40 Gold Street Residential Building

New York, New York

TECHNICAL REPORT 1

Jesse Cooper – Structural Option
Thesis Consultant: Dr. Boothby
Date of Submission: October 5th, 2009

Table of Contents

Diagram Reference System

Tables: T- #

Structural Diagrams: S - #

Figures/Pictures: F - #

Executive Summary

The Structural Concepts and Structural existing Conditions Report provides an in depth explanation of the structural system present in the 40 Gold Street Building. 40 Gold Street is a slender 14 story residential building with 21'6" high ground level retail spaces. The building features two large rooftop terraces and a spacious penthouse space enclosed by a window wall system. The 60,000 SF structure is located in Manhattan and is located in a very crowded area in which surrounding structures and poor site conditions greatly influenced the structural design.

The gravity resisting system is a basic steel frame system composed of primarily W10 and W12 members. The floor system is a 2" – 18 gage metal decks with 2 ½" light weight concrete that runs one way. The bay sizes are kept to a minimum with an average size of approximately 15'8" x 14'0". The columns are spliced at 2'6" above floor levels and span two floors amounting to 21'6" in height.

The floor framing is a uniform grid like layout of W10 and W12 beams. The floor framing layout is typical throughout almost all floors aside from a few minor variations. Beams of larger size can be found throughout the building including several cantilever beams that project outward from the first floor to support the 13 residential stories above. The foundation system consists of a combination of 25' and 35' micro piles. Several of the footings are offset from the optimal locations to avoid disturbing existing foundations. To resist lateral loads, the structure features 5 braced frames and 4 moment frames. The moment frames are skewed due to the offset footings referred to above.

Calculations were performed according to ASCE7-05 and IBC 2006 to obtain the lateral and gravity loads on 40 Gold Street's Structure. These loads include dead loads, live loads, snow loads, seismic, and wind loads. The building loads are compared to the actual building loads used by Severud Associates who performed calculations according to NYC Building Code and ASCE7-02. According to calculations, the controlling lateral condition is the East/West wind forces. The wind forces in this direction create story shears as large as 139 kips and an overturning moment of 13,392 ft-kips. The seismic forces were minimal due to the light weight of the building and lack of weight near the top of the structure. The building Shear value due to seismic conditions was determined to be only 46.8 Kips.

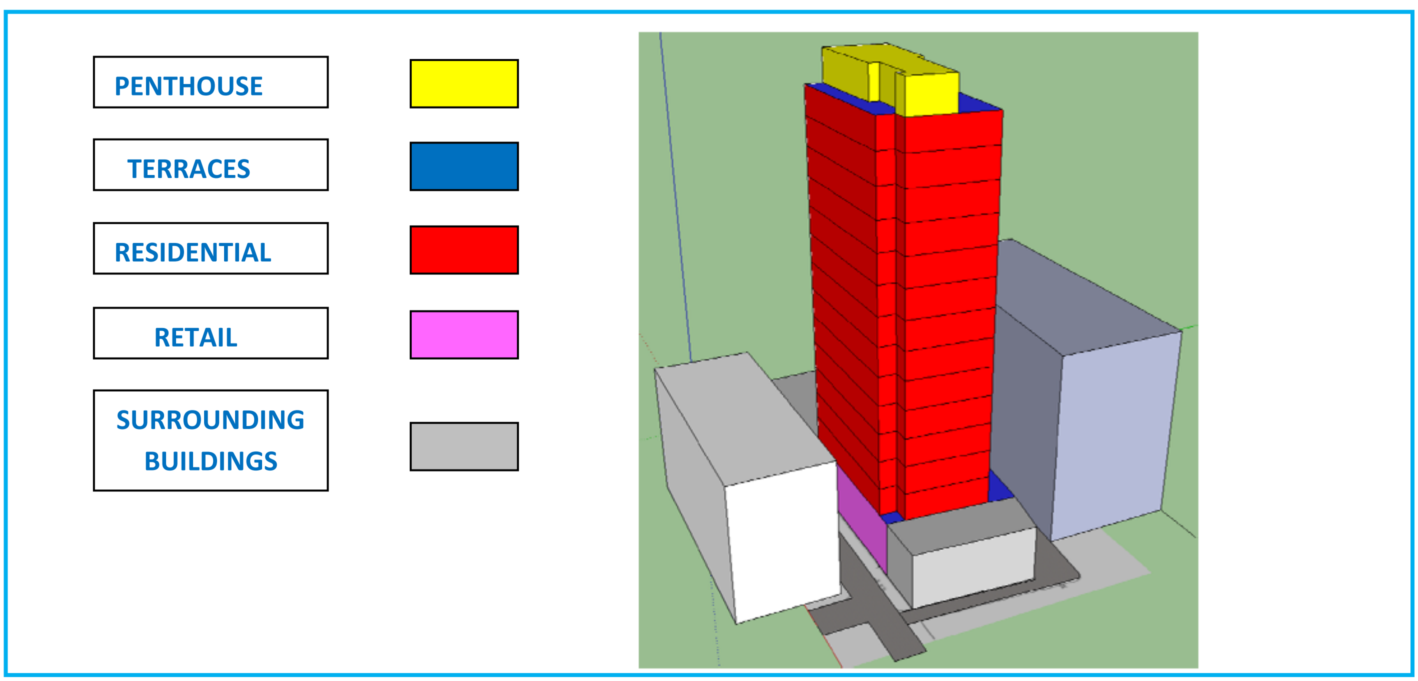
Using the calculated building loads, two spot checks were performed to both verify actual design sizes and to provide a better understanding of the structural system. The spot checks include the analysis of a typical interior beam located on the first floor and a typical exterior beam located on the second floor. Calculation results verified the design sizes.

Introduction

40 Gold Street is an impressive architectural package that offers retail and residential space in lower Manhattan, which is one of the fastest growing residential sections of New York City. The construction of 40 Gold Street began in March 2009 and will conclude in January 2010. The building replaces an old two story brick building and is nestled tightly between two existing structures, a narrow alley (Eden's Alley), and Gold Street. The constricted area presented special restrictions and challenges that greatly affected the final design and construction process.

Standing 175' above grade, the 40 Gold Street Building is a 14 story structure comprised of 5,900 square feet of retail space and 62,000 Square feet of residential space. The lowest two floors are primarily dedicated to retail space and serve as a podium on which a sleek 14 story residential tower rests. The lowest floor, referred to as the cellar, is below grade and functions as extra retail space as well as housing space for mechanical and electrical equipment. Retail spaces are appropriately located at the ground level and are highlighted with traditional floor to ceiling storefront windows to attract customers from the nearby streets and sidewalks. The storefront glazing is complimented very nicely with a pre fabricated assembly of dark stone cladding and a large bronze plaque that boldly recognizes the building as 40 Gold Street. In addition to retail space, there is a residential lobby and mailroom.

The residential tower is comprised of 12 residential floors. Identical in layout, floors 2-9 are comprised of 2 studio apartments and 3 2 bedroom apartments that all encompass the vertical circulation node located at the core of the tower. Two elevators and a stairwell serve as the buildings vertical circulation. Floors 10-13 are identical as well, but have 4 2-bedroom apartments and no studio apartments. At the top of the building, a level referred to as the penthouse provides the building's residents with two spacious recreational terraces sheltered by a gold painted metal trellis, a large recreational room enclosed by a neat window wall system, a kitchenette, a laundry room, and bathrooms.



F -1

Introduction Continued

The trapezoidal shape of the building closely reflects the shape of the site, which is to be expected when working with such a constricted space. The interior spaces are laid out in a very rectangular manner, and the exterior shell is also very rectangular. The residential tower boasts a sleek modern appearance with metal exterior cladding and gold toned trespa paneling.

Overall, the final design solution created by Architects Meltzer/Mandl and Structural Engineers Severud Associates yields an impressive final product that makes the most of a small site, and is certainly playing a major role in the successful rebuilding of Lower Manhattan.

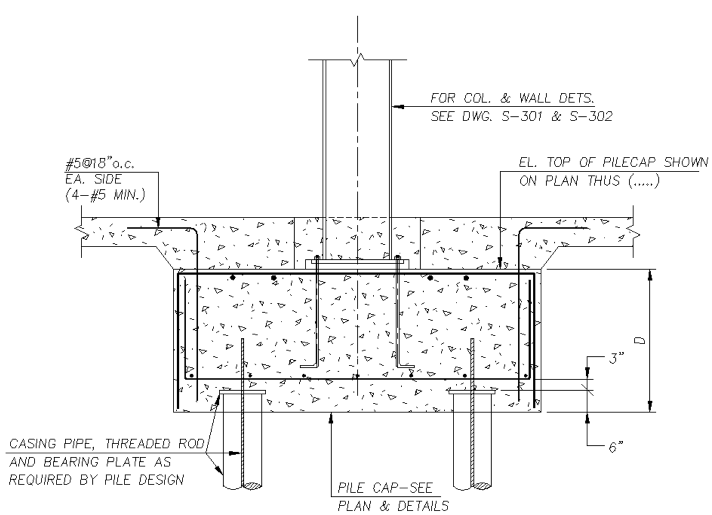
Structural System Overview

Foundations

The site excavation and foundation work required a great deal of design work and creative planning relative to the average building project. As mentioned in the introduction, the site is a very constricted space with two existing structures up against the property line, and there are two streets (Eden's Alley and Gold Street) with great proximity. During excavation and foundation work, the adjacent streets required bracing and shoring for temporary and long term support. In addition, a major foundation design goal was to circumvent the need to do underpinning of the adjacent existing structures. As a result, the depth of the various foundation components is not consistent throughout the site but varies based on location relative to the surrounding structures and existing foundation systems.

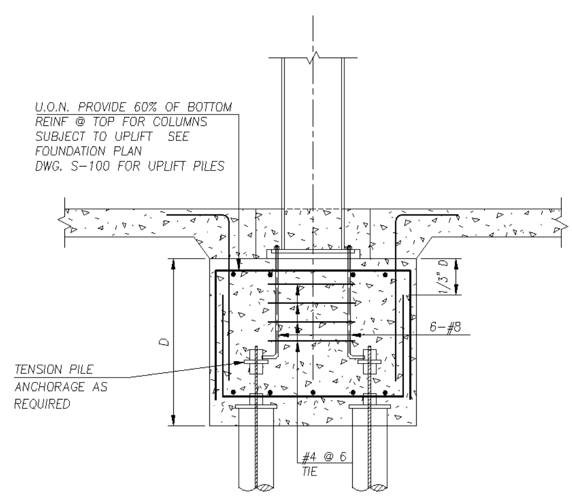
The foundation employs a system of 101 strategically positioned micro piles. There are Eighty Eight 75 Ton compression capacity piles that are 35' long and Thirteen 35 Ton compression capacity piles that are 25' long. Various pile caps are used to transfer the building loads down to the piles and generally range from 36"-39" in depth.

The cellar floor system is an 8" slab on grade with #5 bars @ 12" O.C. top/bottom running both directions. Resting on 6" of crushed stone, the slab on grade is attached to the pile caps via an assortment of connections. As seen in figure **S-1**, the typical pile cap is anchored to the column base plates by 6-#8 bars, and the pile caps are directly anchored to the floor slab by #5 @ 18" on each side of the column (minimum of 4 - #5 required per side). The pile caps subjected to uplift require tension pile anchorage as seen by figure **S-2**.



TYPICAL PILE CAP DETAIL

S-1



TYPICAL PILE CAP SECTION
FOR UPLIFT PILES
(MARKED THUS: * ON PLAN).

S-2

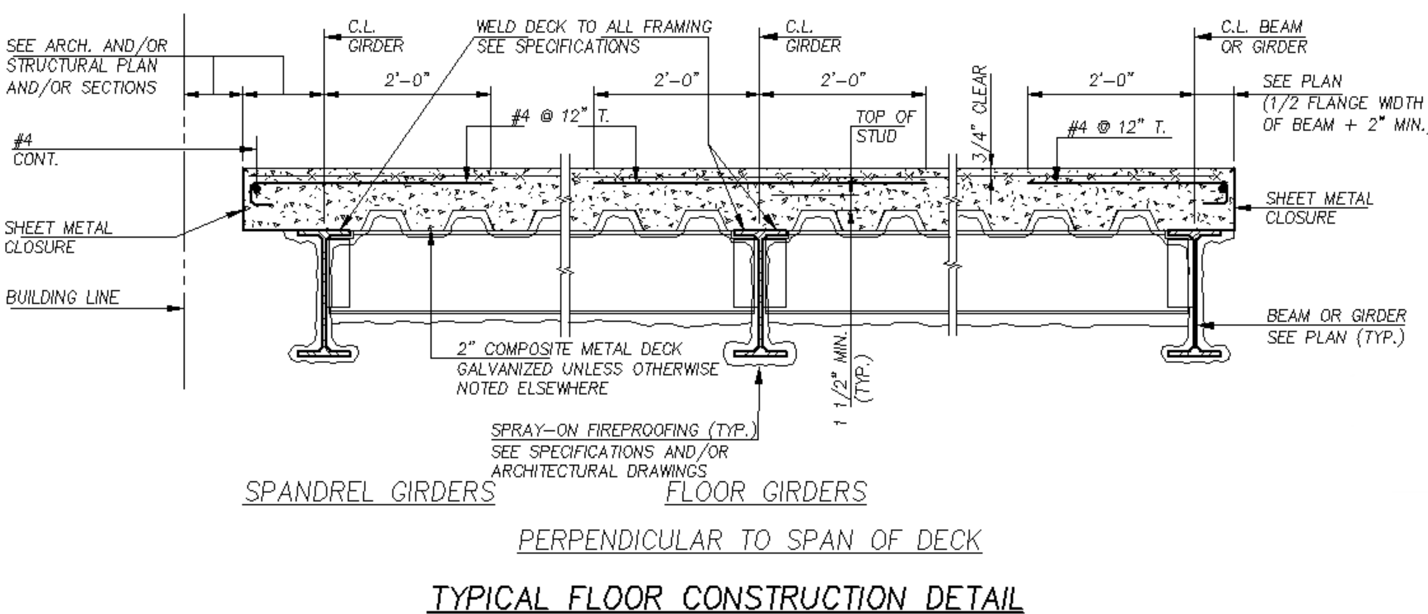
Floor System

The floor system employed in the 40 Gold Street building design is primarily slab on metal decking. Aside from the cellar floor system, the floor system is a 2" – 18 gage metal decks with 2 ½" light weight concrete topping as shown below in figure S-3. This one-way floor system operates to transfer gravity loads down to the supporting beams, girders, and columns.

