360 State Street will consist of a more traditional steel frame. Design guide #5 will be used as a reference from AISC and RAM Structural Systems will produce preliminary member sizes. The main objective will be to design an alternative structural system that does not negatively impact the original architectural floor plans. Again, only the upper twenty-six stories of the building will be considered and hollow core planks will be maintained as the floor system. The intent of the redesign is to change only one variable, the framing system, to determine how it affects the overall building project.

The design goals for the proposed system include:

- Increasing strength and rigidity with additional structural elements.
- Minimizing floor depth by shortening span lengths.
- Decreasing overall building weight.
- Decreasing floor to floor height.
- Optimizing lateral systems and foundations.

The following section will include a similar organization seen in the discussion of the existing gravity and lateral systems. The design process will be described to illustrate the development of the vital engineering decisions. The proposed systems will then be revealed with framing plans and sections. Load paths will additionally be explored to determine the impact on the existing foundations. Lastly, critical structural members will be analyzed to confirm the sufficiency of the design.

Design Process

In the practice of structural engineering, it is important to design according to the most significant failure mode. Columns typically experience bending or buckling failures while beams fail in terms of bending. Depending on the situation, these structural elements can experience tension or compression. The responsibility of the engineer is to design that element to withstanding failures across the weakest axis which generally depends on the geometry of the member. The alternative design intends to work with the geometry of 360 State Street to create an efficient solution.

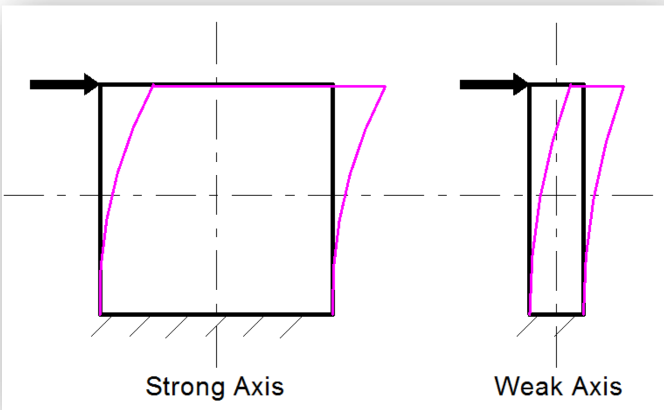


Figure 16: Geometry of 360 State Street

Consider the geometry of the residential tower in 360 State Street. The footprint of the structure is a narrow rectangle. The building can be considered as a cantilevered beam only fixed on one end. If a horizontal force is applied to the top of the building on the shallow width, the force will have to overcome 260' – 0" to displace the structure. If that same force was applied to the larger dimension of the building, it would only have to overcome 62' – 0". This side of the building is considered the weak axis of the structure; it will require more attention during the design process.

In the original design of 360 State Street, the staggered trusses span the 62' – 0" dimension of the building to support across the weak axis. Each column additionally was oriented such that the weak axis of the W – shape was in line with weak axis of the building. This design appears counterintuitive in terms of strengthening the weak axis. The orientation of the columns and trusses may have benefited the architectural layouts however; one axis appears to be over-designed. This circumstance can increase the overall building weight as well as the cost of construction. The alternative design takes the geometry of the building into consideration. For optimal strength, the columns will be oriented such that their strong axis is parallel with the weak axis of the building. Furthermore, the strong axis of the building will not need as much lateral bracing because it will be inherent in the structure.

With a general idea of how the building should behave, the architectural floor plans were studied to determine a least intrusive framing plan. The original grid lines were compared with ideal column placement. In order to ensure constructability, the number of columns was limited to the most essential. Beam placement was also considered with the location of columns so that structural members were not visible within any of the apartment units. The underside of the hollow core planks were finished as ceilings in the original design and will be maintained in the alternative design.

After establishing a rough idea of the framing plan, a structural model of 360 State Street was developed using RAM Structural Systems. The existing grids were aligned with ideal column locations and framed with structural members. After several iterations, various designs were compared in terms of building weight, deflection, and constructability. The ideal design that was chosen demonstrated a balance of those design criteria.

Proposed Gravity System

The redesigned residential tower of 360 State Street consists of a traditional steel framing system with a floor-to-floor height of 9' – 4". Four rows of columns are spaced across the short dimension of the building in varying distances. Each column is roughly two stories high with splices occurring at mid-height on every other level; the splices also alternate location every other gridline. Beams tie the structure together by framing into adjacent columns. Additional beams are located around stairwells and elevators to frame the floor openings. The column spacing in the long direction was maintained at 23' – 8" to avoid altering the apartment layouts. Larger framing plans can be seen in Appendix F.

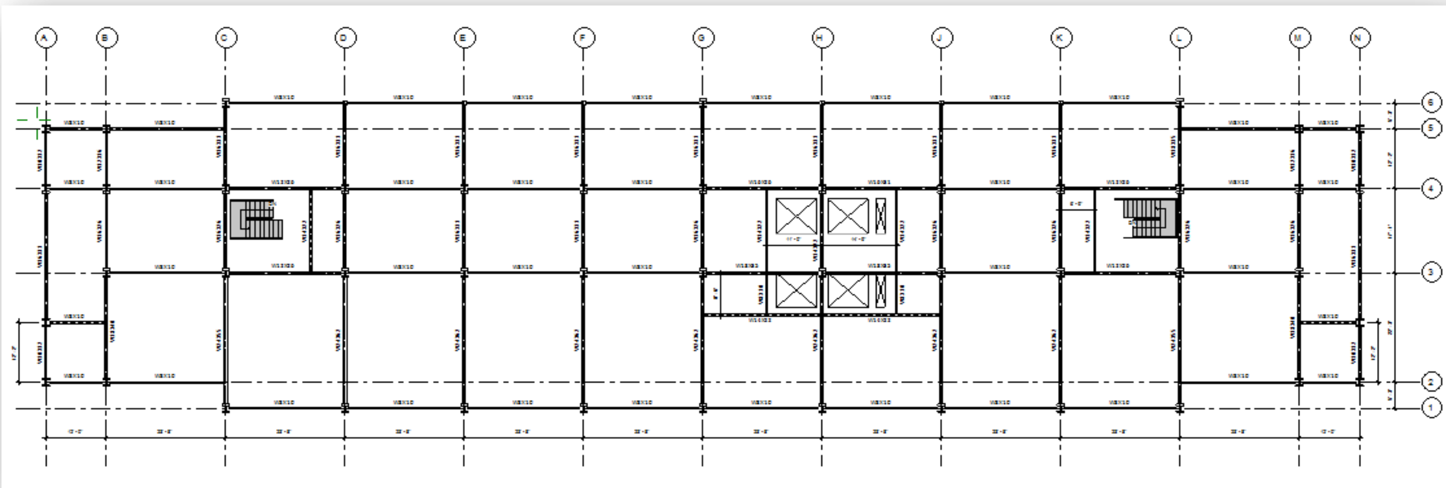


Figure 17: Typical Framing Plan Floors 7th – 29th

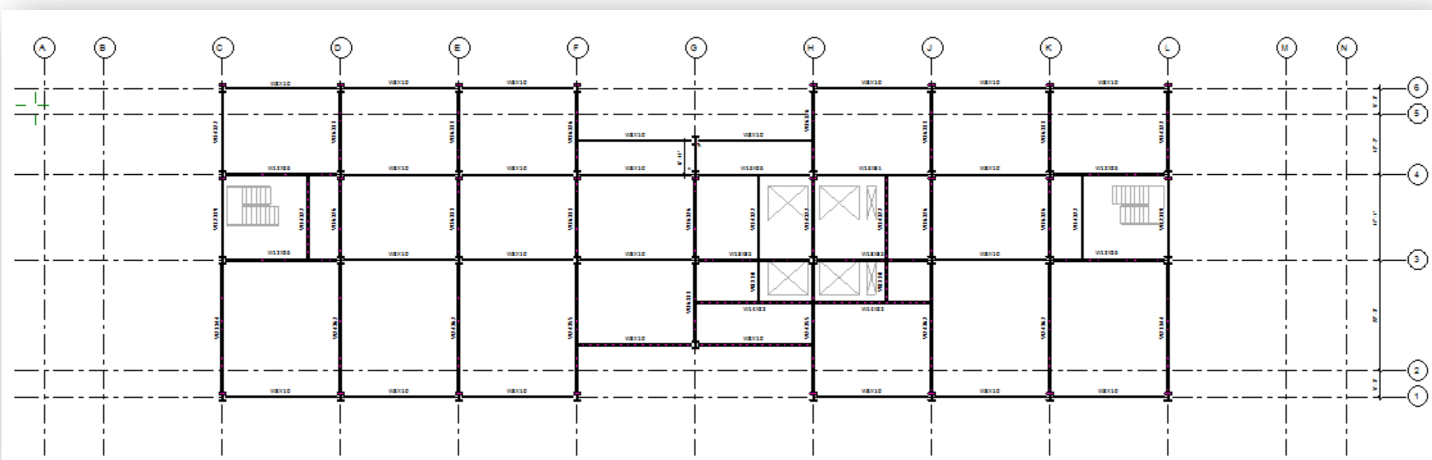


Figure 18: Typical Framing Plan Floors 30th - Roof

The framing system also includes the original 8" hollow core plank floor system that is topped with 2 inches of concrete and an architectural finish. According to the load tables provided by StresCore Inc., the optimal floor thickness for a 24' – 0" span remains an 8 inch plank. It can carry up to 181 pounds per square foot including the concrete topping. A 6 inch plank was considered however; it would deflect beyond code limitations.

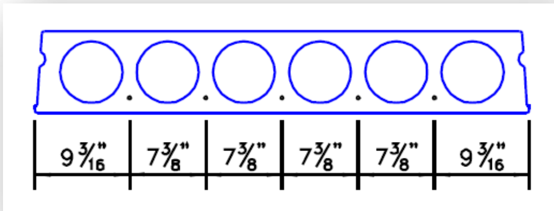


Figure 19: Typical 8" Hollow Core Plank

The planks are manufactured in 4' x 24' sections that span the long direction of the building and bear onto the top flange of the adjacent beams. Figure 19 illustrates the location of the tensioned steel strands that are located within the precast concrete. For an 8 inch plank, (5) 270 ksi low relaxation strands are required to carry the loads of 360 State Street. Additionally, rebar is grouted within the tubular voids to connect the sections of plank. Altogether, the floor system behaves as a rigid diaphragm to distribute loads across the frames. Figure 20 is a typical detail of such a connection.

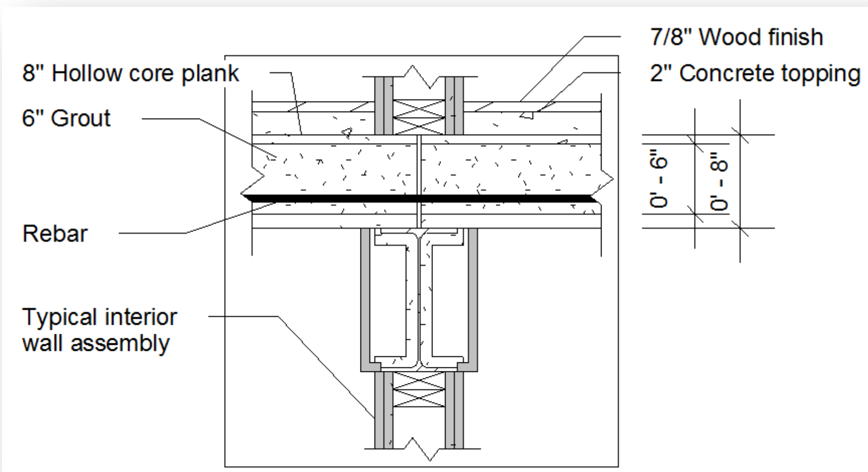


Figure 20: Typical Detail of Plank Connection

Proposed Lateral System

The alternative lateral system for 360 State Street is composed of a series of moment connections and diagonal bracing. Two distinct frames are located along the x-axis. One contains X – braces along the interior corridor; the other consists of a moment frame along gridline 4. These frames were chosen based on their minimal affect on the architectural layout. Additionally, three identical frames are located on the y-axis. Each contains (2) two-story high diagonal braces in the exterior bays.