The floor slab is reinforced with #4 @12" T., and 6x6 / W3 X W3 welded wire fabric is used with a ¾" clearance from top of slab. All concrete used has 4000 psi design strength. In several cases throughout the building, masonry partitions rest directly on the floor system. The areas where the partitions run parallel to the deck span, 2 - #6 bars are required to run on each side of the wall the full length of the wall to the first support beyond each end of the wall. Also, for the situation where the masonry partitions run perpendicular to the deck span, # 4 reinforcement bars run the full extent of the wall in each flute of the metal deck floor system.

The concrete is attached to the metal decking by way of equally spaced shear connectors. The shear studs extend a minimum of 1 ½" above the top of the metal decking. For the most part, the floor system throughout the building requires ¾" headed shear connectors @ 1' 0" or less.

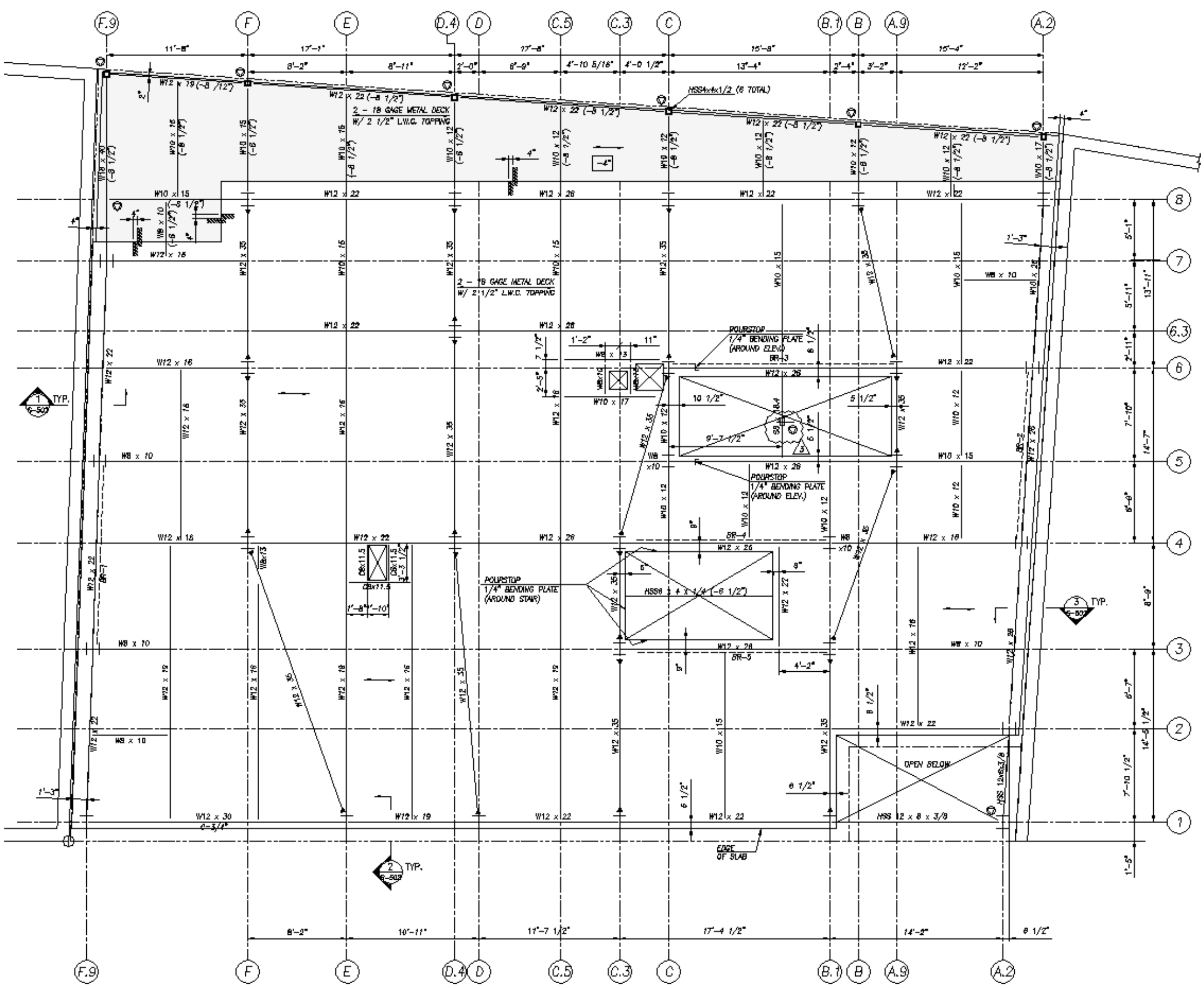
The cellar floor consists of a two-way 8" slab on grade with #5 @ 12" on center, top and bottom each way. The cellar slab rests on a 6" layer of crushed stone. More importantly, the cellar floor which is sub grade required a change in elevation as a consequence of closely surrounding structures and foundations. At the exterior sections of the cellar floor, the slab is raised up relative to the adjacent existing foundation. A slab depression of approximately 8'0" exists, allowing the center part of the cellar floor to rest much lower below grade.



S-3

Floor Framing

The floor system rests on uniform grid like layout of W-shape beams and girders. As seen below in figure S-4, there are only a few irregularities, in which beams do not run directly top to bottom across the plan. If you take notice, these beams are designed with moment connections, and serve as a part of lateral resisting moment frames. Figure S-4 represents the floor framing at level 2, and this same general layout is repeated throughout the rest of the building. Although the bay sizes vary, the average bay size is approximately 15' 8" x 14' 0".



S-4

Level 2 Framing Layout

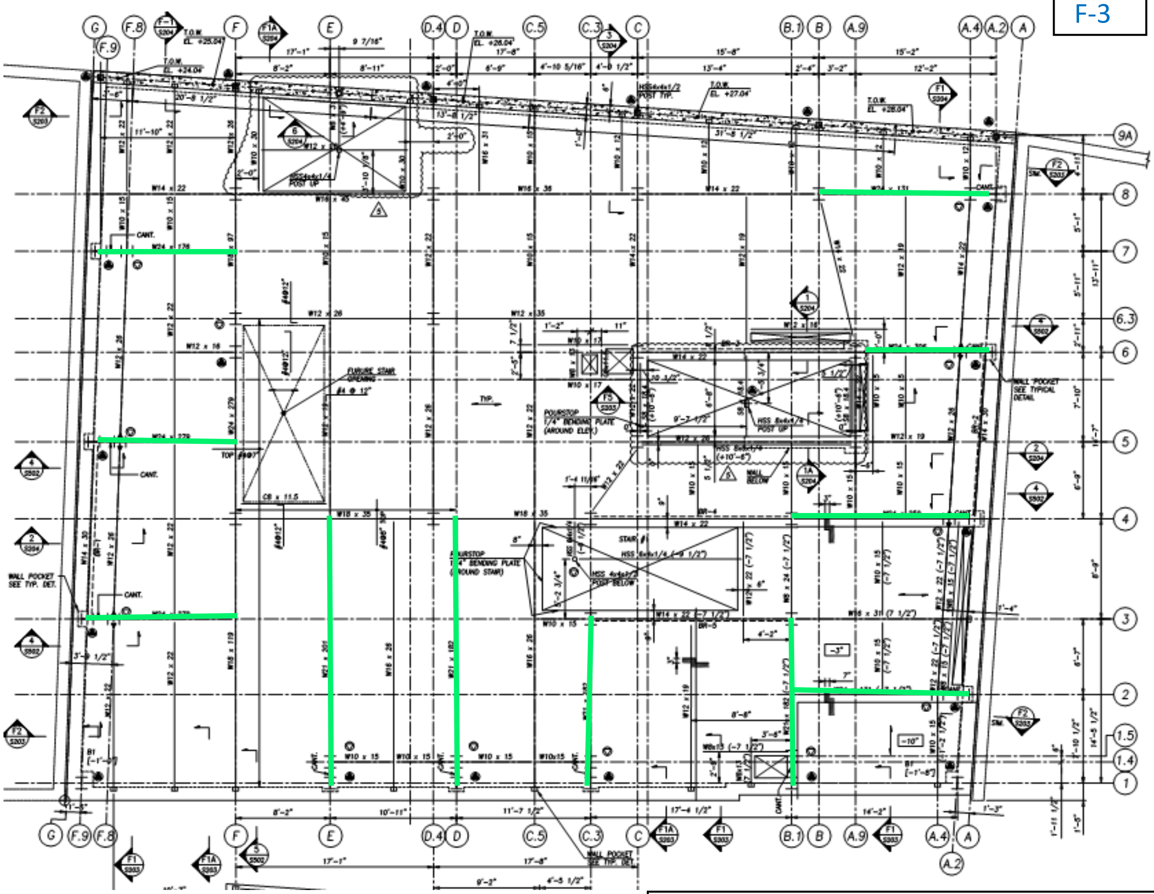
Gravity System

The gravity loads are resisted by a relatively rudimentary steel frame system. Figures **F-2** and **F-3** provide a close up look at the unfinished steel frame structure. The majority of the vertical structural elements are W-shapes aside from a few HSS4/4/3/8. The column sizes are pretty much constant from level to level, but a slight reduction in size is observed near the top of the structure. The steel frame not only resists the gravity loads transferred from the floor system, but also supports the entire exterior envelope. The beams and girders are all W-shapes and are all treated with spray on fireproofing. The beams and girders range from W10's to W14's; however, at the second level several beams project 2 feet outward and behave as cantilevers to support the 13 stories above. Each cantilever is highlighted in figure S-5. These members are as big as W24X279's. The column splices are all located at 2' -6" above each finished floor. Almost all columns span two floors.

Figures F-2 and F-3: 40 Gold Street under construction

F-2

F-3

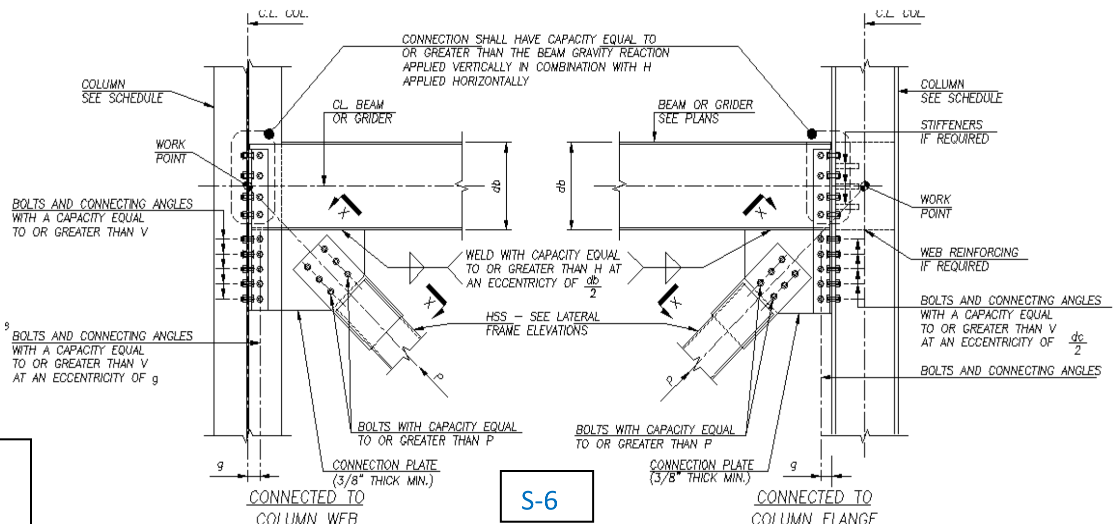


S-5

Highlighted Beams Cantilever outward 2 feet

Lateral System

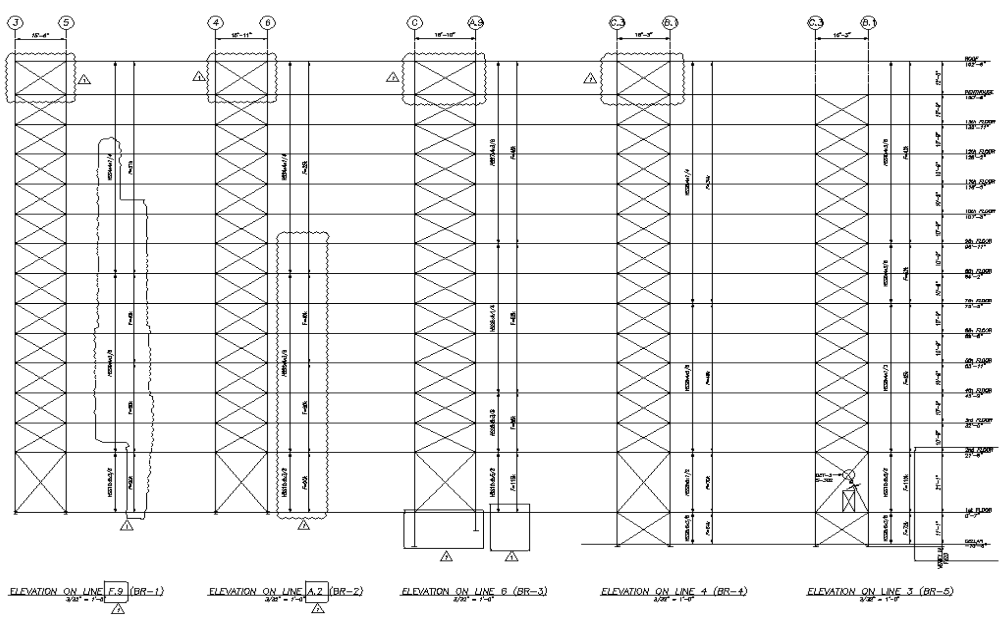
The lateral system of 40 Gold Street consists of 5 braced frames and 4 moment frames. Figure **S-8** shows the moment frames in red which span east to west across the building. The braced frames are shown in yellow. The moment frames are skewed since several of the building's footings are offset to avoid agitating the adjacent structures. The moment frame along column line A.9 is skewed due to architectural constraints. Figure **S-6** illustrates the typical connections and structural members that form the braced frames, and figure **S-7** provides an elevation view of the braced frames spanning from the foundation up to the roof level. The cross brace elements that form the braced frames are HSS shapes. For the most part, the lateral system is laid out symmetrically. In addition, the building's shape and weight distribution is symmetrical. As a result, assuming the rigidity of each lateral resisting frame is not too different; the center of rigidity is located near the center of mass. In consequence, the potential for torsion effect due to seismic load is minimal.