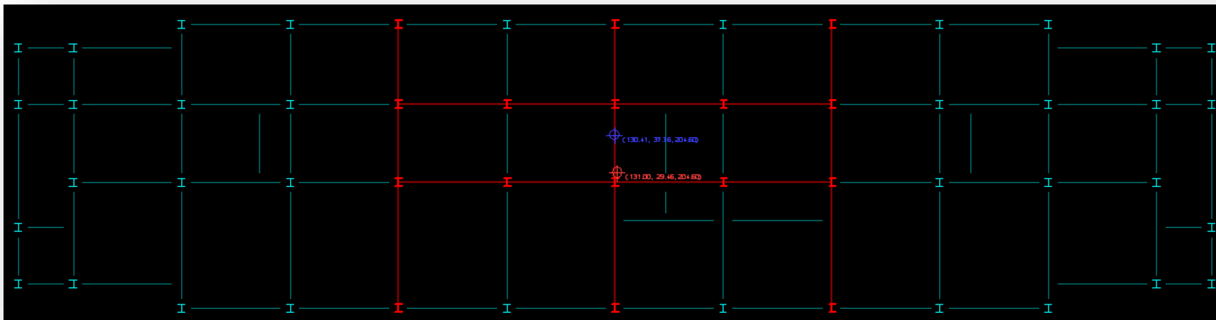


Figure 21: Typical Framing of Proposed Lateral System

In Figure X, a typical framing plan is shown with the lateral system highlighted in red. The blue dot denotes the Center of Rigidity at (131.5', 31.75') and the red dot denotes the Center of Mass at (131.95', 29.75').

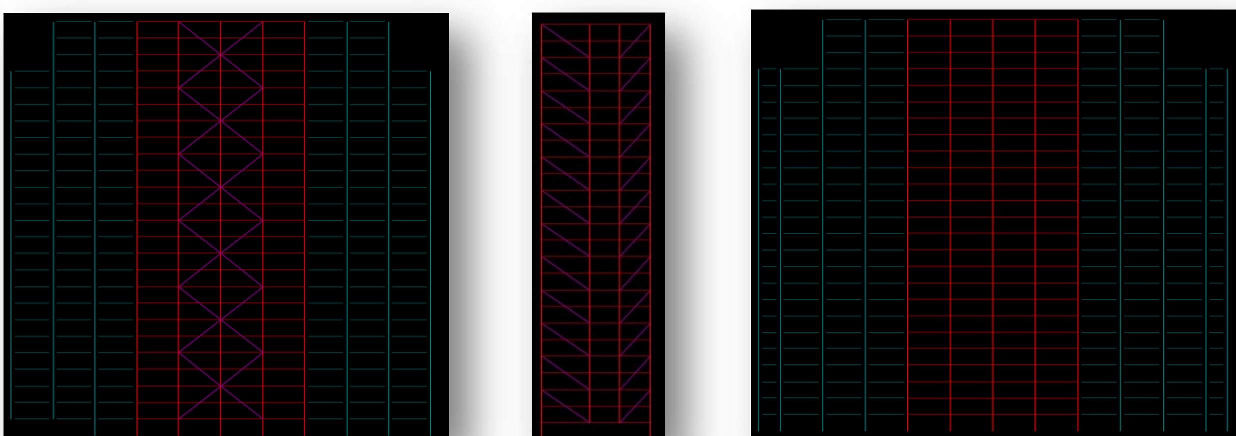


Figure 22: X – brace frame (left); Diagonal braces (middle); Moment Frame (right)

The X – braces are composed of 14 x 14 hollow structural sections from the sixth floor up to the 20th floor. The above braces are HSS 10 x 10. Additionally, the beams throughout the frame are W18x35's. The diagonal braces in the y-direction are also composed of hollow structural sections. The lower half contains HSS 10 x 10's and the upper half contains HSS 8 x 8's. Lastly, the moment frame consists of W16x26's and W16x31's. Altogether, 192 moment connections are present in the alternative lateral design; this is only 72 more than the original system. Further investigation would include designing these connections however this report is only showcasing a preliminary design.

Load Paths

The distribution of loads across a framing system determines the efficiency of the structure. A successful design minimizes the occurrence of loads concentrating in one area by directing the load accordingly. In general, load follows stiffness meaning loads will be carried by rigid structural members down to the foundation. In the alternative design of 360 State Street, every structural member takes part in carrying the load.

Beginning with the loads on the floor system, the hollow core planks direct half the load to each of the adjacent beams. The beams in turn, distribute half their load into the adjacent columns and finally down to the foundations. Similarly, a portion of the lateral forces will transmit through the hollow core planks and throughout the structure. The diagonal bracing will also distribute a portion of its load into an adjoining beam or column. The orientation of the brace as well as the direction of the force will determine the amount of load it will carry. The alternating pattern accounts for the cyclic movement of wind and seismic forces.

Impact on the Foundation

The original foundations will be considered with the alternative design in order to maintain consistency. As previously mentioned, the foundations of 360 State Street consist of a concrete mat slab that varies in thickness from 36 to 68 inches. The thickness depends upon the required capacity of that particular area. The alternative design directs the gravity loads to the array of columns across the entire footprint. These columns channel their forces through the concrete columns below before dissipating into the foundations. The locations of these columns are highlighted in Figure X. The larger columns are 36" x 52" reinforced with (16) # 10 bars and the smaller columns are 30" x 18" reinforced with (6) #9 bars. All columns are cast with 8,000 psi concrete.

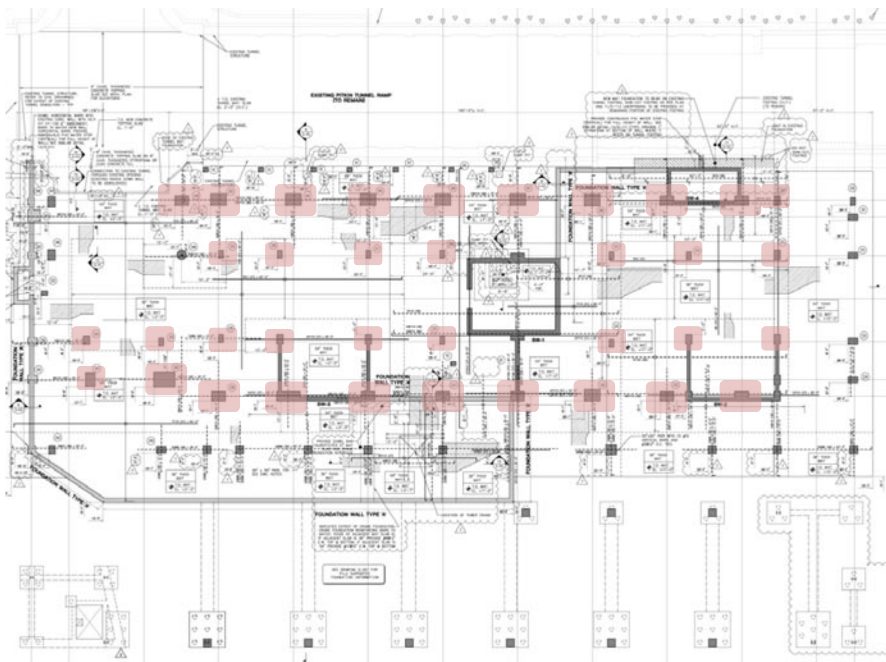


Figure 23: Foundation Plan with columns highlighted

The locations of the interior concrete columns would have to be changed in order to avoid the use of transfer beams however; their current spacing is very similar to the spacing of the steel columns. The interior space on the lower levels is mainly used for storage and a ramp for the parking garage inhabits the space between the larger and smaller columns. If the column locations were moved, neither space would be negatively impacted by the change.

Compared to the existing framing system, it can be seen that the alternative framing system distributes the loads more evenly across the area of the building. As a quick design check, the bearing capacity was calculated as 2.80 tons per square foot. The value is actually lower than the original design. Further investigation is required to check the column bearing capacity.

Quick Check of Bearing Capacity:

$$Qa \geq \frac{Total\ Load}{Total\ Area}\ where\ Qa = 4.5\ tsf \qquad Qa = 4.5\ tsf \geq qa = 2.80\ tsf\ \therefore okay$$

$$qa = \frac{\{90{,}288k\left(\frac{1lb}{2000\frac{lb}{ton}}\right)\}}{(62\ ft \times 260\ ft)} = 2.80\ tsf \qquad F.S. = \frac{4.5\ tsf}{2.80\ tsf} = 1.6$$

Design Confirmation

Although the alternative framing system was designed with the aid of RAM Structural Systems, the critical columns and beams were analyzed. To ensure the members can withstand the design loads, a column line was chosen where the longest spans of beams transfer their load. Additionally, the moments and deflections were checked on the longest spanning member of each size found on a typical floor plan. The deflection limit of $\ell/240$ was chosen to maintain deflections below 1.0 inch in order to ensure the longevity of the structure. The following results conclude each member was sufficiently designed with a safety factor of 1.5.

Critical Column E – 3						
Floor	**ϕPn (kips)**		**Pu (kips)**	**Mu (kip-ft)**	**Size**	
Roof	447	≥	34.3	25.8	W14X43	√
31	447	≥	109.1	12.9	W14X43	√
30	447	≥	163.6	12.9	W14X43	√
29	447	≥	218.2	12.9	W14X43	√
28	447	≥	327.4	12.9	W14X43	√
27	557	≥	382.1	13.1	W14X53	√
26	700	≥	436.8	13.1	W14X61	√
25	700	≥	491.6	13.1	W14X61	√
24	781	≥	546.5	13.1	W14X68	√
23	942	≥	601.4	13.3	W14X82	√
22	1120	≥	656.4	13.1	W14X90	√
21	1120	≥	711.4	13.1	W14X90	√
20	1120	≥	766.5	13.1	W14X90	√
19	1230	≥	821.6	13.3	W14X99	√
18	1360	≥	876.9	13.3	W14X109	√
17	1500	≥	932.2	13.5	W14X120	√
16	1500	≥	987.5	13.5	W14X120	√
15	1640	≥	1042.9	13.6	W14X132	√
14	1640	≥	1042.9	13.6	W14X132	√
12	1820	≥	1098.4	13.7	W14X145	√
11	1820	≥	1153.9	13.7	W14X145	√
10	1990	≥	1209.5	13.8	W14X159	√
9	1990	≥	1265.1	13.8	W14X159	√
8	2210	≥	1320.9	14	W14X176	√
7	2210	≥	1376.7	5.5	W14X176	√

Figure 24: Critical Column Evaluation

Critical Beams on Typical Floor									
Beam Size	**Span (ft)**	**ϕMu (k-ft)**		**Mn (k-ft)**		**L/240 (in)**		**Total Deflection (in)**	
W8X10	23.8	32.9	≥	31	√	1.19	≥	0.18	√
W12X26	23.8	140	≥	31	√	1.19	≥	0.81	√
W14X22	23.8	125	≥	31	√	1.19	≥	0.48	√
W24X55	27.5	503	≥	360	√	1.38	≥	0.92	√
W16X26	17.1	166	≥	139	√	0.86	≥	0.54	√
W18X40	22.25	294	≥	236	√	1.11	≥	0.69	√
W12X16	23.8	75.4	≥	31	√	1.19	≥	0.36	√
W10X12	12.2	46.9	≥	37	√	0.61	≥	0.25	√
W16X31	27.5	203	≥	360	√	1.38	≥	0.85	√

Figure 25: Beam Checks

The following table shows the calculated stiffness values for the lateral system. Each frame was initially modeled in RAM Structural Systems with a 1,000 kip horizontal force at the top story. Once the displacement was found, story drift was calculated for seismic and wind forces. Based upon calculations for the entire height of the building, the lateral forces used are height-appropriate for the isolated frame of the residential tower. The total drift confirms the lateral systems pass against wind and seismic forces. As a preliminary design, these values illustrate that a more optimal lateral system can still be explored.