S-6

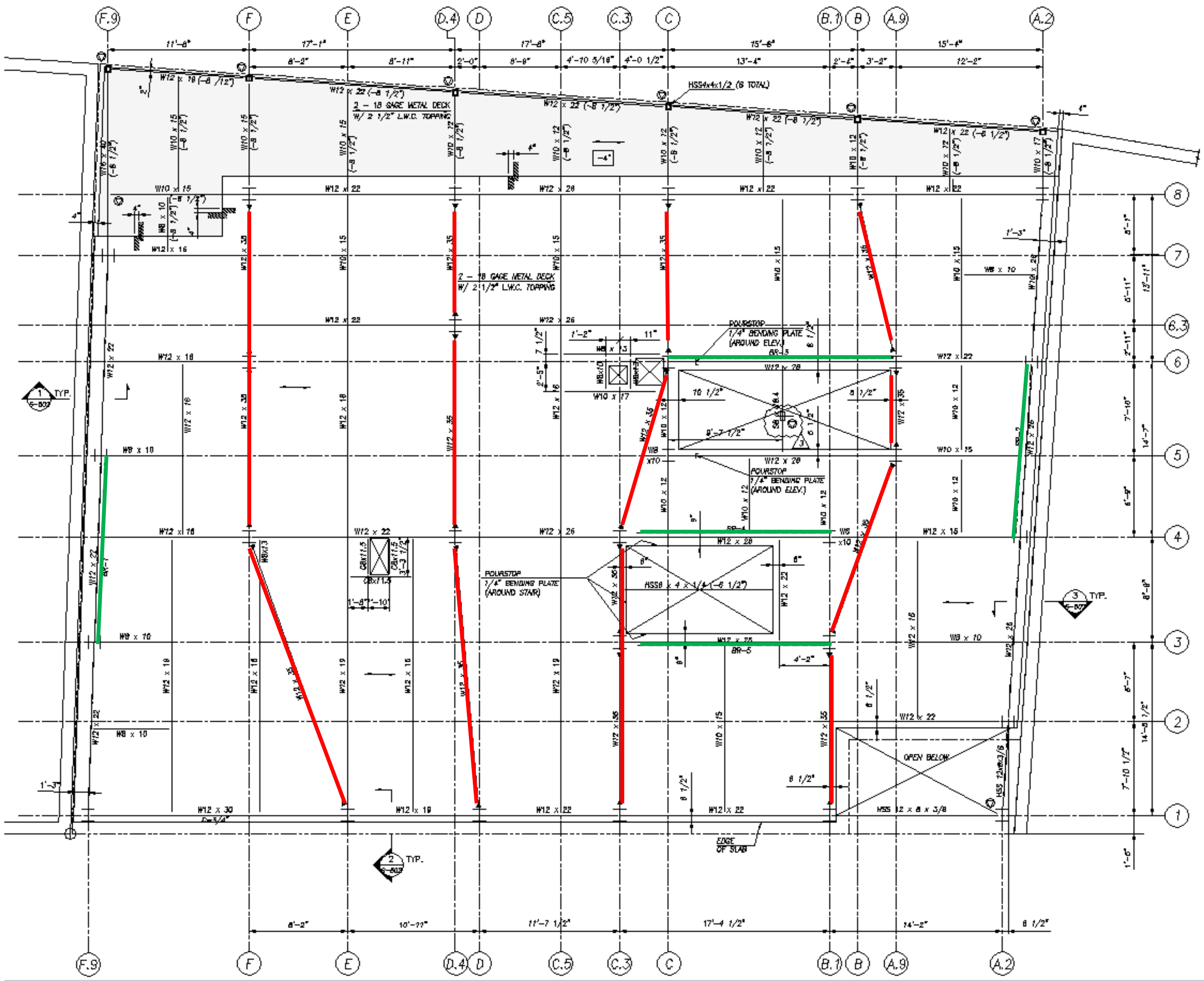
Figure S-6 details the typical brace frame.

Figure S-7 shows the five braced frames in elevation view.



S-7

LATERAL SYSTEM LAYOUT



S-8

Moment Frames

Braced Frames

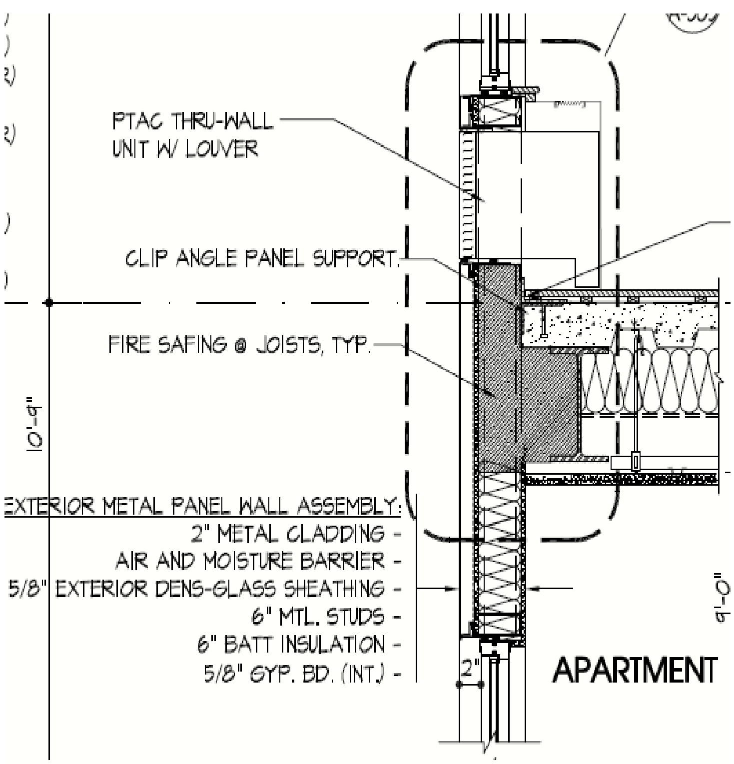
Sustainability

Although the overall design wasn't driven by sustainability, the 40 Gold Street building includes several green features throughout the design. The apartments are equipped with energy star appliances. In addition, the windows are assembled with low-emissive glass. The roofing materials are designed to prevent or minimize the heat island effect, and the building envelope is highly proficient for thermal and moisture protection. The exterior façade also has an 8" metal fin projecting out from above each of residential windows, which serves as a shading device.

Building Envelope

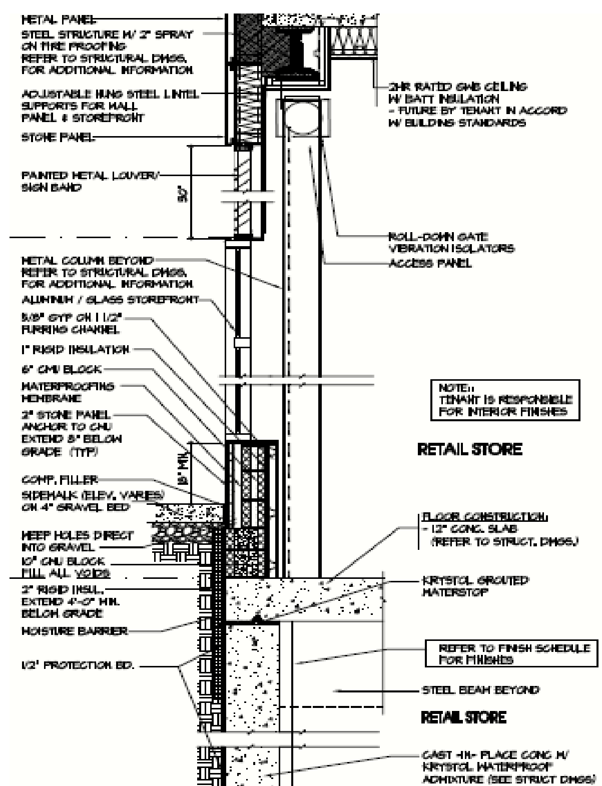
Floors 2-14 are enclosed by a basic non-bearing exterior metal panel wall assembly. The general composition of the wall shown in figure S-9 is 2" metal cladding (exterior), air and moisture barrier, 5/8" exterior dens-glass sheathing, 6" metal studs, 6" batting insulation, and 5/8" gypsum board (interior).

The sub grade spaces, also referred to as the cellar, are enclosed by a cast-in-place concrete wall with Krystol waterproof admixture. A detail of the enclosure can be seen in Figure S-10. Retail areas on the street level are enclosed by a large aluminum and glass storefront anchored to a basic CMU block wall assembly which consists of 2" stone panel (exterior), waterproofing membrane, 6" CMU block, 1" rigid insulation, 5/8" gypsum on 1 ½" furring channel (interior). The storefronts are also equipped with a roll-down gate for security purposes.



S-9

Typical Building Envelope for Residential Tower



S-10

Typical Building Envelope for Ground Floor

Roof System

40 Gold street features an ordinary flat roof comprised primarily of W12x22 and W12x30 beams supporting the typical 2” – 18 gage metal decks with 2 ½” light weight concrete topping. Mechanical equipment is located on the roof and C channels are used for additional support. The roof terraces feature a slight different assembly. The terraces features the Inverted Roof Membrane Assembly (IRMA) that works in conjunction with 2’x2’ Concrete Pavers on pedestals. The insulation layer is an extruded polystyrene layer placed over the roofing membrane.

Codes, Design Standards and References:

- Original Design:

Building Code

New York City Building Code

Lateral Loads

Seismic: New York City Building Code

Wind: American Society of Civil Engineers (ASCE), ASCE7-02

Design Load and Standards

New York City Building Codes

- Thesis Design:

Building Code

American Society of Civil Engineers (ASCE), ASCE7-05
International Building Code (IBC) 2006

Lateral Loads

American Society of Civil Engineers (ASCE), ASCE7-05
International Building Code (IBC) 2006

Design Code References

Steel Construction Manual 13th edition, American Institute of Steel Construction
ACI 318-05, Building Code Requirements for Structural Concrete, American Concrete Institute

Material Summary

Structural Steel

- Rolled Shapes……………………………………………..............................ASTM A572, Grade 50
    - W-Shapes, HSS Shapes, C Shapes
- Connection Materials…………………………..…………………………………..ASTM A36
    - Angles, Plates, etc.

Metal Decking

- Painted 2" – 18 Gage Metal Deck…………………………………………….....ASTM A611, Grade C, Fy = 40 ksi
    - Composite Deck
- Galvanized 2" – 18 Gage Metal Deck……………………………………….ASTM A446, Grade A, Fy = 40 ksi

Headed Shear Studs

- 3/4" diameter minimum

Welding Electrodes

- E-70XX……………………………………………………………………………………….70 ksi Tensile Strength
    - All Butt Welds are 100% Penetration Welds
    - Fillet Welds are ¼" minimum

High Strength Bolts

- 3/4" diameter

Grout

- Grout for Base Plates and Anchor Bolts……………………………………7,500 psi Minimum

Cast-in-place Concrete

- Foundations…………………………………………………………………………….4000 psi
    - N.W. Concrete with water/cement ratio < 0.39
- Floor Slab on deck……………………………………………………………………..4000 psi
    - L.W. Concrete
- Roof……………………………………………………………………………………………..4000 psi
    - L.W. Concrete
- All Other……………………………………………………………………………………….4000 psi
    - N.W. Concrete

Reinforcement:

- Bar Reinforcing…………………………………………………………………………ASTM A615, Grade 60
    - Deformed bars of New Billet Steel
- Weld Wire Fabric………………………………………………………………………ASTM A82 and A185

Required Loads

Building Dead Loads were provided by the Structural Engineering Firm Severud Associates. The loads were verified and used for calculations.

DEAD LOADS		
Floor Level	**Building Component (Location)**	**Design Dead Load**
Ground Floor	Slab	34 psf
	Steel	4 psf
	Ceiling / Mechanical Equip.	8 psf
	Partitions	12 psf
	Miscellaneous Dead Load (Lobby)	38 psf
	Miscellaneous Dead Load (Retail)	20 psf
2nd Floor	Slab	34 psf
	Steel	4 psf
	Ceiling / Mechanical Equip.	3 psf
	Partitions (residential areas)	12 psf
	Miscellaneous Dead Load (Roof Terrace)	30 psf
3rd - 9th Floor	Slab	34 psf
	Steel	4 psf
	Ceiling / Mechanical Equip.	3 psf
	Partitions (residential)	12 psf
10th - 13th Floor	Slab	34 psf
	Steel	4 psf
	Ceiling / Mechanical Equipment	3 psf
	Partitions (residential)	12 psf
Penthouse	Slab	34 psf
	Steel	4 psf
	Ceiling / Mechanical Equip. (terrace)	3 psf
	Ceiling / Mechanical Equip. (Mechanical Area)	8 psf
	Ceiling / Mechanical Equip. (Recreational Area)	8 psf
	Miscellaneous Dead Load (Roof Terrace)	30 psf
	Miscellaneous Dead Load (Mechanical Area)	15 psf
Roof	Slab	25 psf
	Steel	4 psf
	Ceiling/Mechanical Equip.	8 psf
	Miscellaneous Dead Load (Roof Terrace)	10 psf
Bulkhead	Slab	34 psf
	Steel	4 psf
	Ceiling/Mechanical Equip.	8 psf
	Miscellaneous Dead Load (Roof)	25 psf

T-1

Building live loads were determined by referencing ASCE7-05. The Actual Design Loads Were The Same.

Live Loads			
Area	**Actual Design Load**	**Thesis Design Load (ASCE 7-(**	**Code/Table**
Lobby / Open Plan	40 psf (kN/m^2)	40 psf (kN/m^2)	ASCE7-05 Table 4-1
Retail	100 psf (kN/m2)	100 psf (kN/m2)	
Corridors	100 psf (kN/m2)	100 psf (kN/m2)	
Roof	60 psf (kN/m2)	60 psf (kN/m2)	
Terraces/Pedestrian	100 psf (kN/m2)	100 psf (kN/m2)	
Residential	40 psf (kN/m2)	40 psf (kN/m2)	

T-2

SNOW LOADS

- Location: New York, New York
- P_g = 30 PSF
- Terrain Category = B
- Exposure Factor = C_e = 1.0
- Thermal Factor = C_t = 1.0
- Occupancy Category = II
- Importance Factor = I = 1.0

$P_f = 0.7 * C_e * C_t * I * P_g$

$P_f = 0.7 * 1 * 1 * 1 * (30 \text{ PSF})$

$\boxed{P_f = 21 \text{ PSF}}$

ANALYSIS & RESPONSE

Wind Load Summary

The actual wind loads calculated for the 40 Gold Street Design were done according to ASCE7-02. For the Thesis calculations, wind load pressures were obtained by following Method 2 for the main wind-force resisting system for enclosed buildings and referencing the IBC 2006 1609.1.1 and Chapter 6 of ASCE/SEI 7-05 (ASCE7). These wind calculations are summarized in the following tables and diagrams; however, the calculations can be viewed in their entirety in Appendix A. For comparison purposes, the actual design wind pressures used by Severud Associates can be observed in figure **F-6**.