Floor	Stiffness Y	Stiffness X-M	Stiffness X-X	Seismic - Story Drift Y (in)	Seismic - Story Drift X (in)	Wind - Story Drift Y (in)	Wind - Story Drift X (in)
Roof	0	13	13	-	-	-	0.51
31	0	13	13	-	2.81	-	0.47
30	0	14	14	-	3.33	-	0.45
29	15	15	15	1.59	3.00	0.17	0.40
28	16	16	16	1.41	2.35	0.16	0.38
27	17	17	17	1.25	2.10	0.15	0.35
26	18	18	18	1.10	1.88	0.14	0.33
25	20	19	19	0.96	1.67	0.13	0.31
24	22	21	21	0.83	1.47	0.11	0.29
23	24	23	23	0.72	1.30	0.10	0.25
22	26	24	24	0.61	1.14	0.09	0.23
21	29	27	27	0.52	0.99	0.08	0.21
20	33	29	29	0.43	0.85	0.07	0.19
19	37	32	32	0.36	0.73	0.06	0.18
18	43	36	36	0.29	0.62	0.05	0.15
17	50	39	39	0.24	0.52	0.05	0.14
16	59	44	44	0.18	0.46	0.04	0.12
15	71	51	51	0.14	0.36	0.03	0.10
14	87	56	59	0.10	0.29	0.02	0.09
12	109	66	66	0.07	0.23	0.02	0.08
11	142	74	74	0.05	0.18	0.01	0.07
10	194	88	88	0.03	0.15	0.01	0.06
9	260	102	102	0.02	0.11	0.01	0.05
8	311	94	94	0.02	0.09	0.01	0.05
7	510	116	116	0.01	0.08	0.00	0.04
6	-	-	-	-	-	-	-
Total:				10.93”	26.71”	1.50”	5.50”
				$\Delta_{allowable}$=40.64” Passes		$\Delta_{allowable}$=7.61” Passes	

Figure 26: Lateral Analysis on Proposed System

Conclusions

The original structure of 360 State Street consisted of a cast-in-place structural system throughout the entire residential tower. Due to local trades being unable to provide a competitive cost and schedule, the design was later changed to staggered steel trusses. It was assumed that the engineers were not able to fully investigate alternative design solutions in order to meet the original deadline for complete construction documents. The intent of the report was to verify that staggered steel trusses were the best solution by comparing the existing design to an alternative framing system.

Only the upper twenty-six stories of the building were considered and hollow core planks were maintained as the dominate floor system. The main objective to the alternative design was to avoid negatively impacting the original architectural floor plans. The proposed design uses a more traditional steel framing plan for the gravity and lateral system. Steel was chosen based on the ability of local contractors being able to deliver such a construction. Additional goals included making design decisions which would result in a more durable structure with viable and sustainable solutions.

The original design goals for the alternative system included:

- Increasing strength and rigidity with additional structural elements.
- Minimizing floor depth by shortening span lengths.
- Decreasing overall building weight.
- Optimizing the lateral system and foundation.

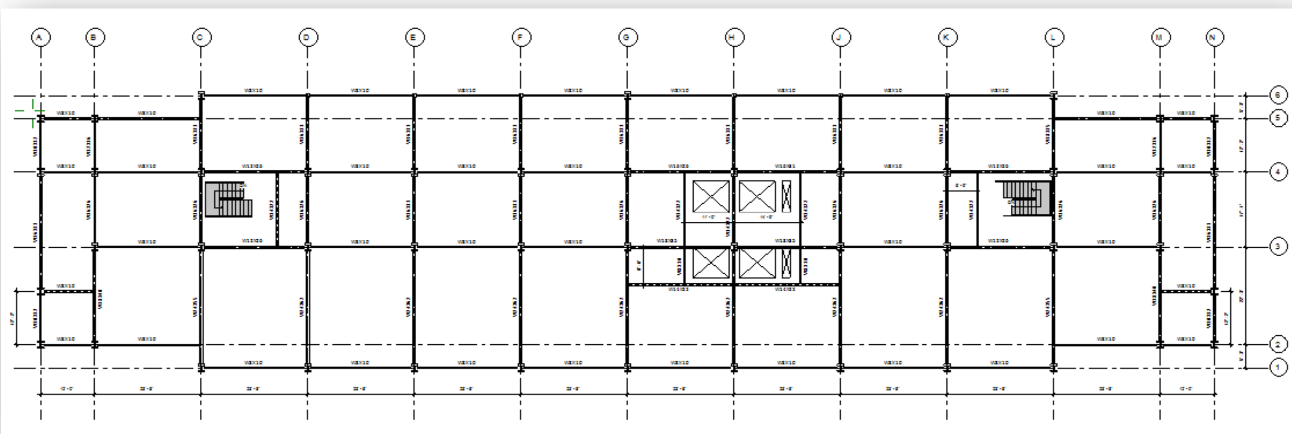


Figure 27: Typical Framing Plan of Proposed Design

As seen in Figure 27, the proposed framing system adds two rows of columns throughout the tower. Beams tie the entire structure together as well as frame the floor openings. Several variations of the above floor plan were considered. An analysis in RAM Structural Systems concluded that five rows of columns would be too congested for construction and three rows of columns would cause undesirable deflections in the beams. Both schemes proved to be heavier than the final design.

The final column locations were also determined upon the architectural floor plans. Each apartment unit was designed to fit between two trusses in the original design. In order to maintain that layout, the columns were spaced similarly at 23’ – 8” across the long dimension of the building. The locations of the interior columns were determined based on the ability to hide the framing within the wall

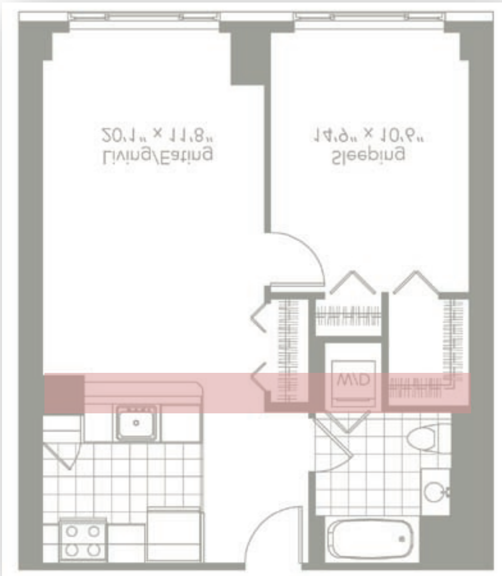


Figure 28: Typical Apartment Unit

assemblies. Columns along gridline 3 are hidden within the corridor wall and the columns on gridline 4 are hidden within the walls separating the apartment units. Although the columns can be easily hidden, the beams along gridline 4 are partially visible. As seen in Figure 28, the beam runs through the opening in the kitchen decreasing the height of the doorway to 7' – 2". This is unacceptable and unfortunately creates a large negative impact on the overall architectural layout. Overall the proposed system uses a total of 3543 members which does increase the strength and rigidity of the structure.

As previously stated, the original floor system was maintained as a constant in the investigation of staggered steel trusses. The hollow core planks span across the long dimension of the building and bear onto the top flange of the adjacent beams. The original intention for the floor system was to decrease the length of each bay in order to achieve a thinner plank however; by maintaining the 23' – 8" spacing, the planks remain at 8 inches thick. It is possible for a 6 inch plank to span 24' – 0" with a 2 inch concrete topping though severe deflections would occur. Although the thickness of the floor assembly did not change, it was possible to decrease the floor-to-floor to 9' – 4" throughout the tower. The original goal was to gain another level of apartment units however; the architect stated the floor plate would have been less efficient with not enough rentable space compared to the gross square footage of the building if the height exceeded thirty-two stories.

The proposed lateral systems of 360 State Street included three unique frames. The first incorporated diagonal braces across two stories in an exterior bay. The second frame consisted of 192 moment connections across five columns on gridline 4. In order to avoid disrupting the apartment layouts any further, moment connections were deemed the least intrusive. Lastly, an X – brace frame was developed similarly to the one found in the original design. After a visit to the site, it was found that the braces actually span across apartment windows in 360 State Street. Therefore; it was the intent of the preliminary design to change the location of the frame. Furthermore, shear walls were initially considered as an addition to the lateral systems however; each would be inefficient in design and expensive to construct.

Figure 29: X – braces on 360 State Street

The changes proposed in the alternative framing plan can be mildly integrated into the existing architectural layout and the existing structural design of the concrete base. The gridlines were changed slightly however; the concrete columns below already have a similar spacing. The steel columns can easily transition down the structure and guide forces to the foundations. The array of columns also distributes the loads more evenly across the foundation. With 3,543 structural members, the overall building weight slightly decreased. A quick check of the bearing capacity has revealed that the foundations are more than capable of supporting the alternative design. Mat slab foundations are typically used for heavy building loads however; an alternative foundation design can be utilized to optimize the cost, schedule, and weight of the building.

Comparing the existing and the proposed systems, staggered steel trusses were most likely chosen for their dual-ability to act as a gravity system as well as a lateral system. Considering the amount of time allotted to the building's redesign, staggered steel trusses provided the easiest solution. Design guides are even available through the *American Institute of Steel Construction* to speed up the process. Perhaps the largest benefit of the system of trusses is the synchronized distribution of loads across the structure. With a high water table and sandy soils, staggered trusses were ideal to complement the mat slab foundation. Additionally, the system provides a strong and rigid structure by incorporating a hollow core plank floor system. The prefabricated structural elements would also have been an ideal solution to the limited storage space available on site. It provides the most flexibility in the architectural design as well as the construction of the building.

Altogether, either framing system would be successful within 360 State Street. A traditional steel design offers consistency throughout a structure while staggered trusses offer the opportunity to make unique variations in the architectural layout. Roughly half of the design goals were met in the proposed design however; the existing framing system remains the best solution. A cost and schedule comparison will be included in a following breadth topic.

Breadth Study – Building Envelop

An alternative glass façade will be studied as a consideration for the longevity of 360 State Street. The architecture of downtown New Haven, Connecticut is consistent with low-rise office buildings and masonry façades. The charm of the old town green coupled with the historic appeal of Yale University's campus has swayed any intentions to express a more exciting aesthetic. With the recent completion of several unique medical facilities, glass and color is starting to bring the city to life. In this study, an all glass façade will be explored using alternative materials to masonry. This study will also explore opportunities to make 360 State Street more sustainable.

Figure 30: Under Construction – April 2010

Existing Façade Design

The current façade of the upper twenty-six stories of 360 State Street consists of precast concrete panels, aluminum panels, and Chicago-style windows. The vertical precast panels accentuate the location of the staggered steel trusses while the horizontal panels highlight the location of the spandrel beams. The aluminum panels are located in between the windows. As seen in Figure 31, the glass varies depending on its location. Spandrel glass (denoted as $ in Figure 31) is a type of wall assembly where glass is located on the exterior but the interior is composed of insulation, studs, and gypsum board with an architectural finish. The remaining panels of the windows consist of 1 inch thick tempered glass.

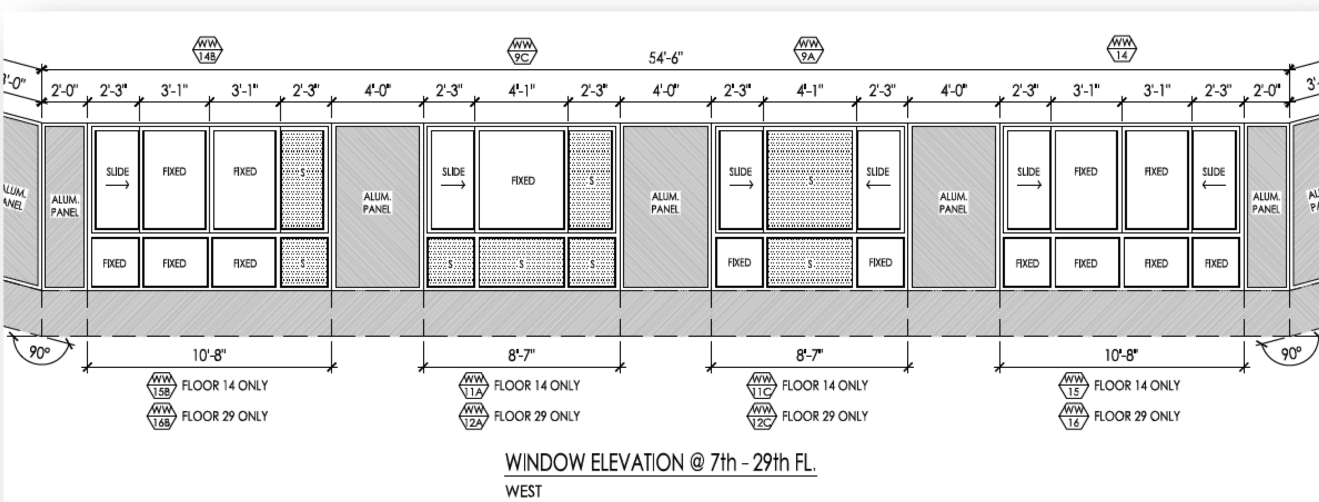


Figure 31: Typical Façade Elevation

Proposed Façade Design

Masonry façades with gridded windows are the norm in downtown New Haven. The location of the city however; provides great views that should be taken advantage of in 360 State Street. The building towers over the town green as well as the historic Wooster Square. Moreover, the building has stunning views of the harbor and nearby state park. The alternative façade design proposes to optimize the window size within each apartment while providing a sustainable alternative.