Summary of Wind Calculations: Variables and Classifications (EAST/WEST DIRECTION)					
Basic Wind Speed (V)	110 mph	**damping ratio (b)**	1.5	**Qp**	29.47
Wind Directionality Factor (Kd)	0.85	**natural frequency (n)**	0.363	**GCpn (windward)**	1.5
Importance Factor (I)	1	**Z**	102.4	**GCpn(Leeward)**	-1
Exposure Category	B	**Iz**	0.4968	**Pp(windward)**	44.207
Topographic Factor (Kzt)	1	**Lz**	466.74	**Pp(leeward)**	-29.47
Alpha	7	**Q**	0.838	**Cp (windward)**	0.8
Zg	1200	**Vz**	96.357	**Cp(leeward)**	-0.5
a	0.50	**N1**	1.7583	**Cp(side walls)**	-0.7
b	0.84	**Rn**	0.0962	**Gcpi**	0.18 / -0.18
c	0.3	**Rb**	0.4837	**Mean Roof Height**	170' 8"
l	320	**Rl**	0.2575	**Enclosure Type**	Fully Enclosed
e	0.3333	**R**	0.09226	**Rigidity**	Flexible
Zmin	30	**Gr**	3.788	**Parapet**	4' high parapet
alpha	0.25	**Gf**	0.8845	**Topography**	No Hill / No Escarpment

T-3

T-4

Summary of Wind Calculations: Variables and Classifications (NORTH/SOUTH DIRECTION)					
Basic Wind Speed (V)	110 mph	**damping ratio (b)**	1.5	**Qp**	0
Wind Directionality Factor (Kd)	0.85	**natural frequency (n)**	0.363	**GCpn (windward)**	1.5
Importance Factor (I)	1	**Z**	102.4	**GCpn(Leeward)**	-1
Exposure Category	B	**Iz**	0.4968	**Pp(windward)**	0
Topographic Factor (Kzt)	1	**Lz**	466.74	**Pp(leeward)**	0
Alpha	7	**Q**	0.8449	**Cp (windward)**	0.8
Zg	1200	**Vz**	96.357	**Cp(leeward)**	-0.42
a	0.50	**N1**	1.7583	**Cp(side walls)**	-0.7
b	0.84	**Rn**	0.0962	**Gcpi**	0.18 / -0.18
c	0.3	**Rb**	0.2605	**Mean Roof Height**	170' 8"
l	320	**Rl**	0.1961	**Enclosure Type**	Fully Enclosed
e	0.3333	**R**	0.07179	**Rigidity**	Flexible
Zmin	30	**Gr**	3.788	**Parapet**	No Parapet
alpha	0.25	**Gf**	0.8877	**Topography**	No Hill / No Escarpment

***Note:** The highlighted cells represent values in the North/South Wind Pressure Calculations that differ from the East/West Wind Pressure Calculations. These differences were due to the changes in building dimensions.

Wind Design Load Tables

(WIND PRESSURES)

Calculated Wind Pressures for the EAST / WEST Direction													
B = 78' 2-1/2"			L = 56' 9-1/2"			$GC_{pi(windward)}$ = 0.18				$GC_{pi(Leeward)}$ = -0.18			
Story	Story Height	Height	k_z	k_{zt}	k_d	V	I	$q_{z\,(psf)}$	G_f	C_{pw}	C_{pL}	$p_{z\,(windward)}$	$p_{z\,(Leeward)}$
2	21' 8"	21'8"	0.7	1.00	0.85	110	1.00	18.43	0.8845	0.8	-0.5	7.587	-7.9476
3	10' 9"	32'5"	0.7145	1.00	0.85	110	1.00	18.81	0.8845	0.8	-0.5	7.857	-7.9476
4	10' 9"	43'2"	0.7158	1.00	0.85	110	1.00	18.85	0.8845	0.8	-0.5	7.881	-7.9476
5	10' 9"	53'11"	0.8256	1.00	0.85	110	1.00	21.74	0.8845	0.8	-0.5	9.927	-7.9476
6	10' 9"	64'8"	0.8687	1.00	0.85	110	1.00	22.87	0.8845	0.8	-0.5	10.730	-7.9476
7	10' 9"	75'5"	0.9117	1.00	0.85	110	1.00	24.00	0.8845	0.8	-0.5	11.531	-7.9476
8	10' 9"	86'2"	0.9485	1.00	0.85	110	1.00	24.97	0.8845	0.8	-0.5	12.216	-7.9476
9	10' 9"	96'11"	0.98075	1.00	0.85	110	1.00	25.82	0.8845	0.8	-0.5	12.817	-7.9476
10	10' 9"	107'8"	1.009	1.00	0.85	110	1.00	26.57	0.8845	0.8	-0.5	13.344	-7.9476
11	10' 9"	118'5"	1.036	1.00	0.85	110	1.00	27.28	0.8845	0.8	-0.5	13.847	-7.9476
12	10' 9"	129'2"	1.063	1.00	0.85	110	1.00	27.99	0.8845	0.8	-0.5	14.350	-7.9476
13	10' 9"	139'11"	1.089	1.00	0.85	110	1.00	28.67	0.8845	0.8	-0.5	14.834	-7.9476
Penthouse	10' 9"	150'8"	1.111	1.00	0.85	110	1.00	29.25	0.8845	0.8	-0.5	15.244	-7.9476
Roof	12' 0"	162'8"	1.135	1.00	0.85	110	1.00	29.88	0.8845	0.8	-0.5	15.691	-7.9476
Bulkhead Roof	8' 0"	170'8"	1.151	1.00	0.85	110	1.00	30.31	0.8845	0.8	-0.5	15.989	-7.9476

T-5

Calculated Wind Pressures for the North / South Direction													
B = 56' 9-1/2"			L = 78' 9-1/2"			$GC_{pi(windward)}$ = 0.18				$GC_{pi(Leeward)}$ = -0.18			
Story	Story Height	Height	k_z	k_{zt}	k_d	V	I	$q_{z\,(psf)}$	G_f	C_{pw}	C_{pL}	$p_{z\,(windward)}$	$p_{z\,(Leeward)}$
2	21' 8"	21'8"	0.7	1.00	0.85	110	1.00	18.43	0.8877	0.8	-0.42	7.634	-5.84391
3	10' 9"	32'5"	0.7145	1.00	0.85	110	1.00	18.81	0.8877	0.8	-0.42	7.905	-5.84391
4	10' 9"	43'2"	0.7158	1.00	0.85	110	1.00	18.85	0.8877	0.8	-0.42	7.929	-5.84391
5	10' 9"	53'11"	0.8256	1.00	0.85	110	1.00	21.74	0.8877	0.8	-0.42	9.982	-5.84391
6	10' 9"	64'8"	0.8687	1.00	0.85	110	1.00	22.87	0.8877	0.8	-0.42	10.788	-5.84391
7	10' 9"	75'5"	0.9117	1.00	0.85	110	1.00	24.00	0.8877	0.8	-0.42	11.592	-5.84391
8	10' 9"	86'2"	0.9485	1.00	0.85	110	1.00	24.97	0.8877	0.8	-0.42	12.280	-5.84391
9	10' 9"	96'11"	0.98075	1.00	0.85	110	1.00	25.82	0.8877	0.8	-0.42	12.883	-5.84391
10	10' 9"	107'8"	1.009	1.00	0.85	110	1.00	26.57	0.8877	0.8	-0.42	13.412	-5.84391
11	10' 9"	118'5"	1.036	1.00	0.85	110	1.00	27.28	0.8877	0.8	-0.42	13.916	-5.84391
12	10' 9"	129'2"	1.063	1.00	0.85	110	1.00	27.99	0.8877	0.8	-0.42	14.421	-5.84391
13	10' 9"	139'11"	1.089	1.00	0.85	110	1.00	28.67	0.8877	0.8	-0.42	14.907	-5.84391
Penthouse	10' 9"	150'8"	1.111	1.00	0.85	110	1.00	29.25	0.8877	0.8	-0.42	15.319	-5.84391
Roof	12' 0"	162'8"	1.135	1.00	0.85	110	1.00	29.88	0.8877	0.8	-0.42	15.768	-5.84391
Bulkhead Roof	8' 0"	170'8"	1.151	1.00	0.85	110	1.00	30.31	0.8877	0.8	-0.42	16.067	-5.84391

T-6

P_z Values Calculated Using Equations

Windward $P_z = q_z G_f C_p - q_h(GC_{pi})$

Leeward $P_z = q_h G_f C_p - q_h(GC_{pi})$

See Appendix A For Wind Calculations

Table T-5 Is Illustrated in Diagram F-4

Table T-6 Is Illustrated in Diagram F-5

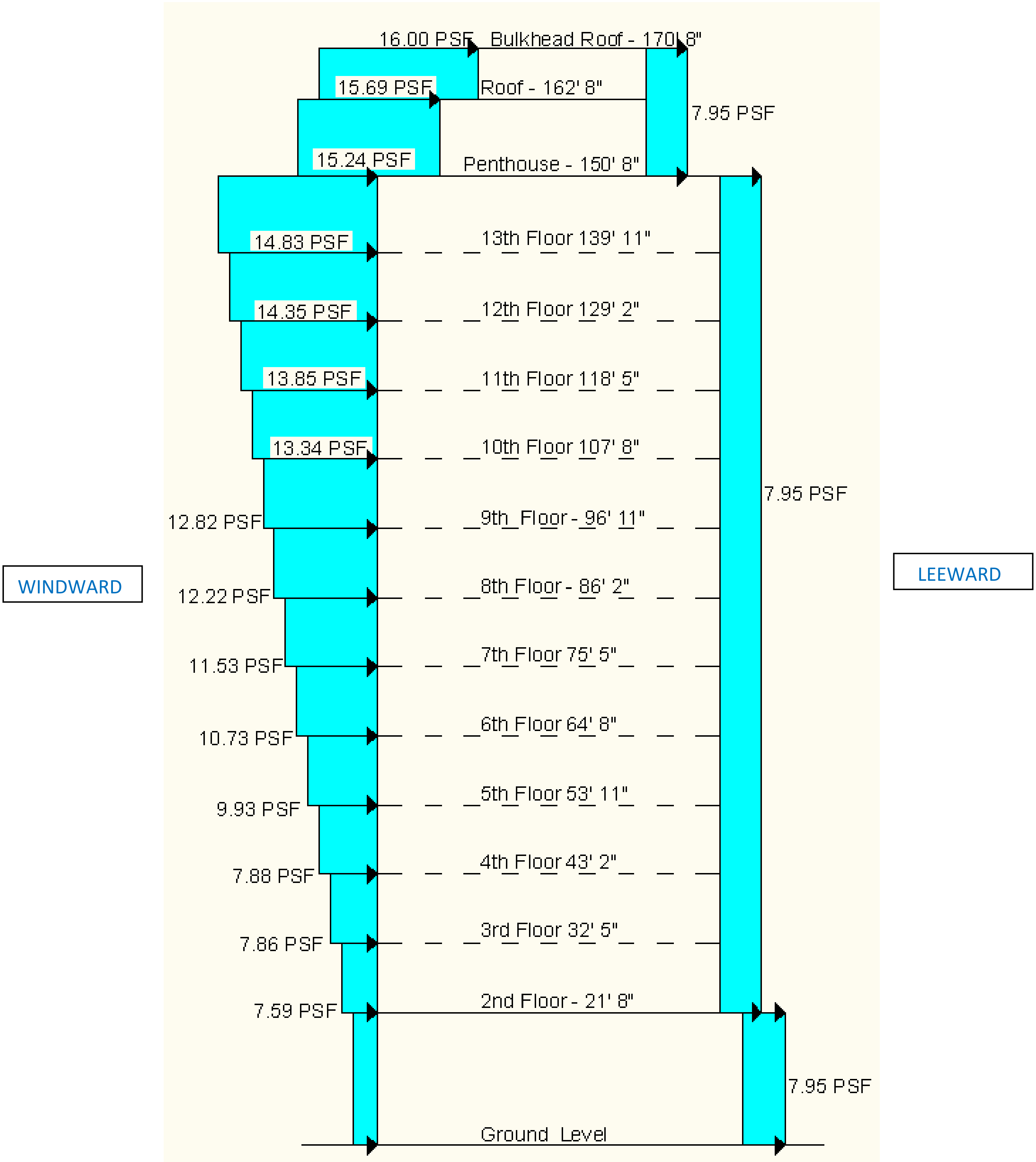
WIND CALCULATIONS: STORY SHEARS AND OVERTURNING MOMENT

WIND FORCES, STORY SHEARS, OVERTURNING MOMENT										
EAST/WEST DIRECTION										
story	story height	height from ground	Tributary Area Below	Tributary Area Above	Pz below	Pz Above	Fx	Vx	Moment Contribution Fx*(height)	Overturing Moment (Kip*ft)
	ft	ft	SF	SF	PSF	PSF	kips	kips		
2	21.667	21.667	845	419	7.587	7.857	9.703	139.328	210.2370244	
3	10.75	32.417	419	419	7.857	7.881	6.594	129.625	213.7648946	
4	10.75	43.167	419	419	7.881	9.927	7.462	123.031	322.0928152	
5	10.75	53.917	419	419	9.927	10.73	8.655	123.031	466.6668935	
6	10.75	64.667	419	419	10.73	11.531	9.327	106.914	603.1723245	
7	10.75	75.417	419	419	11.531	12.216	9.950	97.587	750.3986221	
8	10.75	86.167	419	419	12.216	12.817	10.489	87.637	903.7907561	
9	10.75	96.917	419	419	12.817	13.344	10.961	77.148	1062.351722	
10	10.75	107.667	419	419	13.344	13.847	11.393	66.186	1226.653253	
11	10.75	118.417	419	419	13.847	14.35	11.815	54.793	1399.042738	
12	10.75	129.167	419	419	14.35	14.834	12.228	42.979	1579.466476	
13	10.75	139.917	419	419	14.834	15.244	12.603	30.751	1763.329457	
Penthouse	10.75	150.667	419	468	15.244	15.691	13.731	18.148	2068.751926	
Roof	12	162.667	200	40	15.691	15.989	3.778	4.417	614.5168859	
Bulkhead	8	170.667	40	0	15.989	0	0.640	0.640	109.1517865	13293.38758
NORTH/SOUTH DIRECTION										
2	21.667	21.667	614	305	7.634	7.905	7.098	98.568	153.7988878	
3	10.75	32.417	305	305	7.905	7.929	4.829	91.470	156.5536873	
4	10.75	43.167	305	305	7.929	9.982	5.463	86.641	235.8150618	
5	10.75	53.917	305	305	9.982	10.788	6.335	86.641	341.5561075	
6	10.75	64.667	305	305	10.788	11.592	6.826	74.843	441.4104753	
7	10.75	75.417	305	305	11.592	12.28	7.281	68.017	549.1081603	
8	10.75	86.167	305	305	12.28	12.883	7.675	60.736	661.3071674	
9	10.75	96.917	305	305	12.883	13.412	8.020	53.061	777.2719171	
10	10.75	107.667	305	305	13.412	13.916	8.335	45.041	897.4087517	
11	10.75	118.417	305	305	13.916	14.421	8.643	36.706	1023.452671	
12	10.75	129.167	305	305	14.421	14.907	8.945	28.064	1155.403982	
13	10.75	139.917	305	305	14.907	15.319	9.219	19.119	1289.885029	
Penthouse	10.75	150.667	305	125	15.319	15.768	6.643	9.900	1000.925328	
Roof	12	162.667	125	40	15.768	16.067	2.614	3.256	425.1594846	
Bulkhead	8	170.667	40	0	16.067	0	0.643	0.643	109.6842676	9218.740978