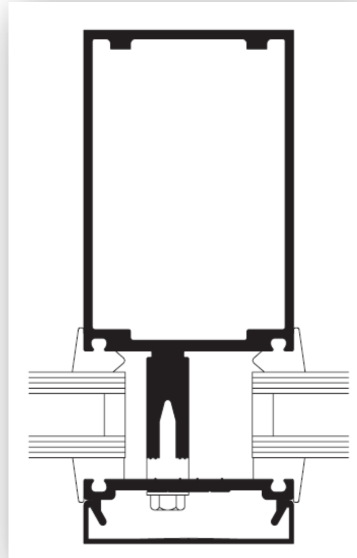


Figure 32: Detail of Typical Mullion

Kawneer, an Alcoa Company, produces a prefabricated façade called *1600 Wall System 3*. The product consists of a 6 inch deep assembly with aluminum or steel mullions that locks in a 1 inch thick piece of glass. The system utilizes a patented invention called *IsoStrut Thermal Break* to create a continuous thermal barrier. Various options are available to customize the façade's appearance. A variety of glass colors are available as well as spandrel glass panels that provide shading and insulation. Furthermore, the wall system has the option to use solar photovoltaic in-fills to generate DC power.

The alternative façade consists of a typical panel that is repeated throughout the tower. Each apartment unit has a window with blue tinted tempered glass that is roughly 6' x 10' in size. Aluminum mullions are used to join the glass together and accentuate the horizontal and vertical lines of the structure. The window is additionally accented with teal and pastel yellow spandrel glass as seen in Figure 33. Overall this façade is easy to install and has unlimited architectural design possibilities.

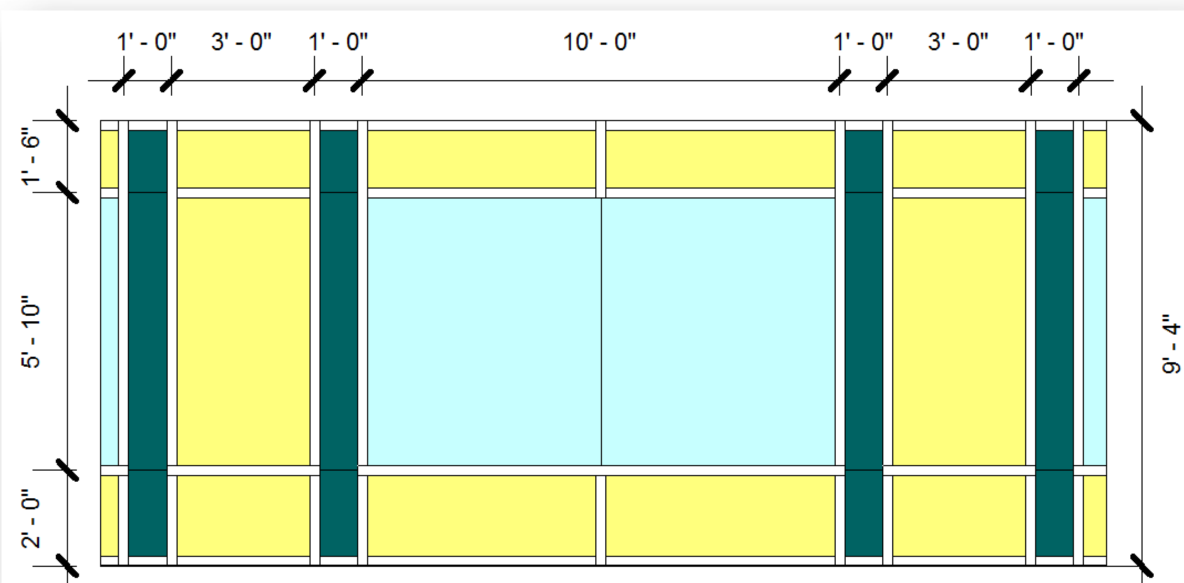


Figure 33: Typical Façade Panel

Conclusion

The existing design of 360 State Street currently expresses the structural system through an arrangement of aluminum and precast panels. Chicago-style windows accent the building providing spectacular views as well as plenty of day-lighting. The alternative façade system can achieve the same design goals intended by the architects. The *1600 Wall System 3* can be customized with a variety of glass products as well incorporate a distinct grid pattern with steel or aluminum mullions.

Figure 34: Existing Façade (left) Proposed Façade (right)

To compare the existing and proposed facades, the thermal properties were investigated. The *1600 Wall System 3* was chosen based on its ability to provide adequate insulation from the elements. The proposed design was found to have an overall heat transfer coefficient (U – value) of 0.07 for a typical panel. In comparison, the existing design was found to have a higher value of 0.12. The proposed panel experiences roughly 240 watts of heat loss per hour compared to 400 watts from the existing façade system. With electricity costing 17.6 ¢ per kilowatt-hour in Connecticut, the new design would save approximately $120,000 a year for the entire residential tower. For this reason alone, the proposed façade would be a positive alternative for 360 State Street.

Description	Overall U - Value	Heat Loss (BTU/hr)	Heat Loss (Watt/hr)	Cost per year for one typical apartment unit
Existing	0.12	1350	396	$611.92
Proposed	0.07	811	238	$367.77

Figure 35: Thermal Performance of Panels

Breadth Study – Cost & Schedule Comparison

An alternative design for the gravity and lateral systems of 360 State Street will significantly impact the overall cost of the project as well as the construction schedule. In this breadth study, a detailed estimate will be conducted for the structural elements in both the existing and proposed framing systems. Additionally, a simple schedule with be developed to compare construction time required for the framing systems. The phasing of the project will be touched upon to understand the construction process. This breadth study will also confirm the feasibility of the alternative design solutions.

Cost Estimate

A rough estimate has been developed using CostWorks®—an electronic catalogue provided by *R. S. Means*—for the existing and proposed building systems. In the following tables, a breakdown of the prices can be seen for materials, labor, and equipment. All categories include overhead and profit.

The overall cost for the staggered steel truss system and the hollow core planks is roughly $9.5 million. Compared to the traditional steel framing with the same hollow core plank system, the cost of construction is roughly $9.8 million. The systems are fairly competitive with one another.

Description	Mat. O&P	Labor O&P	Equip. O&P	Total O&P	Ext. Mat. O&P	Ext. Labor O&P	Ext. Equip. O&P	Ext. Total O&P
Staggered Steel Trusses	$35.39	$ 6.24	$3.21	$ 44.84	$1,803,509.79	$ 317,996.64	$163,584.81	$2,285,091.24
8" Hollow Core Planks	$14.60	$2.01	$0.65	$17.26	$ 6,119,152.00	$842,431.20	$272,428.00	$7,234,011.20
Curtain Wall - Aluminum & Glazing	$68.48	$12.13	$ -	$80.61	$10,804,774.40	$ 1,913,871.40	$ -	$12,718,645.80

Figure 36: Estimate for Existing Systems

Description	Mat. O&P	Labor O&P	Equip. O&P	Total O&P	Ext. Mat. O&P	Ext. Labor O&P	Ext. Equip. O&P	Ext. Total O&P
Traditional Steel	$35.39	$6.24	$3.21	$ 44.84	$2,055,928.97	$362,503.44	$186,480.14	$2,604,912.54
8" Hollow Core Planks	$14.60	$2.01	$0.65	$17.26	$ 6,119,152.00	$842,431.20	$272,428.00	$7,234,011.20
2" Spandrel Glass	$22.28	$7.97	$ -	$ 30.25	$ 2,109,203.04	$754,503.96	$ -	$2,863,707.00
Window Glass - Tempered	$10.33	$2.00	$ -	$ 12.33	$ 651,946.96	$126,224.00	$ -	$778,170.96

Figure 37: Estimate for Proposed Systems

The façade designs have significantly different outcomes in terms of pricing. The original design costs roughly $12.7 million while the proposed design costs around $3.6 million. The design preference of the façade is ultimately up to the architect however; there are many products that can give the same results.

Construction Schedule

The construction schedule is a very important detail in the success of 360 State Street. In order for the building to obtain its Certificate of Occupancy in a timely manner, every activity needs to be planned and coordinated. For the purpose of this breadth study, only the construction time of the major structural elements will be considered. Using *R.S. Means* in combination with the material take-off, it has been calculated that the staggered steel truss framing system would require roughly 102 days of 40 hour work-weeks to erect the trusses. Another 241 days are estimated for the placement of the hollow core planks however; that task can be overlapped with the steel erection. In comparison, the traditional steel framing would take roughly 116 days to complete.

The estimated construction schedule for the façades is significantly longer than the framing systems' timeline. It is estimated to take about 702 days to install the existing façade. Similarly, the proposed façade is estimated at 367 days. The numbers do not look reasonable and perhaps a manufacturer and/or contractor can provide a more accurate timeline.

Construction Process

The site of 360 State Street is roughly half a block between Chapel Street and Court Street. The final footprint of the building covers the entire area therefore; it is important to take advantage of every square foot. The project began with two phases. As the Pitkin Tunnel and the footprint of the tower were being excavated, supplies and equipment were stored in the location of the future retail space. The field offices located offsite in nearby rented spaces. Once the concrete had been poured and cured, storage was moved into the base of the building. As seen in Figure 36, the tower crane is located adjacent to the residential tower.

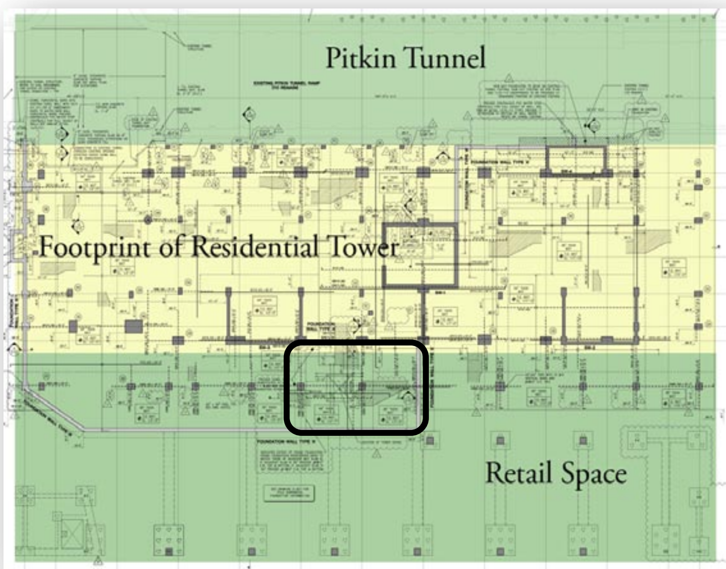


Figure 38: Foundation Plan with tower crane location circled

The next step in the first phase was the erection of the steel trusses. Storage was limited to the hollow core planks because the trusses were lifted right off of the truck beds into place. Every two to three floors, the planks were lifted into place; the façade followed soon after. Once the tower tops off, the second phase of the project can begin. With the footings and piles already in place, steel will be erected on half of the area followed by the placement of concrete double-tee beams. The other half of the area will be used as storage for equipment. Throughout the second phase, all the building material is kept inside the parking garage area safe from the elements. Once the tee-beams are in place, construction on the second half of the retail space will begin. Storage of materials will remain in the parking garage and equipment will be storage on the corner of Chapel and State Street where there is a large setback for pedestrians. The phasing of the project will be similar with either framing system or façade design.