T-7

See Appendix A For Wind Calculations

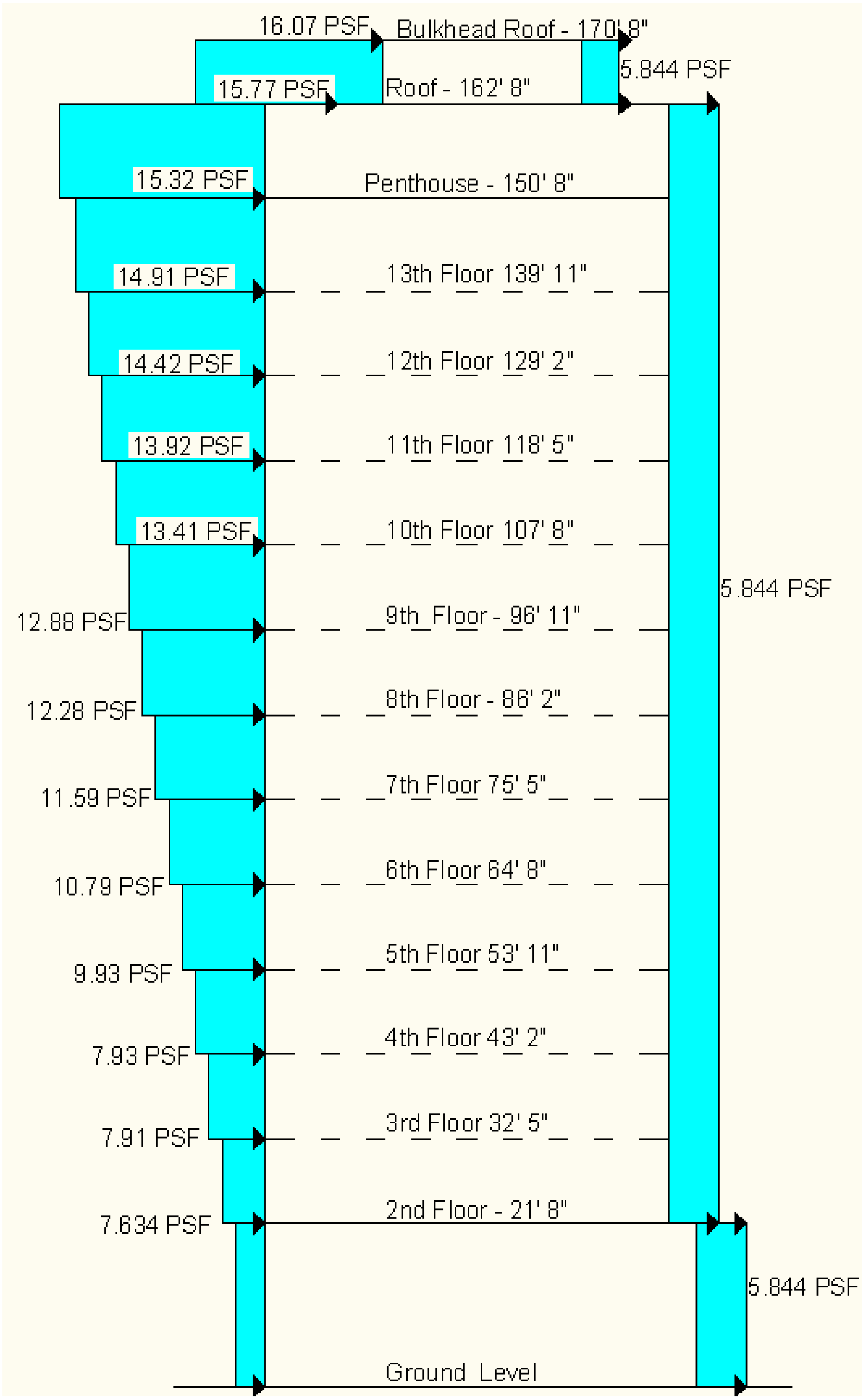
Wind Pressure Diagram for the East / West Diagram



F-4

Wind Pressure Diagram for the North/South Direction

WINDWARD

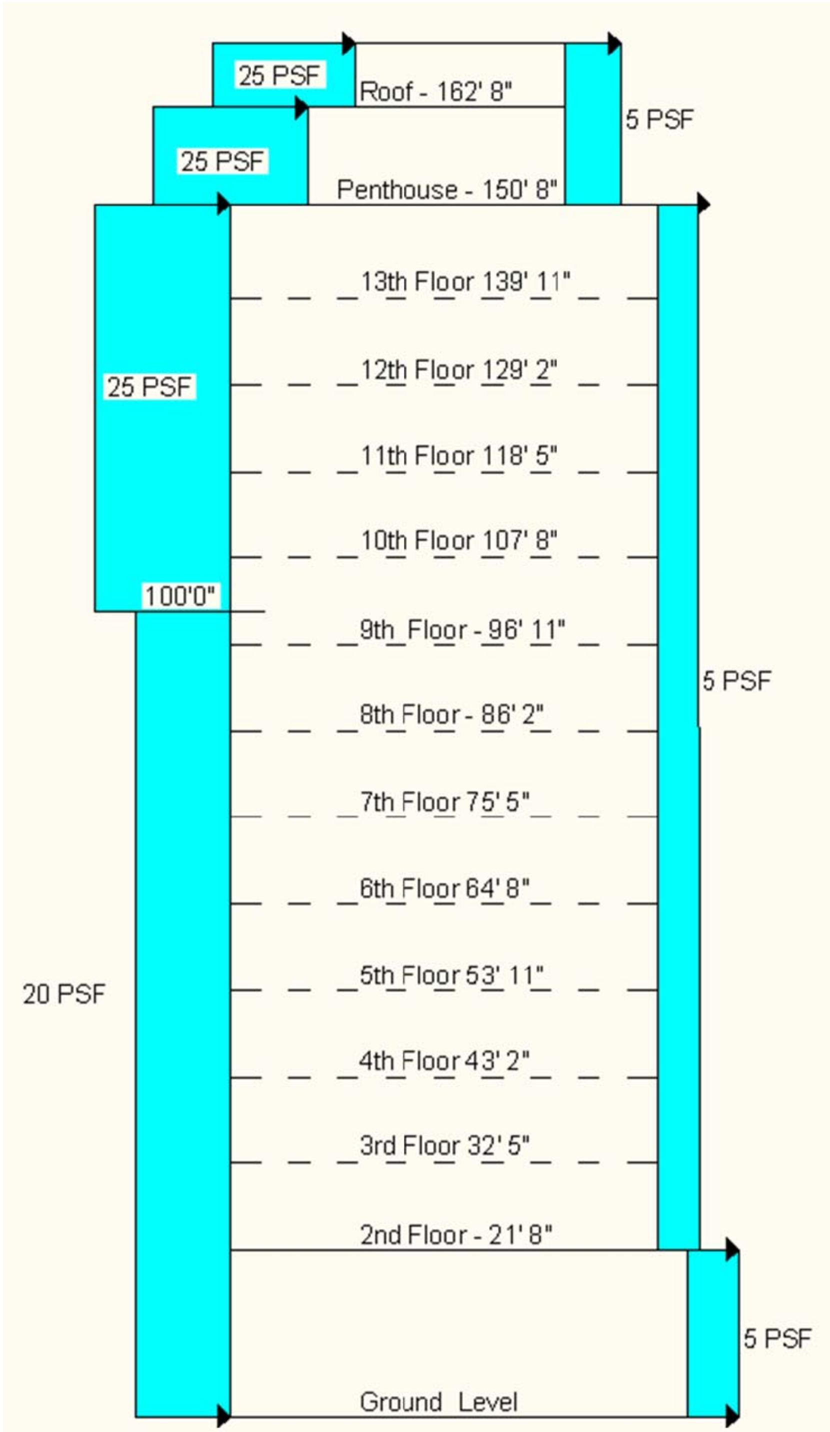


LEEWARD

F-5

WIND DIAGRAM USED IN ACTUAL DESIGN

WINDWARD



LEEWARD

F-6

SEISMIC CALCULATIONS

The following tables T-8 – T-16 , and corresponding calculations found in Appendix B, were obtained in accordance with IBC 2006 Section 1613.1 and by Referencing Chapters 12 and 13 of ASCE7-05. The 40 Gold street building is a slender steel framed structure located in Manhattan, New York. To quickly summarize the following tables, it is important to note the site class was recorded as D, the Seismic Design Category (SDC) was determined to be E, and the overall building weight was only 4,681,330 lbs (4,681.33 kips).

Based on the IBC Chapter 6 seismic flowcharts, it was determined that the Modal response spectrum Analysis should be conducted to determine seismic loads. However, for the purposes of this Thesis project, performing the analysis is not practical. Analytical procedures were therefore conducted according to the Equivalent Lateral Force Procedure.

Note: Dead Load values in column three of Table T-8 are broken down by more specific building components in table T-1.

Total Building Weight Calculations Due To Slab and Superimposed Loads

Floor	Floor Area	Dead Load	Additional Loads Required By Code	Floor Weight
Cellar				
1st				
Retail	3815	78	N/A	297570
Residential Lobby	714	96	N/A	68544
Retail Storage**	70	78	25% of Corresponding Live Load (100x.25 = 25)	7210
Mech. Mezzanine	176	71	N/A	12496
2nd				
Residential	4149	53	N/A	219897
Outdoor Terrace Exposed to Snow Loads	374	71	N/A	26554
3rd				
Residential	4149	53	N/A	219897
4th				
Residential	4149	53	N/A	219897
5th				
Residential	4149	53	N/A	219897
6th				
Residential	4149	53	N/A	219897
7th				
Residential	4149	53	N/A	219897
8th				
Residential	4149	53	N/A	219897
9th				
Residential	4149	53	N/A	219897
10th				
Residential	4149	53	N/A	219897
11th				
Residential	4149	53	N/A	219897
12th				
Residential	4149	53	N/A	219897
13th				
Residential	4149	53	N/A	219897
Penthouse				
Residential Recreation	2338	58	N/A	135604
Outdoor Terraces -West	869	71	N/A	61699

T-8

Outdoor Terraces - East	636	71	N/A	45156
Boiler Room / Mech	174	61	N/A	10614
Roof Level				
Flat Roof Exposed to Snow Loads	1895	47	N/A	89065
Mechancial Facility	236	61	N/A	14396
Bulkhead Roof				
Flat Roof Exposed to Snow Loads	525	71	N/A	37275
Note: Partitions, mechanical equipment, miscellaneous items were all properly superimposed and included with the dead loads (see table S-9 page 13) **TOTAL = 3,407,672 lbs**				

T-8 Continued

Building Envelope Dead Load Broken Down By Level (lbs)					
Floor	**h/2 below (ft)**	**h/2 above (ft)**	**Dead Load (PSF)**	**Perimeter of Building at Corresponding Story Elevation**	**Weight = 15*Perimeter*(h/2 below + h/w above)**
1	N/A	10'10"	15	274	44,525
2	10'10"	5' 4.5"	15	270	65,644
3	5' 4.5"	5' 4.5"	15	270	43,538
4	5' 4.5"	5' 4.5"	15	270	43,539
5	5' 4.5"	5' 4.5"	15	270	43,540
6	5' 4.5"	5' 4.5"	15	270	43,541
7	5' 4.5"	5' 4.5"	15	270	43,542
8	5' 4.5"	5' 4.5"	15	270	43,543
9	5' 4.5"	5' 4.5"	15	270	43,544
10	5' 4.5"	5' 4.5"	15	260	41,925
11	5' 4.5"	5' 4.5"	15	260	41,926
12	5' 4.5"	5' 4.5"	15	260	41,927
13	5' 4.5"	5' 4.5"	15	260	41,928
Penthouse	5' 4.5"	6'0"	15	252	42,998
Roof	6'0"	4'0"	15	156	23,400
Bulkhead Roof	4'0"	0	15	50	3,000
Total Building Envelope (lbs)					**652,060**

T-9

BUILDING WEIGHT DUE TO COLUMNS (lbs)							
Floor	**Column Shape**	**# Identical Columns**	**Column Height**	**Weight / Column**	**Total Weight**	**Final Breakdown**	
1st							
	W10x33	1	21'1"	696	696		
	W12x96	1	23'2"	2224	2224		
	W12x120	16	23'2"	2780	44480		
	W12x106	1	23'2"	2456	2456		
	W10x77	1	23'2"	1784	1784		
	W10x54	1	23'2"	1251	1251		
	W14x132	3	31'8"	4180	12540		
	W10x60	1	23'2"	1390	1390		
	W14x132	1	23'2"	3058	3058		
	W14x109	1	31'8"	3451	3451		
	W10x88	1	31'8"	2787	2787		
				Total = 76,117		First Floor Weight	76,117
2nd - 3rd							
	W10x88	10	21'6"	1892	18920		
	W10x77	10	21'6"	1656	16560		
	W10x59	1	21'6"	1268	1268		
	W10x49	2	21'6"	1054	2108		
	W10x54	1	21'6"	1162	1162		
	W10x68	3	21'6"	1462	4386		
				Total = 44,404		Second Floor Weight	22,202
						Third Floor Weight	22,202
4th-5th							
	W10x68	10	21'6"	1462	14620		
	W10x49	3	21'6"	1054	3162		
	W10x39	1	21'6"	839	839		
	W10x54	5	21'6"	1162	5810		
	W10x45	3	21'6"	968	2904		
	W10x60	2	21'6"	1290	2580		
	W10x77	2	21'6"	1656	3312		
	W10x35	1	21'6"	753	753		
				Total Weight = 33,980		Fourth Floor Weight	16990
						Fifth Floor Weight	16990
6th-7th							
	W10x68	7	21'6"	1462	10234		
	W10x60	3	21'6"	1290	3870		
	W10x54	2	21'6"	1162	2324		
	W10x39	6	21'6"	839	5034		
	W10x33	2	21'6"	710	1420		
	W10x49	5	21'6"	1054	5270		
	W10x33	1	29'9"	988	988		
				Total Weight = 29,140		Sixth Floor Weight	14570
						Seventh Floor Weight	14570