Conclusions

The success of a building project depends upon the feasibility of the design. As illustrated through 360 State Street: if one system doesn't work, find an alternative. The cost and schedule comparison has illustrated that the alternative framing system is a competitive design. Both the staggered steel trusses and the typical steel frame have a similar price range. Additionally, the systems have a comparable construction schedule. More accurate prices and timelines can be achieved by contacting the steel fabricator and/or contractor directly. The only negative found was the beam obstruct along gridline 4 that lowers the head clearance in the apartment units on the North side of the building. More investigation can be made into the traditional steel frame design however; the staggered steel truss system appears to have the complete package.

The façade designs had a very different outcome in their comparison. The existing design was found to be significantly more expensive and time-consuming to install. Although unit pricing was found for the materials based on *R.S. Means*, a manufacturer should be contacted for further information regarding pricing and installation procedures. Prefabrication of wall panels should significantly decrease the installation time required as well as decrease the cost of the material.

Altogether, the existing and proposed systems can easily fit into the current construction schedule of 360 State Street. The phasing of the project allows for clever onsite storage and a carefully coordinated schedule to ensure a timely delivery. To conclude, this report completed what it intended to do—verify the use of the staggered steel trusses.

Appendix A – Existing Floor Plans & Sections

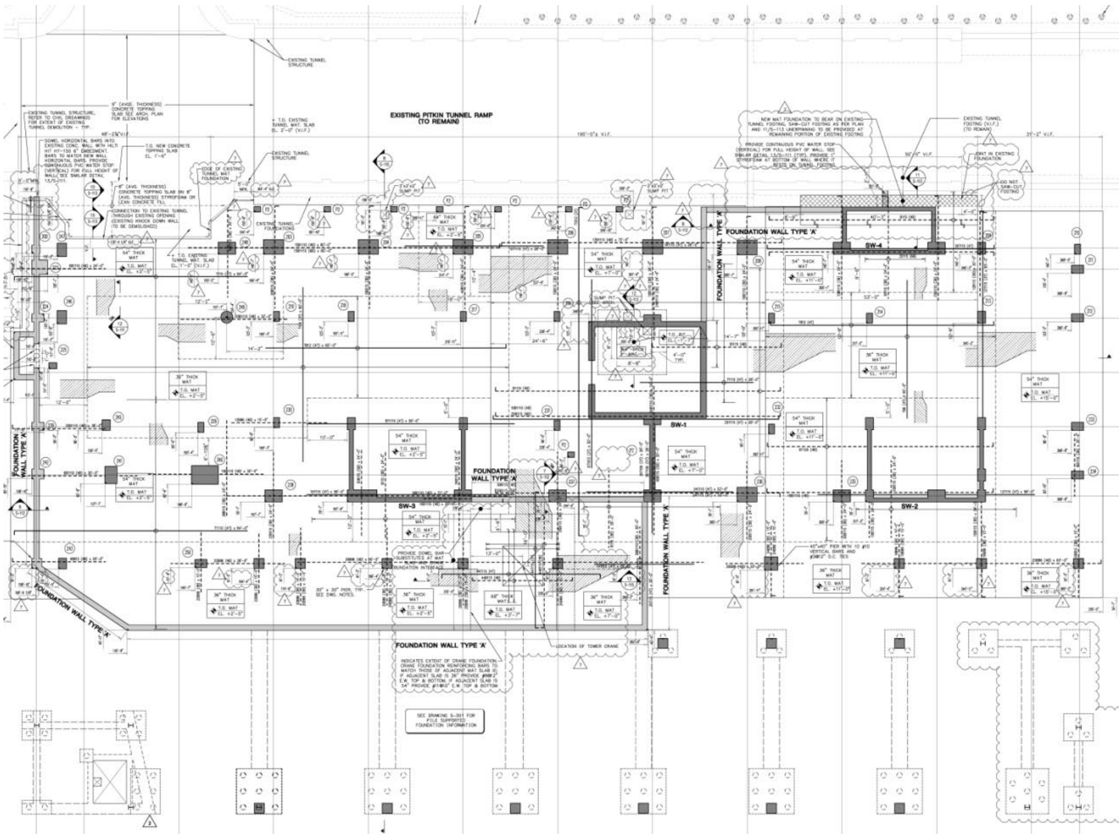


Figure A.1 Foundation Plan, Shear Walls are Shaded

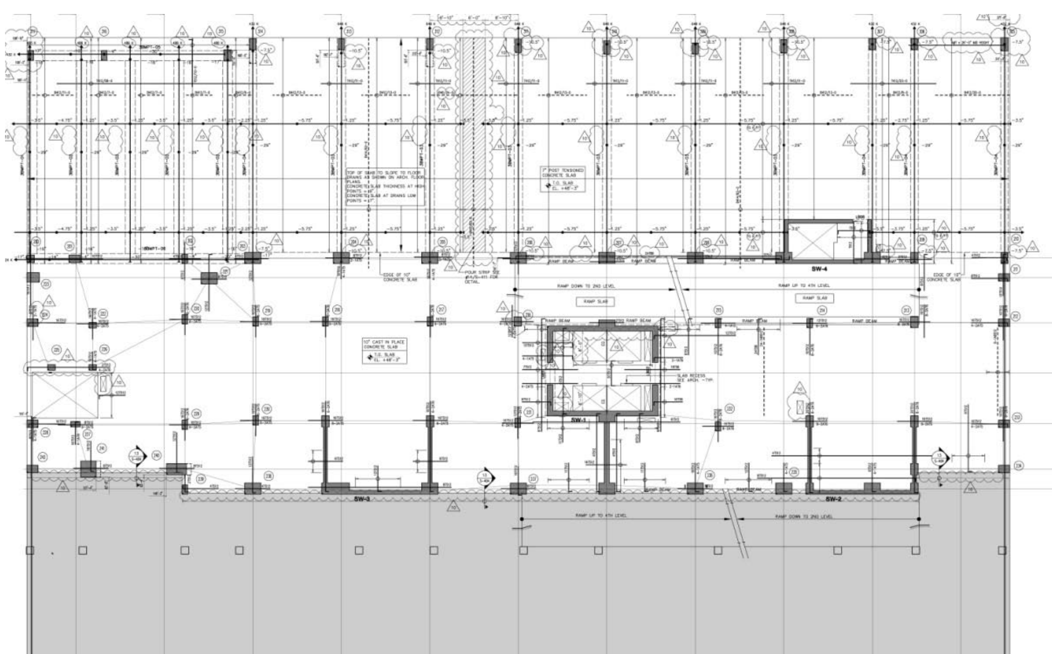


Figure A.2 Second - Fifth Typical Floor Plan

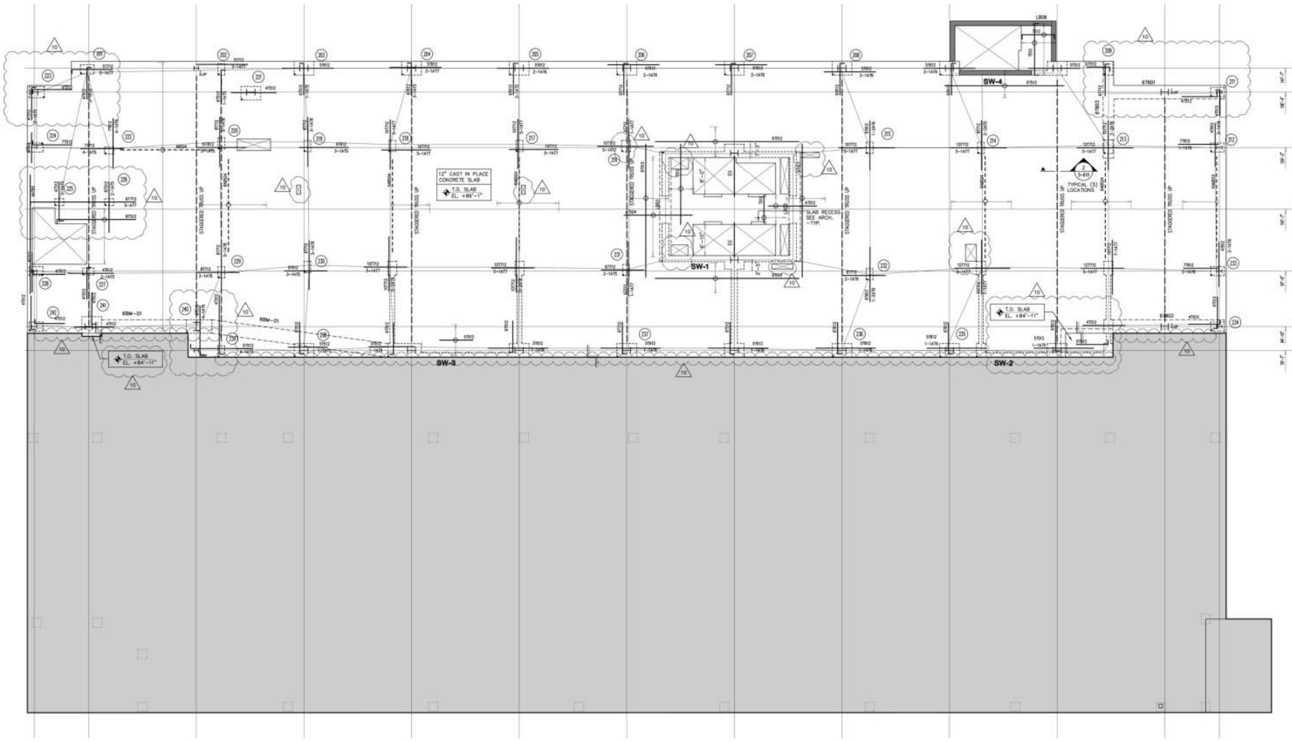


Figure A.3 Terrace & Sixth Floor Plan

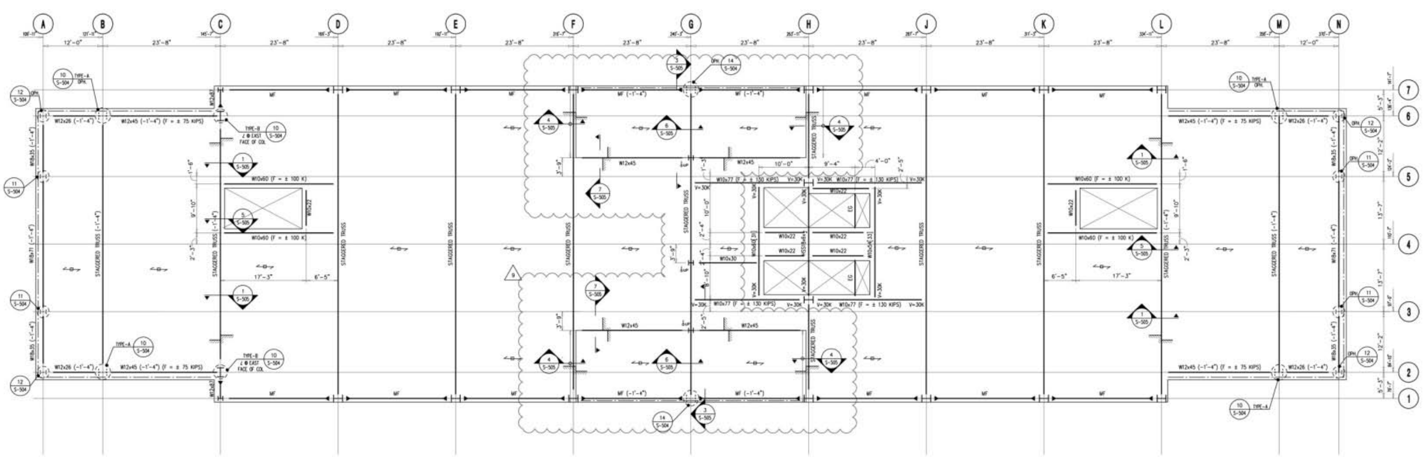


Figure A.4 Typical Floor Plan for Residential Tower

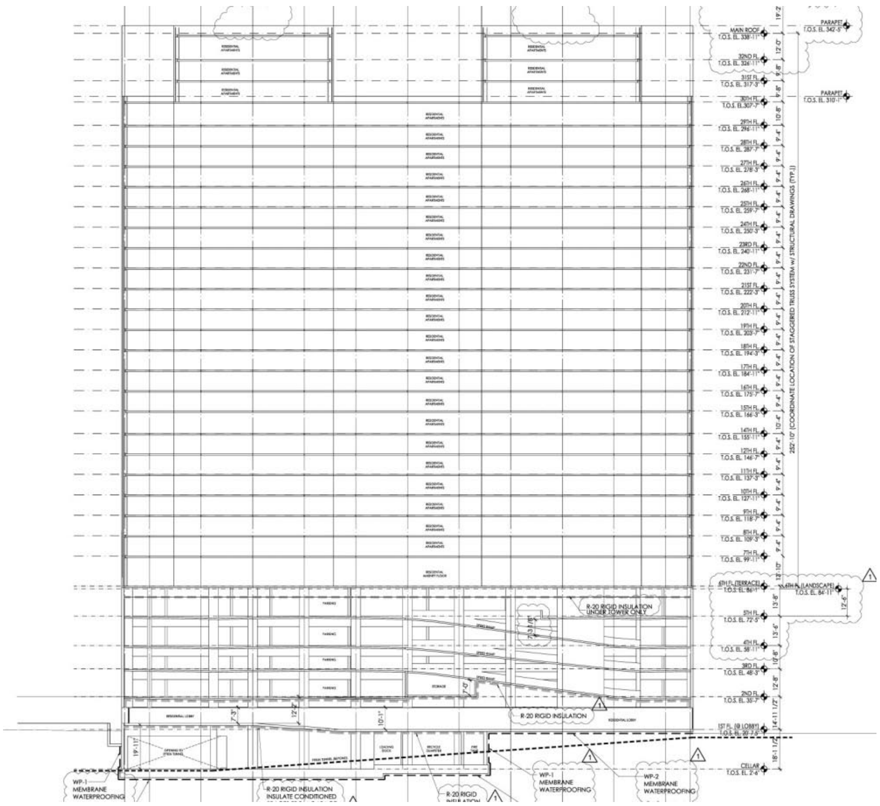


Figure A.5 North/South Building Elevation

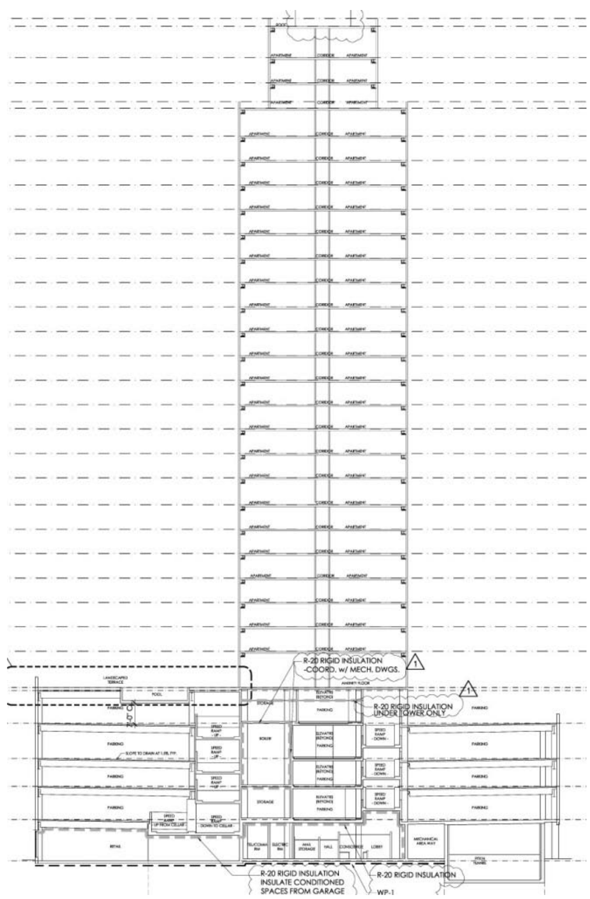


Figure A.6 East/West Building Elevation

Appendix B – Design Standards

Codes Applied to Original Design	Codes Substituted for Analysis
2005 Connecticut State Building Code consisting of the 2003 International Building Code as modified by the 2005 Connecticut Supplement	American Society for Civil Engineers *Minimum Design Loads for Buildings and Other Structures* ASCE-7-05
American Institute of Steel Construction *Specification for Structural Steel Buildings – Allowable Stress Design and Plastic Design* 01 June 1989 (AISC)	American Institute of Steel Construction *Steel Construction Manual, Thirteenth Edition* April 2007 (AISC)