T-10

8th-9th							
	W10x54	3	21'6"	1162	3486		
	W10x45	2	21'6"	968	1936		
	W10x49	6	21'6"	1054	6324		
	W10x33	4	21'6"	710	2840		
	W10x60	3	21'6"	1290	3870		
	W10x39	8	21'6"	839	6712		
				Total Weight = 25,168		Eight Floor Weight	12584
						Ninth Floor Weight	12584
10th-11th							
	W10x49	7	21'6"	1054	7378		
	W10x39	6	21'6"	839	5034		
	W10x45	3	21'6"	968	2904		
	W10x33	9	21'6"	710	6390		
	W10x54	1	21'6"	1162	1162		
				Total Weight = 22,868		Tenth Floor Weight	11434
						Eleventh Floor Weight	11434
12th-13th							
	W10x39	3	19'0"	741	2223		
	W10x33	6	19'0"	627	3762		
	W10x45	4	19'0"	855	3420		
	W10x39	2	21'6"	839	1678		
	W10x33	5	21'6"	710	3550		
	W10x45	4	21'6"	968	3872		
	W10x54	1	21'6"	1162	1162		
	W10x49	1	21'6"	1054	1054		
				Total Weight = 20,721		Twelfth Weight	10361
						Thirteenth Weight	10361
Penthouse/Roof							
	W10x39	3	19' 1.5"	746	2238		
	W10x39	3	9'5"	367	1101		
	W10x39	1	24'6"	956	956		
	W10x33	1	11' 7.5"	384	384		
	W10x33	2	9'5"	311	622		
	W10x33	1	19' 1.5"	631	631		
	W10x33	3	24'6"	809	2427		
				Total Weight = 8,359		Penthouse Weight	4179.5
						Roof Terrace Weight	4179.5
Total Weight Due to All Building Columns = 260,757 lbs							

T-10 Continued

Column Weights were determined by referencing the provided column Schedule. Calculations involved determining the height and lb/ft of each column.

FIRST FLOOR Building Weight Due to Beams				
Beam	# of Identical Beams	Beam Length	Weight / Beam	Total Weight
W12x22	4	14'0"	308	1232
W12x22	1	17'6"	385	385
W12x22	5	8'9"	193	965
W12x22	1	6'7"	146	146
W12x22	1	12'0"	264	264
W12x26	4	17'6"	455	1820
W12x26	2	12'0"	312	624
W10x15	6	6'9"	101	606
W10x15	1	8'9"	131	131
W10x15	2	7'10"	118	236
W10x15	6	5'1"	77	462
W10x15	2	12'0"	180	360
W14x22	1	12'7"	279	279
W14x22	3	13'11"	306	918
W14x22	1	14'4"	315	315
W14x22	1	21'0"	662	662
W14x22	1	15'8"	345	345
W12x19	2	13'11"	264	528
W12x19	1	14'6"	276	276
W12x19	1	10'0"	190	190
W12x19	1	17'5"	331	331
W10x12	5	5'1"	61	305
W16x26	2	23'2"	602	1204
W21x182	1	23'2"	4216	4216
W21x182	2	14'5"	2639	5278
W24x176	1	12'6"	2200	2200
W12x16	1	5'0"	80	80
W12x16	1	8'6"	136	136
W24x279	2	13'0"	3627	7254
W18x35	1	11'7"	406	406
W18x35	1	18'6"	648	648
W10x30	2	6'8"	200	400
W16x45	1	17'1"	770	770
W10x17	2	8'10"	151	302
W24x306	1	15'2"	4641	4641
W16x36	1	17'8"	636	636
W16x31	1	14'2"	440	440
W24x131	1	14'2"	1858	1858
W24x250	1	15'5"	3861	3861
W8x24	1	8'9"	210	210
W12x35	1	17'8"	618	618
W14x30	1	14'7"	440	440
W18x119	1	23'2"	2759	2759
W21x201	1	23'2"	4660	4660
W8x15	2	7'9"	116	232
W12x35	1	17'8"	618	618
W24x279	1	17'6"	4883	4883
W8x13	1	10'2"	132	132
W8x13	2	2'11"	38	76
TOTAL WEIGHT DUE TO BEAMS AT FIRST FLOOR = 60,338 lbs				

T-11

SECOND FLOOR Building Weight Due to Beams				
Beam	**# of Identical Beams**	**Beam Length**	**Weight / Beam**	**Total Weight**
W12x22	3	15'9"	346	1038
W12x22	1	14'0"	308	308
W12x22	1	11'0"	242	242
W12x22	3	17'0"	374	1122
W12x22	3	14'7"	322	966
W12x22	2	11'8"	256	512
W8x10	4	7'0"	70	280
W8x10	9	2'4"	240	2160
W8x10	2	8'2"	81	162
W8x10	3	5'4"	53	159
W8x10	2	12'0"	120	240
W10x12	4	6'9"	81	324
W10x12	2	7'10"	94	188
W10x15	1	13'11"	208	208
W10x15	3	11'0"	165	495
W10x15	1	14'5"	216	216
W12x30	2	14'5"	433	866
W12x30	2	13'11"	417	834
W12x30	1	15'0"	450	450
W12x30	1	17'6"	525	525
W12x30	1	28'0"	840	840
W12x30	1	8'9"	262	262
W12x30	1	7'10"	234	234
W12x16	3	15'5"	246	738
W12x16	1	22'8"	362	362
W12x16	1	17'6"	280	280
W12x16	1	23'2"	371	371
W12x16	2	10'5"	166	332
W12x19	5	23'2"	440	2200
W12x19	2	15'5"	292	584
W12x19	1	10'11"	207	207
W8x13	3	8'10"	114	342
W12x35	1	25'0"	875	875
W12x35	1	17'0"	595	595
W12x26	2	15'8"	407	814
W12x26	3	17'8"	459	1377
W12x26	1	11'7"	303	303
W12x26	1	19'0"	494	494
W12x30	1	14'0"	420	420
W12x30	1	13'0"	390	390
W10x17	1	10'0"	170	170
TOTAL WEIGHT DUE TO BEAMS AT SECOND FLOOR = 23,485 lbs				
Note: This is approximately same weight for floors 3 - 13 and PENTHOUSE as well				

T-12

ROOF Building Weight Due to Beams				
Beam	**# of Identical Beams**	**Beam Length**	**Weight / Beam**	**Total Weight**
W12x22	2	12'0"	264	528
W12x22	1	8'2"	180	180
W12x22	1	10'11"	239	239
W12x22	1	17'8"	388	388
W12x22	5	20'0"	440	2200
W12x22	6	16'6"	363	2178
W10x15	8	8'9"	131	1048
W10x15	4	4'2"	62	248
W10x15	2	7'10"	107	214
W10x15	3	1'5"	22.5	67.5
W10x17	2	8'9"	148	296
W12x26	1	15'5"	400	400
W12x26	4	17'3"	448	1792
W12x26	1	14'4"	372	372
W12x19	1	14'8"	278	278
W12x19	2	11'6"	218	436
W12x19	1	10'0"	190	190
W12x16	1	5'6"	88	88
W12x16	2	6'7"	104	208
W8x10	2	3'0"	30	60
W10x22	6	7'0"	154	924
W12x35	1	15'5"	539	539
W12x35	1	17'4"	606	606
W12x30	2	15'0"	450	900
W12x50	1	11'7"	583	583
C8x11.5	16	3'8"	42	672
W8x13	3	3'5"	44	132
Total Weight At Roof Level Due To Beams = 15,766.5				

T-13

BULKHEAD ROOF Building Weight Due To Beams				
Beam	**# of Identical Beams**	**Beam Length**	**Weight / Beam**	**Total Weight**
W10x22	2	18'10"	410	820
W0x22	1	13'4"	293	293
W8x13	2	6'9"	87	174
W8x13	2	8'9"	113	226
W12x26	1	24'0"	624	624
W12x26	1	30'0"	780	780
Total Weight At Bulkhead Roof Level Due to Beams = 2,917				

T-14

Table T-14 Represents the Culmination of Tables T11-T13

Total Building Weight	kips
Columns:	260.757
1st Floor Beams	60.338
2nd - 13th Floor and Penthouse Beams	281.82
Roof Beams	15.766
Roof Bulkhead Beams	2.917
Slab and All Superimposed Loads	652.06
Building Envelope	3407.672
TOTAL SUM	**4681.33**

T-15

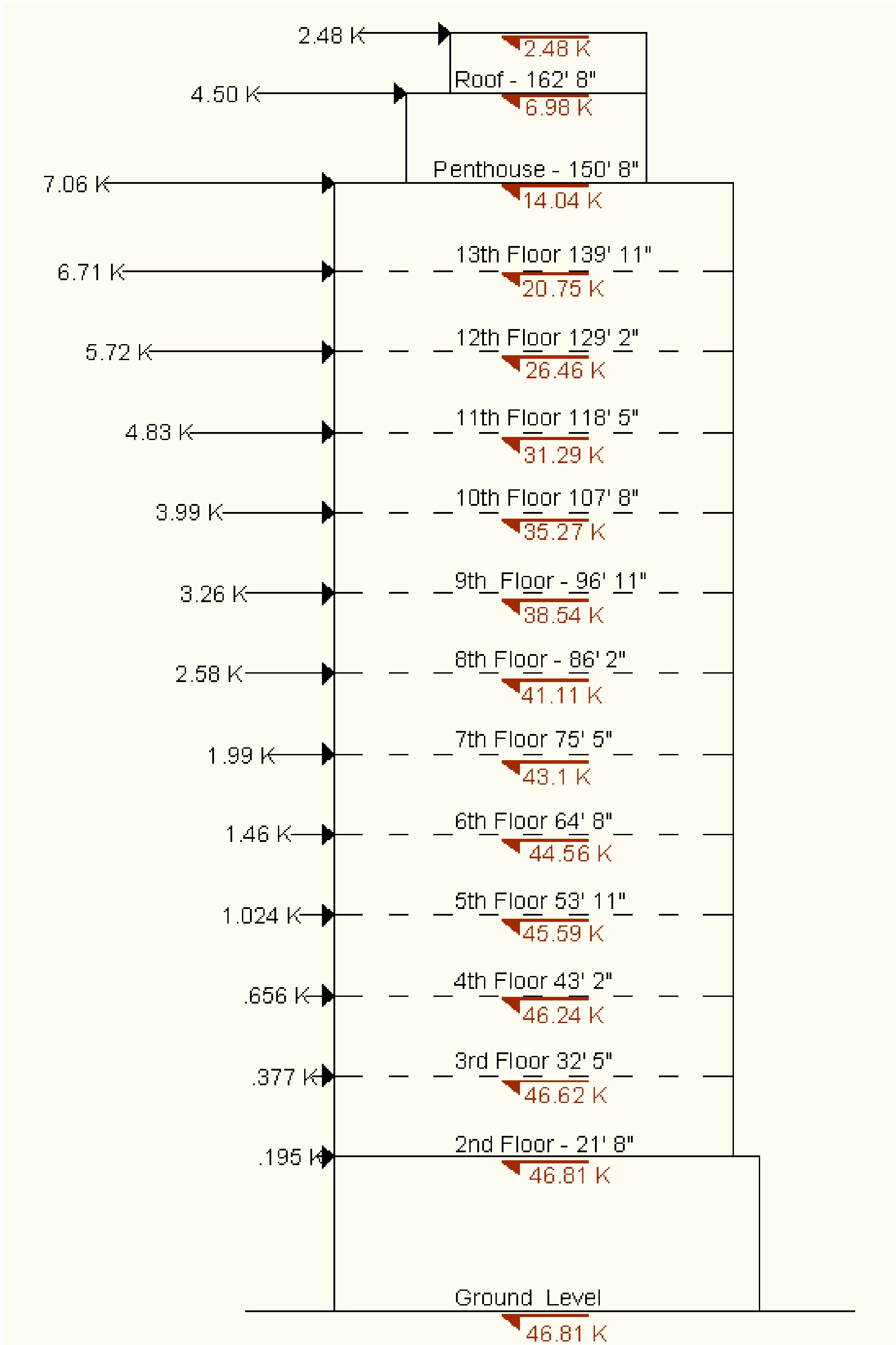
SEISMIC CALCULATIONS

Floor Level	Height (feet)	Total Weight (kips)	Exponent Related To Structure	Weight*Height^k	Wx*hxk / (∑Wx*hxk)	Base Shear (kips)	Lateral Seismic Force	Story Shear
	hx	Wx	K	Wx*hx^k		V	Fx (kips)	Vx (kips)
Ground / 1st	0	547.094	2	0	0	46.81	0	46.8133
2nd Floor	21.667	357.782	2	167963.9402	0.004158738	46.81	0.19468427	46.8133
3rd Floor	32.4167	309.122	2	324838.5164	0.008042907	46.81	0.376515038	46.61861573
4th Floor	43.1667	303.911	2	566296.8132	0.014021345	46.81	0.656385421	46.24210069
5th Floor	53.9167	303.911	2	883472.4799	0.021874522	46.81	1.024018574	45.58571527
6th Floor	64.667	301.493	2	1260789.725	0.031216788	46.81	1.461360853	44.5616967
7th Floor	75.4167	301.493	2	1714795.296	0.042457834	46.81	1.987591322	43.10033584
8th Floor	86.167	299.5	2	2223713.191	0.055058493	46.81	2.577469772	41.11274452
9th Floor	96.9167	299.5	2	2813157.598	0.069652966	46.81	3.260685192	38.53527475
10th Floor	107.667	296.74	2	3439864.35	0.085170043	46.81	3.98709079	35.27458956
11th Floor	118.4167	296.74	2	4161041.053	0.103026169	46.81	4.82299497	31.28749877
12th Floor	129.167	295.67	2	4932991.954	0.12213945	46.81	5.717750697	26.4645038
13th Floor	139.9167	295.67	2	5788237.845	0.14331509	46.81	6.709052291	20.7467531
Penthouse	150.667	268.21	2	6088513.145	0.150749819	46.81	7.057096505	14.03770081
Roof	162.667	146.8065	2	3884581.158	0.096181102	46.81	4.502554804	6.980604304
Bulkhead Roof	170.667	73.4	2	2137938.307	0.052934732	46.81	2.4780495	2.4780495
				∑Wx*hx^k= 40,388,195.37				