American Concrete Institute *Building Code Requirements for Structural Concrete* ACI 318-02 (ACI)	American Concrete Institute *Building Code Requirements for Structural Concrete and Commentary* ACI 318-08 (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)	

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 6^{th} 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.

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 Criteria outlined by IBC 2003.

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

Appendix C – Load Combinations

Basic Load Combinations			
	All load types included.	Available load types.	Lateral load types only.
i	$1.4(D + F)$	$1.4(D)$	-
ii	$1.2(D + F + T) + 1.6(L + H) + 0.5(Lr \text{ or } S \text{ or } R)$	$1.2(D) + 1.6(L) + 0.5(Lr \text{ or } S \text{ or } R)$	-
iii	$1.2D + 1.6(Lr \text{ or } S \text{ or } R) + (L \text{ or } 0.8W)$	$1.2D + 1.6(Lr \text{ or } S \text{ or } R) + (L \text{ or } 0.8W)$	$0.8W$
iv	$1.2D + 1.6W + L + 0.5(Lr \text{ or } S \text{ or } R)$	$1.2D + 1.6W + L + 0.5(Lr \text{ or } S \text{ or } R)$	$1.6W$
v	$1.2D + 1.0E + L + 0.2S$	$1.2D + 1.0E + L + 0.2S$	$1.0E$
vi	$0.9D + 1.6W + 1.6H$	$0.9D + 1.6W$	$1.6W$
vii	$0.9D + 1.0E + 1.6H$	$0.9D + 1.0E$	$1.0E$

D = dead load
E = earthquake load
F = load due to fluids w/ defined pressures
H = load due to lateral earth pressure, or ground water pressure
L = live load
Lr = roof live load
R = rain load
S = snow load
T = self-straining force
W = wind load

Table C.1: Summary of Load Combinations from ASCE 7 – 05

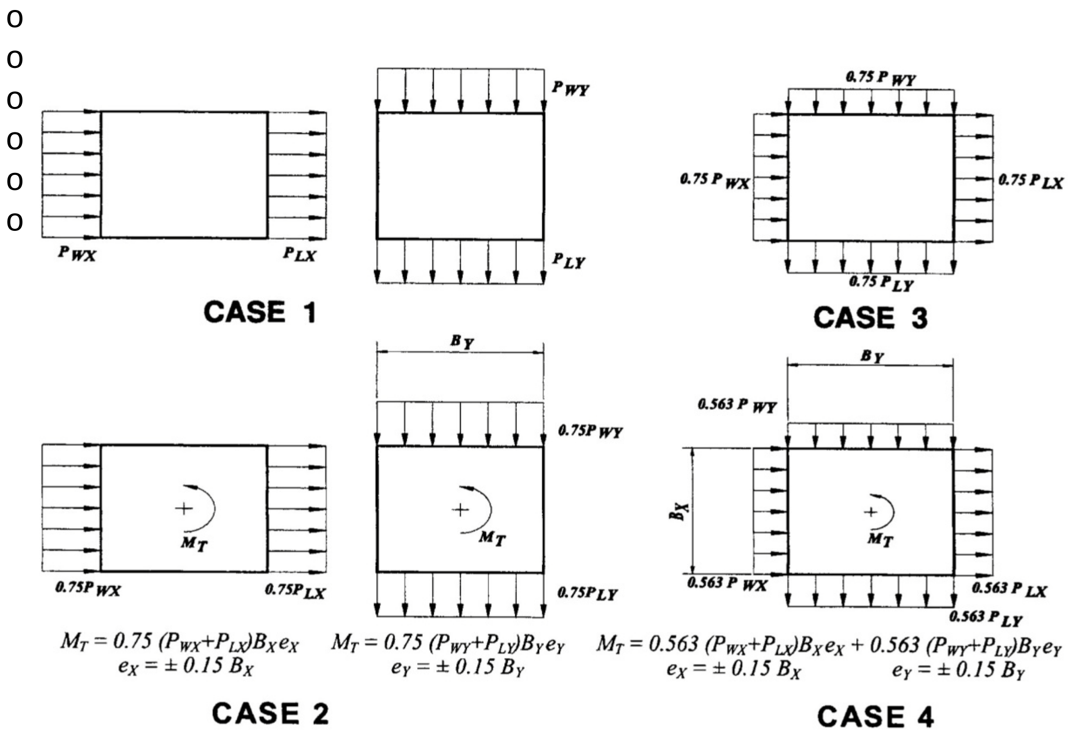


Figure C.2: Wind Load Cases from ASCE 7 – 05

Appendix D – Loads

The following loads have been determined based on the Live and Dead Load Schedule on 360 State Street's S001 drawing in combination with ASCE 7 – 05. Additionally, the building material self-weights have been found according to manufacturer specifications as well as the ASIC Steel Manuel. Details have been provided to illustrate typical assemblies found throughout the building. Furthermore, the snow load of 21 pounds per square foot has been calculated according to Chapter 7 of ASCE 7 – 05. The values found in this section will be used throughout the analysis of 360 State Street.

Level	Load Type	Dead Load (psf)	Super-Imposed Dead Load (psf)	Live Load (psf)
Foundation	Loading Dock	Varies on Mat Slab Thickness	40	100
Typical Residential	Residential	61	20	40
	Private Terrace	61	10	60
	Public (Corridor)	61	20	100
Mechanical/Roof	Mechanical	61	20	40

Table D.1: Design Dead & Live Load Schedule.

Building Material Self-Weight (psf)	
Floor Assembly:	
8" Hollow Core Plank	57
2" Concrete Topping	25
7/8"Wood Finish	4
Interior Walls:	
2 x 4 Metal Studs	4
1/2" Gypsum Board	2
Precast Panel & Glazing Façade:	
Aluminum Panel	2
1" Structural Glass	15
Window Assembly	8
Roof Assembly:	
8" Hollow Core Plank	57
Rigid Insulation	1.5

Table D.2: Building Material Schedule

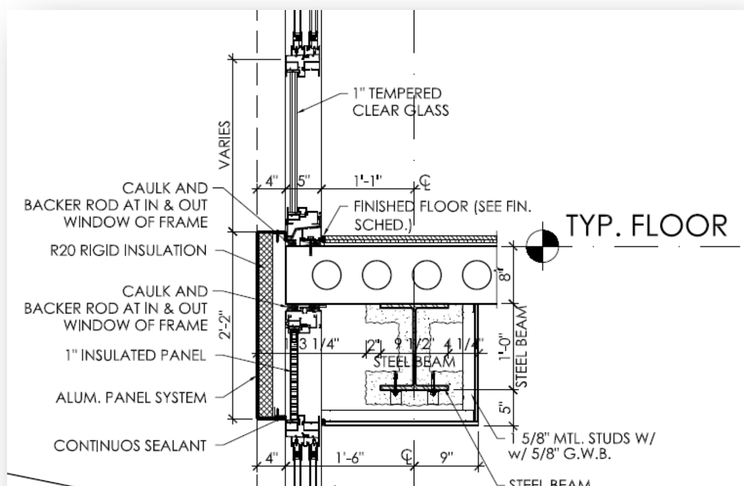


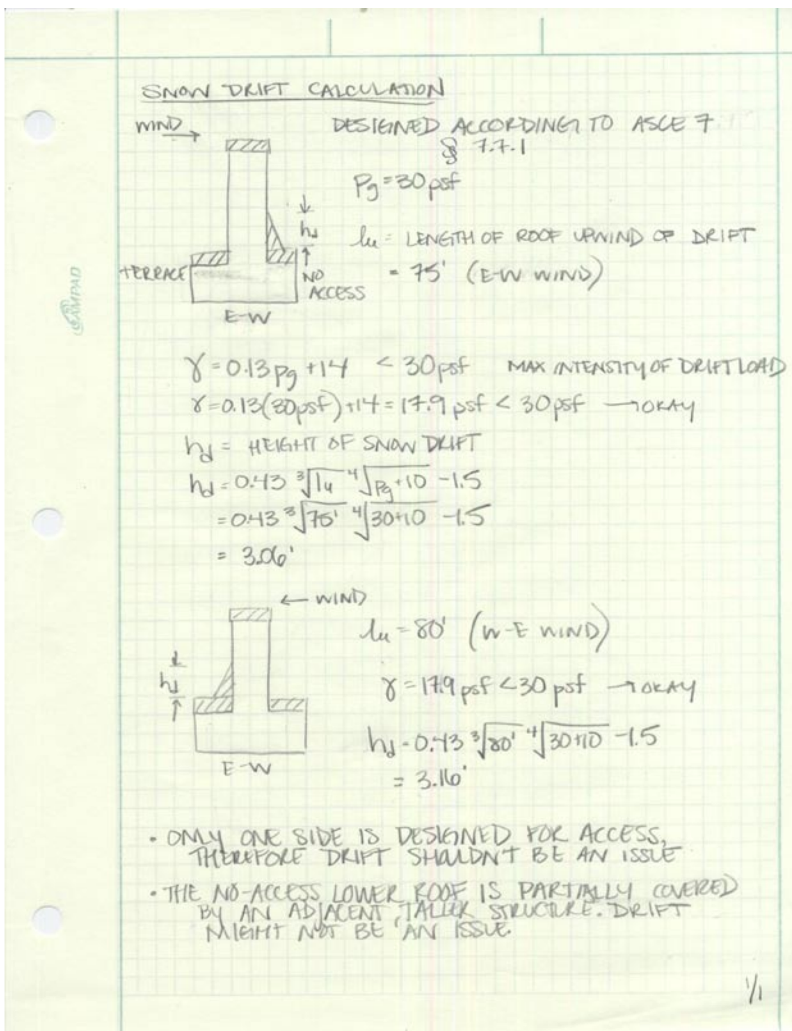
Figure D.1: Typical Wall Section

Snow Load

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 D.3: Snow Design Criteria as outlined on drawing S001

A calculation for snow drift was also completed and can be found below. 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 main roof could miss the lower levels entirely however; the parapet walls running along the perimeter of the terrace could cause some 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 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.2: Snow Drift Calculation

Wind Load

For the lateral analysis of 360 State Street, only wind and seismic forces will be considered. The essential design criteria have been given by the structural engineers on page S001 of the construction document. This section will include a summary of the un-factored design loads as found through ASCE 7 – 05. Both wind and seismic loads were calculated by following an example provided by David A. Fanella in his publication *Structural Load Determination Under 2006 IBC & ASCE/SEI 7 – 05.*

Wind Pressures		
Height above ground level, z (ft)	K_z	q_z (psf)
338.58	1.40	36.9
300	1.35	35.5
250	1.28	33.7
200	1.20	31.6
180	1.17	30.8
160	1.13	29.8
140	1.09	28.7
120	1.04	27.4
100	0.99	26.1
90	0.96	25.3
80	0.93	24.5
70	0.89	23.4
60	0.85	22.4
50	0.81	21.3
40	0.76	20.0
30	0.70	18.4
25	0.66	17.4
20	0.62	16.3
15	0.57	15.0
Leeward (all)	-	36.9

Table D.4: Schedule of Wind Pressures

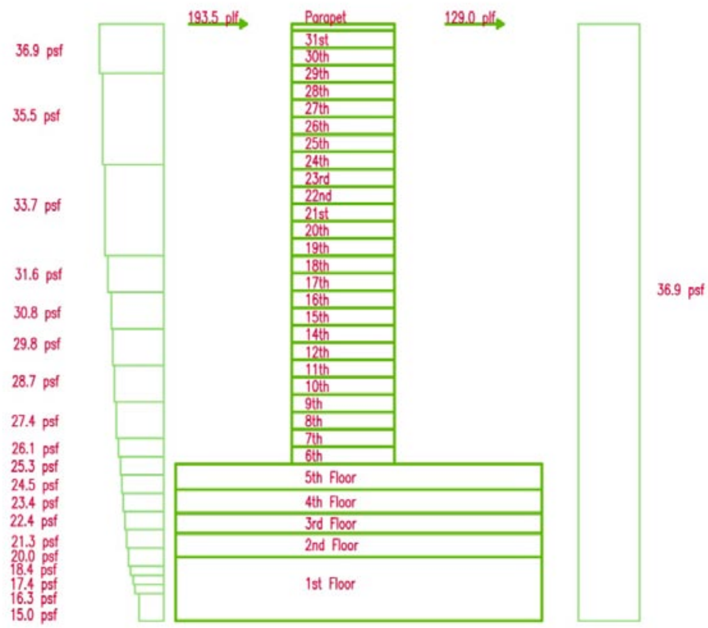


Figure D.2: Diagram of Wind Pressures

Wind Load Design Criteria			
Basic Wind Speed (3 s Gust)	V = 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 D.5 Wind Design Criteria According to ACSE 7 – 05

Sample Calculation

The wind calculations were determined according to Chapter 6 in ASCE 7 – 05. Additionally, an example problem from David A. Fanella's *Structural Load Determinations Under 2006 IBC and ASCE/SEI 7-05* was used as a reference. Table B.1 can be referenced for specific design criteria for 360 State Street.

$q_z = 0.00256 K_z K_{zt} K_d V^2 I$ (eq 6-15)

K_z was interpolated for each floor height (Table 6-3)

$q_{z\ (25th\ floor)} = (0.00256)*(1.29)*(1.0)*(0.85)*(110^2)*(1.0) = 33.97$ psf

qGC_p = External Pressure where C_p = 0.8 windward

$qGC_{p\ (25th\ floor)} = (33.97\ psf)*(0.85)*(0.8) = 23.10$ psf

$q_h GC_{pi}$ = Internal Pressure

$q_h GC_{pi\ (25th\ floor)} = (33.97\ psf)*(-0.18) = -6.54$ psf

Net Pressure p was determined by the summation of the external and internal pressures.

Force (k) = (Floor height)*(Length of building)*(External pressure)/1000

$F_{(25th\ floor)} = (9.34\ ft)*(276\ ft)*(23.10\ psf) = 59.5$ k E/W

Shear (k) = Force of current floor + Force of above floor

$S_{(25th\ floor)} = 59.5\ k + 465.8\ k = 525.3$ k E/W

The frame analysis excludes internal pressures and suction/uplift pressures.

Seismic Loads

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 = 1266 k**

Table D.6: Seismic Design Criteria used for Calculations

The seismic calculations were determined according to Chapter 12 in ASCE 7 – 05. Additionally, an example problem from David A. Fanella's *Structural Load Determinations Under 2006 IBC and ASCE/SEI 7-05* was used as a reference. Table B.1 can be referenced for specific design criteria for 360 State Street.

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	1552	326.92	10299988	70.6	70.6
31	2020	317.25	12808016	87.8	158.4
30	1992	307.58	12050139	82.6	240.9
29	1739	296.92	9970223	68.3	309.3
28	1726	287.58	9426726	64.6	373.9
27	1737	278.25	9022799	61.8	435.7
26	1728	268.92	8522431	58.4	494.1
25	1739	259.58	8128295	55.7	549.8
24	1730	250.25	7648473	52.4	602.2
23	1743	240.92	7273374	49.8	652.1
22	1734	231.58	6814017	46.7	698.8
21	1747	222.25	6449000	44.2	742.9
20	1738	212.92	6010761	41.2	784.1
19	1756	203.58	5672985	38.9	823.0
18	1751	194.25	5267383	36.1	859.1
17	1773	187.92	5071509	34.8	893.9
16	1762	175.58	4545893	31.2	925.0
15	1773	166.25	4209848	28.8	953.9
14	1762	155.92	3794932	26.0	979.9
12	1775	146.58	3480530	23.9	1003.7
11	1765	137.25	3131580	21.5	1025.2
10	1778	127.92	2834410	19.4	1044.6
9	1768	118.58	2511865	17.2	1061.8
8	1783	109.25	2236413	15.3	1077.1
7	1782	99.92	1951460	13.4	1090.5
6	7345	86.03	6407073	43.9	1134.4
5	11229	72.42	7538739	51.7	1186.1
4	11171	58.92	5480975	37.6	1223.7
3	10208	48.25	3697061	25.3	1249.0
2	10889	35.58	2482428	17.0	1266.0
	94995		184739326	1266.0	
V =	**1266**				

Appendix E – Building Weight Calculations

Table E.1: Existing Building Design						
Floor	Floor Weight (lbs)	∑ Column Weight (lbs/flr)	∑ Truss Weight (lbs/flr)	Curtain Wall Weight (lbs/flr)	Shear Wall Weight (lbs/flr)	Story Weight (k)
Roof	1111200	17808	63580	359550	-	1552
31	1570000	25945	64120	359550	-	2020
30	1570000	25945	36416	359550	-	1992
29	1198100	35809	45520	459660	-	1739
28	1198100	31345	36416	459660	-	1726
27	1198100	34119	45520	459660	-	1737
26	1198100	34119	36416	459660	-	1728
25	1198100	35866	45520	459660	-	1739
24	1198100	35866	36416	459660	-	1730
23	1198100	39462	45520	459660	-	1743
22	1198100	39462	36416	459660	-	1734
21	1198100	44188	45520	459660	-	1747
20	1198100	44188	36416	459660	-	1738
19	1198100	52612	45520	459660	-	1756
18	1198100	52612	40296	459660	-	1751
17	1198100	64427	50370	459660	-	1773
16	1198100	64427	40296	459660	-	1762
15	1198100	64427	50370	459660	-	1773
14	1198100	64427	40296	459660	-	1762
12	1198100	67304	50370	459660	-	1775
11	1198100	67304	40296	459660	-	1765
10	1198100	70284	50370	459660	-	1778
9	1198100	70284	40296	459660	-	1768
8	1198100	74907	50370	459660	-	1783
7	1198100	74907	48928	459660	-	1782
6	6725050	99365	61160	459660	-	7345
5	6575000	634835	-	-	3026928	-
4	6575000	626940	-	-	2989285	-
3	6575000	495515	-	-	2362642	-
2	6002625	588395	-	-	2805499	-
Totals (k)	63062075	3677094	1182734	11650830	11184354	
Overall Building Weight = 90, 757 kips						

Table E.2: Proposed Building Design						
Floor	Floor Weight (lbs)	∑ Column Weight (lbs/flr)	∑ Beam Weight (lbs/flr)	Curtain Wall Weight (lbs/flr)	Shear Wall Weight (lbs/flr)	Story Weight (k)
Roof	1111200	7351	34550	359550	-	1513
31	1570000	7351	34550	359550	-	1971
30	1570000	7351	34550	359550	-	1971
29	1198100	17183	47409	459660	-	1722
28	1198100	17244	46093	459660	-	1721
27	1198100	17576	46093	459660	-	1721
26	1198100	18168	46093	459660	-	1721
25	1198100	18756	46093	459660	-	1719
24	1198100	19332	46093	459660	-	1723
23	1198100	20079	46093	459660	-	1723
22	1198100	20844	46093	459660	-	1724
21	1198100	21341	46093	459660	-	1725
20	1198100	21930	46093	459660	-	1722
19	1198100	22581	46093	459660	-	1726
18	1198100	23610	46093	459660	-	1726
17	1198100	17244	46093	459660	-	1727
16	1198100	17244	46093	459660	-	1721
15	1198100	17244	46093	459660	-	1721
14	1198100	17244	46093	459660	-	1718
12	1198100	17244	46093	459660	-	1718
11	1198100	17244	46093	459660	-	1721
10	1198100	17244	46093	459660	-	1718
9	1198100	17244	46093	459660	-	1721
8	1198100	17244	46093	459660	-	1721
7	1198100	17244	46093	459660	-	1721
6	6725050	446830	46093	459660	-	1718
5	6575000	634835	-	-	3026928	-
4	6575000	626940	-	-	2989285	-
3	6575000	495515	-	-	2362642	-
2	6002625	588395	-	-	2805499	-
Totals (k)	63062075	3225652	1165105	11650830	11184354	
Overall Building Weight = 90,288 kips						