T-16

Reference Appendix B for Full Seismic Calculations

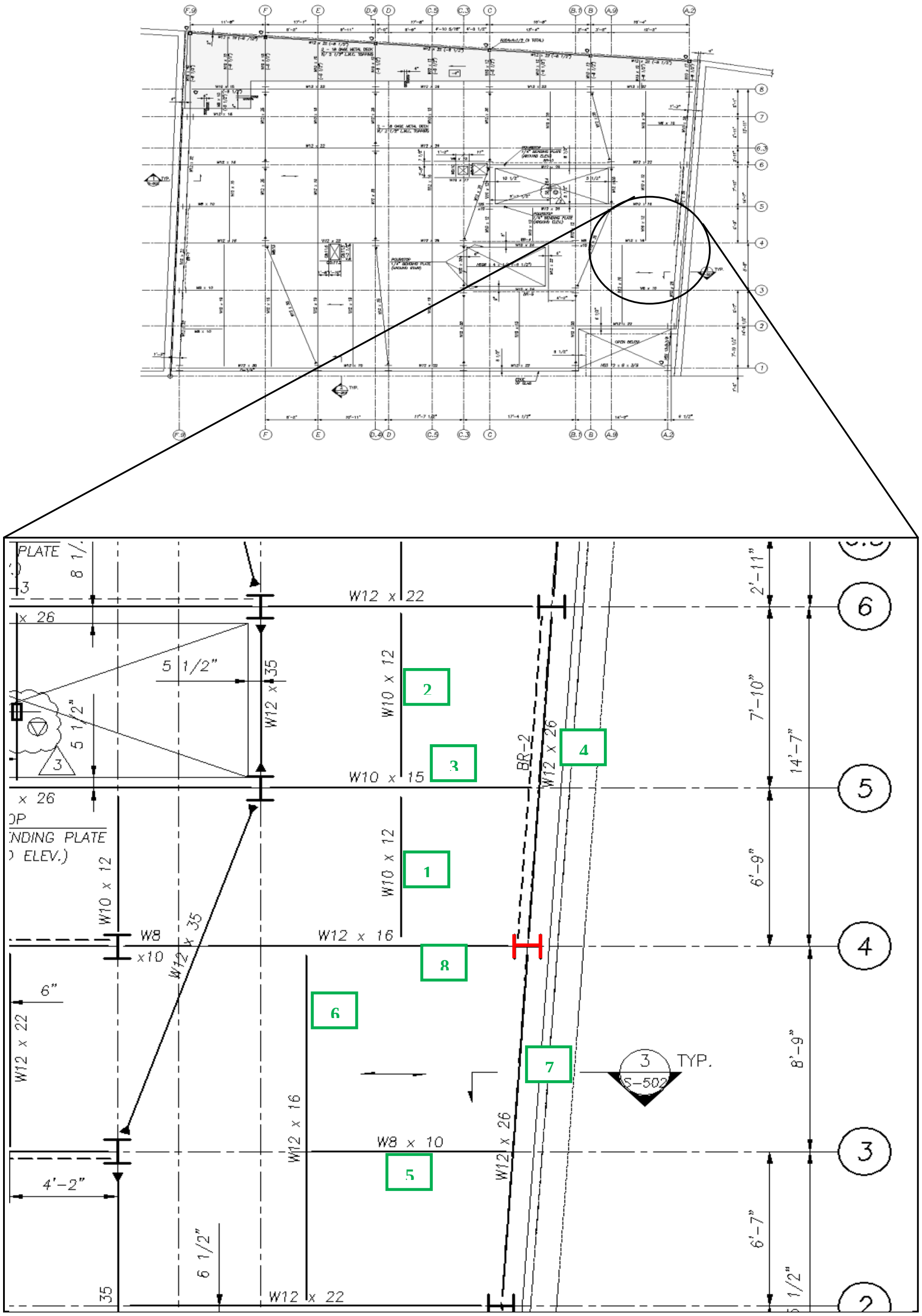
Seismic Forces and Story Shears Diagram



F-7

SPOT CHECKS - COLUMN

Calculations were done to verify the design of an exterior 2nd Floor Column that carries the load of 13 floors above. Using the load paths and tributary areas, the compressive force and moments that are applied to the column were determined. The framing around the column is typical all the way up through floor 13.
It is important to note, the self weights of the beams and columns were included in the calculations.



Please note, figure S-11 shows the column of interest in red. The Green numbers represent the beams that transfer loads (ultimately) to the column, and are referenced in the following calculations.

S - 11

FIRST: MUST DETERMINE COMPRESSIVE FORCE ON COLUMN. THE LOADS FROM THE SECOND FLOOR (LIVE, DEAD, SELF WEIGHT OF BEAMS) WILL BE CALCULATED, AND MULTIPLIED BY # OF STORIES SUPPORTED ABOVE. THIS METHOD IS VALID SINCE THIS SECTION OF FRAMING IS TYPICAL THROUGHOUT THE FLOORS. THE COLUMN WEIGHTS WILL THEN BE CALCULATED.

REFER TO FIGURE S-11 FOR BEAM # REFERENCES → "$\boxed{\#}$"

$\boxed{1}$ 2ND FLOOR RESIDENTIAL W10x12

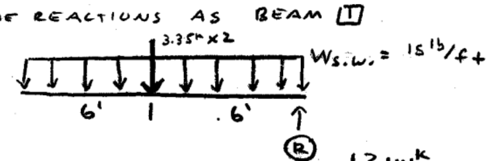
$1.2D + 1.6L + \text{Self weight} = 1.2(83\text{ psf} \times 6') + 1.6(40\text{ psf} \times 6') = 981\text{ lb/ft}$

$981\text{ lb/ft} + 12\text{ lb/ft} = 993.6\text{ lb/ft}$

REACTION: $R = \dfrac{w_u l}{2} = \dfrac{(.9936\text{ k/ft})(6'9'')}{2} = 3.35\text{ kips}$

$\boxed{2}$ SAME REACTIONS AS BEAM $\boxed{1}$

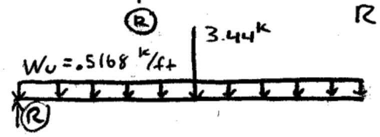
$\boxed{3}$



SYMMETRICAL: $R = \dfrac{(15/1000 \cdot 1.2)\cdot 12' + 6.7}{2}$

$R = 3.44^k$

$\boxed{4}$ W12x26



$W_u = 1.2D + 1.6L + W_{sw} = 1.2(3' \times 83 + 26\text{ lb/ft}) + 1.6(40 \times 3)$

$W_u = .5168$

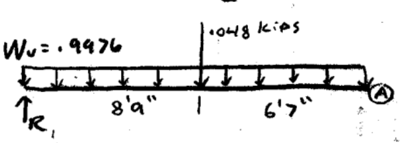
SYMMETRY: $R = \dfrac{3.44^k + .5168\text{ k/ft}(14'7'')}{2}$

$R = 12.49\text{ kips}$

$\boxed{5}$ W8X10 – BEAM ONLY SERVES AS TIE DOWN FOR DECKING
S.W. WILL STILL TRANSFER TO COLUMN

$\text{Reactions} = \dfrac{1.2 \times 10 \times 8'}{2} / 1000 = .048\text{ kips}$

$\boxed{6}$ W12x16



NOT SYMMETRICAL

$W_u = 1.2(6 \times 83\text{ psf}) + (1.6 \cdot 40\text{ psf} \cdot 6') + (1.2 \cdot 16\text{ lb/ft})$

$W_u = .9976\text{ kip/ft}$

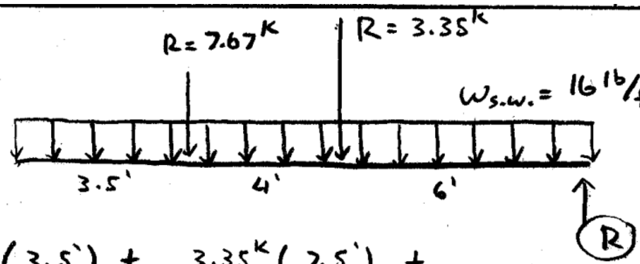
$\circlearrowright + \quad \sum M_A = 0 = -0.48(6'7'') - (.9976 \times 15.333)(15.333/2) + R(15.3$

$R = 7.67\text{ kips}$

$\boxed{7}$ W12x26 Assume $R = \frac{1}{2}(7.67)$ → ONLY ½ TRIBUTARY AREA OF BEAM $\boxed{6}$

$R = 3.835^k$

8 W12X16



$$R = \frac{(7.67^k)(3.5') + 3.35^k(7.5') + 1.2(16^{lb}/ft)(13.5')(13.5'/2)}{13.5'}$$

$R = 3.979$ kips

LOAD ON COLUMNS $= 12.619^k + 3.979^k + 3.835^k = 20.3^{kips}$

MUST INCLUDE FLOORS 3–13 AS WELL $\therefore$ $20.3^k \times 13 = 262^k$

Column s.w. →

HEIGHT	lb/ft	WEIGHT
21.5'	77	1.65^k
↓	45	.967
	39	.838
	39	.838
	33	.71
	33	.71
	$\Sigma =$	5.713^k

TOTAL WEIGHT ON COLUMN =
$5.713^k + 262 = \underline{267^k}$

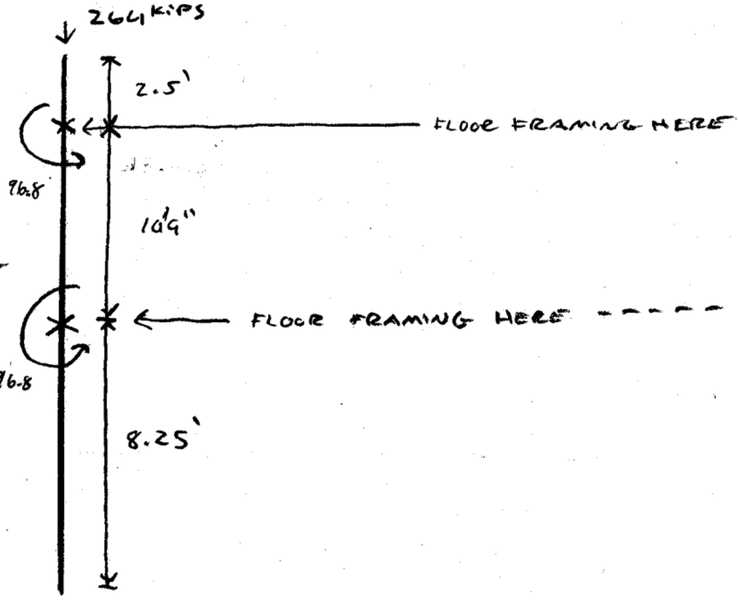
COLUMN SPOT CHECK

LOCATION – SECOND FLOOR EXTERIOR COLUMN, ACTUAL DESIGN = W10 x 88
BEAM GRID (4) AND GRID (A.2)

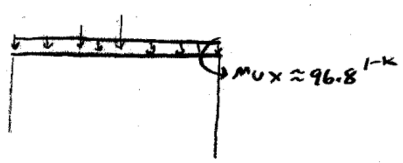
COMPRESSIVE FORCE = SOLVED → KIPS = P_u = 263.9^k

COLUMN HEIGHT = 21'6"

- COLUMNS SPLICED 2½' ABOVE EACH FLOOR
- FLOOR FRAMES INTO COLUMN PROVIDING LATERAL BRACING AS SHOWN



THE MOMENT THE COLUMN MUST RESIST IS BASED ON THE MOMENTS AT THE BEAM ENDS THAT FRAME INTO IT! W12x26 BEAMS CREATE CANCELLING MOMENTS. HOWEVER THE W12x16 BEAMS CREATE A MOMENT M_{UX} =



DESIGN METHOD – BRACED FRAME – NO TRANSLATION
– COLUMN IS PART OF BRACED FRAME 2.

STEP ONE: $M_{ux} = 96.8^{1-k}$
$P_u = 264$ K

STEP TWO: $C_m = 0.6 - 0.4\left(\frac{-96.8}{96.8}\right) = .6 + .4 = .2$

REVERSE CURVATURE, SO RATIO NEGATIVE

STEP THREE $P_{e1} = \frac{\pi^2 E I}{(KL \times 12)^2} = \frac{\pi^2 (29000)(534)}{(\cdot 1 \times 10'9 \times 12)^2} = \underline{18,744}$

STEP FOUR: $B_1 = \dfrac{C_m}{1 - \dfrac{P_u}{P_{e1}}}$

$$B_1 = \frac{0.2}{1 - \frac{264}{18,744}} = .203 < 1 \quad \text{SO} \quad B_1 = 1$$

STEP FIVE: TABLE 6-1

$p \times 10^3 = 1.03 \rightarrow .00103$

$b_x \times 10^3 = 2.13 \rightarrow .00213$

$.00103(264) > .2$

$.2719 > .2$ OKAY USE H1-1A

$pP_u + b_x \cdot B_1 \cdot M_{ux} < 1$

$.00103(264) + \left(\frac{2.13}{1000} \cdot 1.0 \cdot 96.8\right)$

$= .478 < 1$ OKAY ✓

ACTUAL DESIGN VERIFIED

SPOT CHECK – BEAM

A spot check for a typical interior beam was performed, and the calculations can be found below. The beam is located in the retail sector of the building on the first floor.