Table E.3: Center of Rigidity

	Existing		Proposed	
Floors	X	Y	X	Y
Roof	94.8	32	131.45	31.53
31	94.8	32	131.45	31.57
30	94.8	32	131.45	31.5
29	94.8	32	131.45	32.55
28	94.8	32	131.44	31.46
27	94.8	32	131.44	31.47
26	94.8	32	131.43	31.37
25	94.8	32	131.43	31.39
24	94.8	32	131.41	31.27
23	94.8	32	131.42	31.26
22	94.8	32	131.4	31.13
21	94.8	32	131.4	31.1
20	94.8	32	131.38	31
19	94.8	32	131.39	30.96
18	94.8	32	131.37	30.86
17	94.8	32	131.41	30.9
16	94.8	32	131.39	30.91
15	94.8	32	131.53	31.49
14	94.8	32	131.48	31.21
12	94.8	32	131.7	32.23
11	94.8	32	131.67	32.07
10	94.8	32	131.86	32.75
9	94.8	32	131.91	33.12
8	94.8	32	132.16	34.43
7	94.8	32	132.18	34.17
Avg	94.8	32	131.544	31.748

Appendix F – Proposed Design Framing Plans

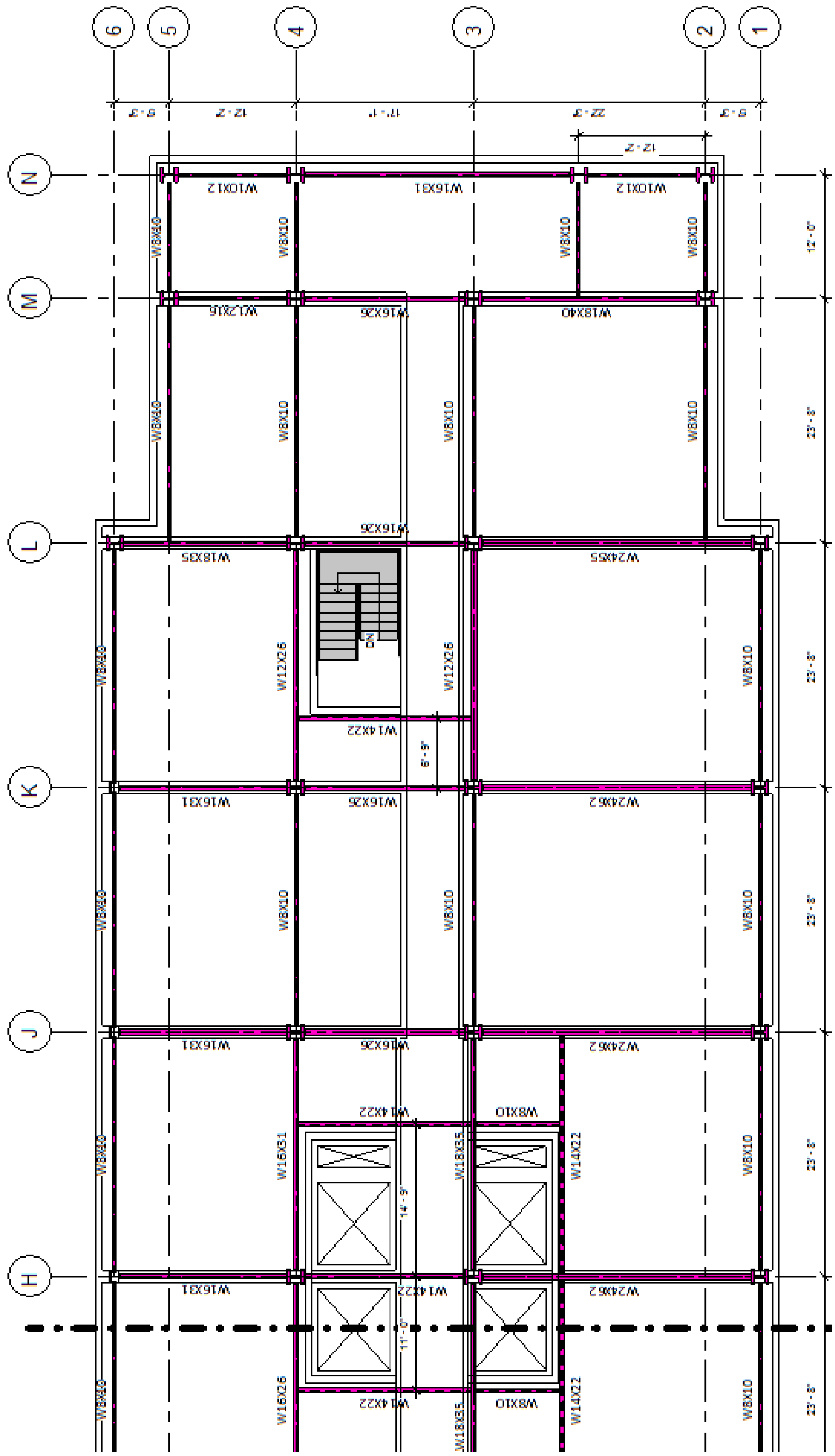


Figure F.1: Typical Framing Plan for Residential Levels of Proposed Design
Note: Match line to Figure F.2

Figure F.2: Typical Framing Plan for Residential Levels of Proposed Design
Note: Match line to Figure F.1

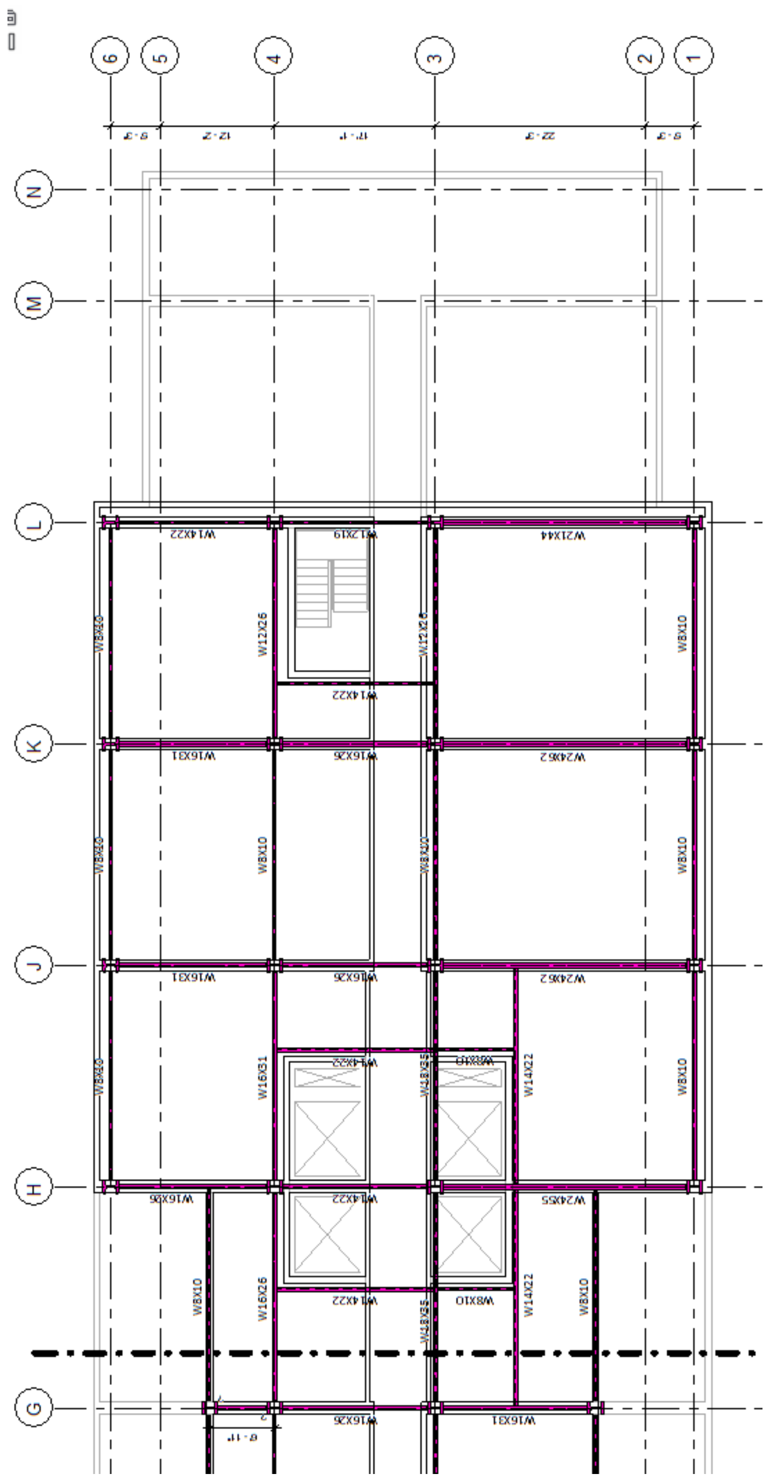


Figure F.3: Typical Framing Plan for Levels 29 – 32 of Proposed Design
Note: Match line to Figure F.4

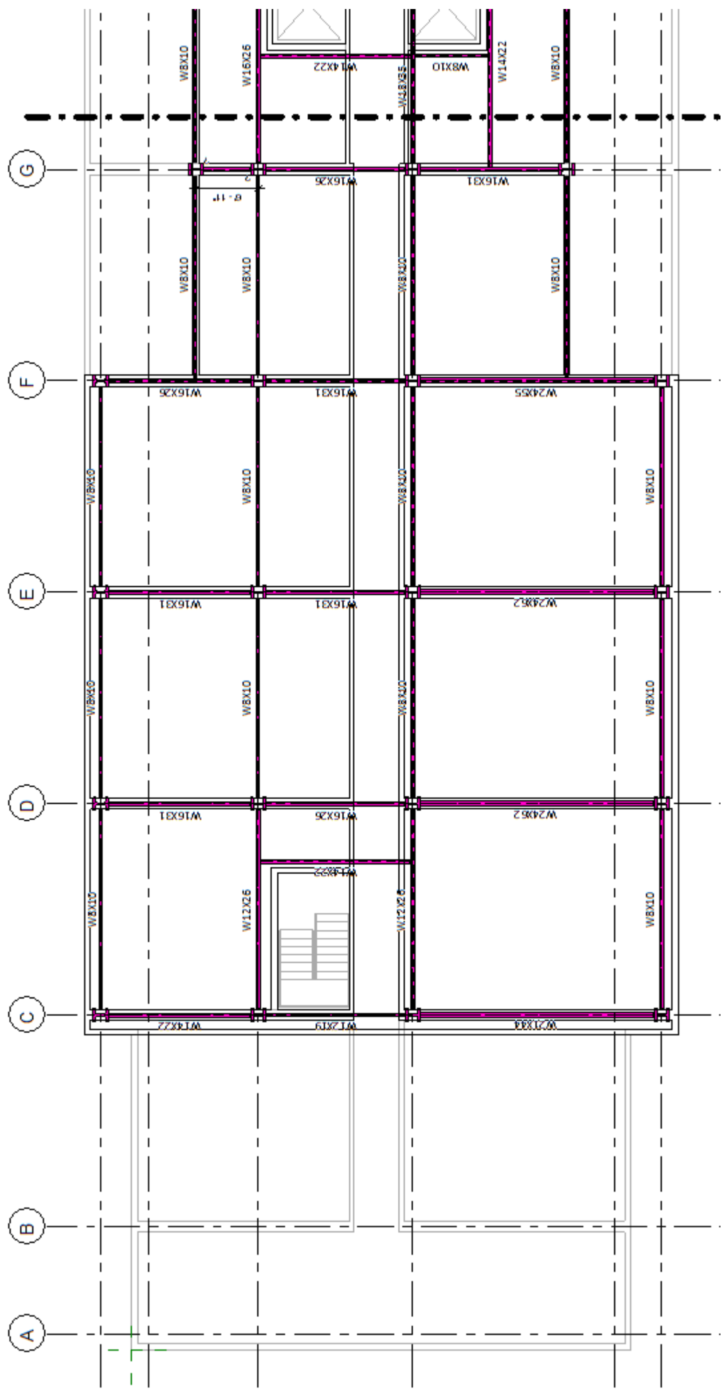


Figure F.4: Typical Framing Plan for Levels 29 – 32 of Proposed Design
Note: Match line to Figure F.3

Appendix G – References

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