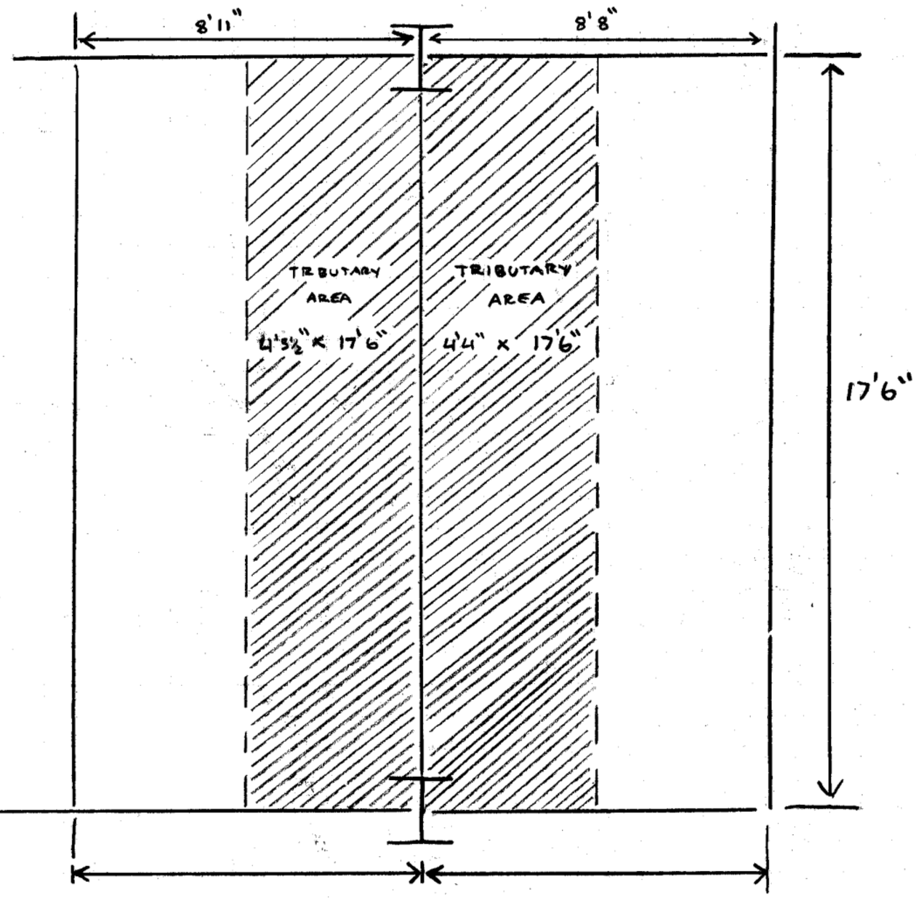
JESSE COOPER

SIMPLY SUPPORTED BEAM – SPOT CHECK

LOCATION – Grid D.4, BETWEEN 4 AND 6.3 Grid lines (Ground Floor)

LIVE LOAD – 100 PSF (RETAIL)
DEAD LOAD – 34 PSF SLAB, 4 PSF STEEL, 8 PSF CEILING/MECH,
12 PSF PARTITIONS, 20 PSF MISCELLANEOUS DEAD LOAD
TOTAL DEAD LOAD = 78 PSF

ASSUME FULL LATERAL BRACING – BEAM WELDED TO THE STEEL DECK
→ WELDING ELECTRODES CONFORM TO ASTM SPECIFICATIONS E-70XX.



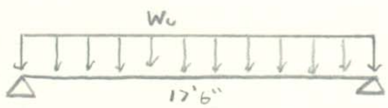
STEEL IS ASTM A572, GRADE 50
yield pt. = 50 ksi
TENSILE STRENGTH = 65 ksi

STEP ONE: LIVE LOAD = 100 PSF DOES NOT EXCEED 100 PSF
HOWEVER WILL TAKE CONSERVATIVE APPROACH AND NOT USE LIVE LOAD REDUCTION

STEP TWO: $W_D = 78 \text{ PSF} \times (4'4'' + 4'5\tfrac{1}{2}'') = 685.75 \text{ lb/ft}$
$W_L = 879.16 \text{ lb/ft}$

$W_U = 1.2D + 1.6L = 2{,}230 \text{ lb/ft} = 2.23 \text{ kip/ft}$ * CONTROLS
$W_U = 1.4D = 960.05 \text{ lb/ft}$

STEP THREE:



FOR LOADING CONDITION

$$M_U = \frac{W_U L^2}{8} =$$

INCLUDE 26 lb/ft SELF WEIGHT LOAD AND CHECK LATER
NEW $W_U = 2.256$

$$M_U = \frac{(2.256)(17'6'')^2}{8} = 86.36 \text{ k-ft}$$

STEP FOUR: DON'T NEED A LATERAL-TORSIONAL BUCKLING MODIFICATION FACTOR C_b
↳ "FULLY SUPPORTED"

TABLE 3-2 (AISC MANUAL) – FIND LEAST WEIGHT W-SHAPE WITH $\phi M_p > M_U$

$W12\times19 \rightarrow \phi M_p = 92.6 \text{ kip-ft} > 86.36 \text{ k-ft}$

STEP FIVE: MAX SHEAR DUE TO LOADING $= \dfrac{W_U L}{2} = 19.51 \text{ kips}$

TABLE 3-2

$85.7^k > 19.51^k$
OKAY

STEP SIX: DEFLECTION CHECK

LIVE LOAD $= W_L = 1{,}407 = 1.41^k$

AISC – TABLE 3-23 $\quad \Delta_{MAX} = \dfrac{5wl^4}{384EI} = \dfrac{5(1.41^k)(17.5)^4\,1728}{384(29000)(I_{required})}$

LIMIT TO $l/360$

$$\frac{(17.5 \times 12)}{360} = \frac{164.16}{I_{required}} \qquad I_{required} = 176 \text{ in}^4$$

$W12\times19 \quad I_x = 130 \text{ in}^4 < 176 \text{ in}^4$ WON'T WORK

$W12\times22 \quad I_x = 156 \text{ in}^4 < 176 \text{ in}^4$ WON'T WORK

$W12\times26 \quad I_x = 204 \text{ in}^4 > 176 \text{ in}^4$ WORKS

THIS DESIGN AGREES WITH ACTUAL DESIGN – W12×26 ✓

CONCLUSION

Response To Wind Analysis
The wind pressures, story shears, and overturning moment were determined using ASCE7-05 Method 2 for Main Wind Force Resisting Systems. The building is located in a crowded area of Manhattan New York, and so it is not overly exposed. However, the structure is classified as having a partially exposed roof. The building does rise up to 170'8" above grade; however, the building dimensions are rather small and so the wind pressure values do not have a very significant effect. When comparing the lateral wind force values Fx found in table **T-7** to the seismic lateral forces Fx, it was apparent that the wind forces in the East-West direction are the controlling lateral condition. This makes a good deal of sense since the building is very light and the heaviest floors are located near the bottom reducing seismic effects. Also, the East-West Direction Wind Forces being larger than the North/South Wind Forces makes sense since the building dimension perpendicular to the East/West Wind Direction (76'8") is greater than the other Building dimension (56'0").

Response To Seismic Analysis

The Seismic Design Loads calculated for 40 Gold Street are relatively low compared to the actual Design Loads used by Severud Associates. The actual base shear design value is 309 kips. However, the actual design value was obtained according to New York City Building Code as opposed to ASCE7-05. Until further analysis is completed, it will be assumed the differences in seismic design loads is a result of differing codes.

An extensive total building weight calculation was done, and is documented in figures **T-8 –T-15.** In table **T-8** the weight contribution from the concrete slabs, metal decking, miscellaneous dead loads, mechanical loads, partitions loads, and other superimposed loads were accounted for using floor area values and PSF values found in table **T-2**. Table **T-9** documents the weight contributed by all the columns which was calculated by determining the height of every column and the lb/ft of every column. Table **T-10**, represents the weight of the exterior façade which was assumed to be 15 psf. Finally, table T-11 represents the weight contributed by all the beams which was determined using lb/ft of each beam and the span of each beam. The total building weight calculated is only 4681.33 kips. At first, the value appears small; however, the value should be expected since it is only 56' x 78' steel framed building with a floor system composed of slab on deck (concrete topping only 2 ½"). In fact, the structure was originally intended to be concrete; however, due to poor site conditions steel was favored as the most appropriate structural material. The steel was also favored since it is more versatile and was more able to accommodate for the fact that footings were offset to avoid disturbing nearby existing foundations.

The Building Shear was calculated to be 46.81 kips using the equation: $V = C_s * W_{Building}$. As seen in figure……, the moment frames and braced frames are located in very symmetrical manner. In addition, the overall building shape and weight distribution is very symmetrical. As a result, the center of rigidity and Center of Mass are located close to each other, and it is fair to conclude that the potential for torsion effects due to seismic loading is minimal. Overall, despite being located on a poor site, seismic loads are minimal since 40 Gold Street is constructed with light steel framing consisting of mainly W10/W12 and resulting in a low overall building weight. Therefore, the overturning moment and story shears are minimal.

Response To Structural Spot Checks

Spot checks were performed to verify the structural system design. A typical interior beam check was completed and yielded verification of the original W12x26 Design. Prior to calculations, the W12x26 sizing and other similar beams commonly found throughout the structure appeared appropriate. The framing is laid out with small rectangular bays. As a result, the beams do not span great distances and are not required to support large tributary areas. In addition, aside from retail areas, the building serves as residential space, and so the live load is only 40 PSF. In addition, the floor system is extremely light being composed of slab on 2” deck with 2-1/2” light weight concrete topping. Originally, the calculations suggested a W12x19 may be the lightest weight W-shape beam that could support the load; however, the live load deflection obtained by $(5*w*l^4) / (384*E*I)$ exceeded the span/360 limit. Once again, this result was expected since the beam spans a large distance (17’6”) relative to the other beams in the structure.

A column spot check was also performed and verified the design of a Common Exterior Column located at the second Floor. The column is part of a braced frame and has two floors framing into it. As a result, the column was analyzed assuming no sway or translation. The column supports 13 stories of load which included self weights and was calculated with an in depth procedure involving tributary areas and load path procedures. The column spans two floors amounting to a 21’6” length. However, since the column is part of a braced frame and beams frame into the column at the middle section of the column the effective length was shortened and the sway problem is eliminated. As a result, the column is not overly large. The design moments were determined by summing the moments at the beam ends that frame into the column. The H1-1A interaction equation $pP_u + b_x*B_1*M_{ux} < 1$ was used to verify the design. In the end, the equation yielded a value of .478 meaning the W10x88 is able to resist a simultaneous moment load Mux = 96.8 kip-ft and a compressive force Pu = 264 kips. Once again, the column sizing benefited from small tributary areas and the low residential live load of 40 PSF.

APPENDIX A – WIND CALCULATIONS

WIND CALCULATIONS | TECHNICAL REPORT 1 | JESSE COOPER

DETERMINING WIND LOADS

CODE - IN ACCORDANCE WITH IBC 2006 SECTION 1609.1.1
↳ PROVISIONS OF CHAPTER 6 IN ASCE / SEI 7-05 (ASCE7)

- METHOD 2 FOR THE MWFRS

FLOWCHART 5.5 : VELOCITY PRESSURES, q_z AND q_H

1.) DETERMINE BASIC WIND SPEED V FROM FIG. 6-1 OR FIG 1609 (6.5.4)

NEW YORK CITY - 110 MPH (49 m/s)

2.) DETERMINE WIND DIRECTIONALITY FACTOR K_d (TABLE 6-4)
STRUCTURE TYPE — BUILDINGS - MWFRS COMPONENTS / CLADDING
$K_d = 0.85$

3.) DETERMINE IMPORTANCE FACTOR I FROM IBC TABLE 1604.5 AND TABLE 6-1 (6.5.5)
OCCUPANCY = II → NON - HURRICANE PRONE AREA
$I = 1.0$

4.) EXPOSURE CATEGORY (6.5.6)
SURFACE ROUGHNESS - B - URBAN AREA, CLOSE SPACED OBSTRUCTIONS, NYC.
EXPOSURE B APPLIES IN ALL WIND DIRECTIONS

5.) ARE ALL 5 CONDITIONS OF 6.5.7.1 MET? NO
TOPOGRAPHY - NOT SITUATED ON A HILL
- NOT ON AN ESCARPMENT
$K_{zT} = 1.0$ → TOPOGRAPHIC FACTOR

6.) DETERMINE VELOCITY PRESSURE COEFFICIENTS K_z AND K_h.
TABLE 6-3 (6.5.6.6)

TABLE 6-2	α	z_g	$\hat{a}$	$\hat{b}$	$\bar{\alpha}$	$\bar{b}$	c	ℓ	$\bar{\epsilon}$	z_{min}
	7.0	1200	1/2	.84	1/4	.45	.3	320	1/3	30

WIND CALCULATIONS | TECHNICAL REPORT 1 | JESSE COOPER

ELEVATION	HEIGHT	K_z
GROUND FLOOR	0'	.7
2ND	21' 8"	.7
3RD	32' 5"	.7145
4TH	43' 2"	.7158
5TH	53' 11"	.8256
6TH	64' 8"	.8687
7TH	75' 5"	.9117
8TH	86' 2"	.9485
9TH	96' 11"	.98075
10TH	107' 8"	1.009
11TH	118' 5"	1.036
12TH	129' 2"	1.063
13TH	139' 11"	1.089
PENTHOUSE	150' 8"	1.111
ROOF	162' 8"	1.135
BULKHEAD ROOF	170' 8"	1.151 = K_H

LINEAR INTERPOLATION

USING TABLE 6-3 CASE 2 EXPOSURE B

SAMPLE LINEAR INTERPOLATION

$$\left(\frac{32'5"-30}{40-30}\right)(.76-.7)+.7$$

$$= .7145$$

* FOR 3RD FLOOR

TABLE 6-3 CASE 2, EXPOSURE B

HEIGHT ABOVE GROUND VS. K_z

FEET	K_z
0-15	0.7
20	0.7
25	0.7
30	0.7
40	0.76
50	0.81
60	0.85
70	0.89
80	0.93
90	0.96
* 100	0.99
120	1.04
140	1.09
160	1.13
180	1.17

SAMPLE CALCULATION FOR K_z

FOR $Z = 100$ FEET

$$K_z = K_{100} = 2.01\left(Z/Z_g\right)^{(2/\alpha)}$$

$$K_{100} = 2.01\left(100/1200\right)^{(2/7)}$$

$K_{100} = 0.99$

WIND CALCULATIONS	TECHNICAL REPORT 1	JESSE COOPER

AFTER LINEAR INTERPOLATION FOR K_z VALUES,

- DETERMINE VELOCITY PRESSURE AT HEIGHT Z AND H BY EQ. 6-15

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

$$q_H = 0.00256\, K_H\, K_{zt}\, K_d\, V^2 I$$

Sample CALCULATION – 8TH FLOOR

$$q_z = q_{86'2''} = (.00256)(.9485)(1.0)(.85)(110^2)(1.0$$

$$q_{86'2''} = \underline{24.97 \text{ PSF}}$$

LEVEL	HEIGHT	K_z	q_z
GROUND			
2ND			
3RD			
4TH			
5TH			
6TH			
7TH			
8TH			
9TH			
10TH			
11TH			
12TH			
13TH			
PENTHOUSE			
ROOF			
BULKHEAD ROOF			

WIND CALCULATIONS | TECHNICAL REPORT 1 | JESSE COOPER

FLOWCHART 5.6 - METHOD 2 - GUST EFFECT FACTORS, G AND G_F.

$B = 78' \, 2\frac{1}{2}''$ BUILDING DIMENSION PERPENDICULAR TO WIND
$L = 56.79'$ BUILDING DIMENSION HORIZONTAL (PARALLEL TO WIND
$h = 170' \, 8''$ MEAN ROOF HEIGHT.
$\beta = 1.5$ DAMPING RATIO
$n_1 = 0.363$ NATURAL FREQUENCY

REFERENCE C6.5.8 - APPROXIMATE FUNDAMENTAL FREQUENCY
↳ STEEL MOMENT RESISTING FRAMES

$$n_1 = 22.2 / h^{(.8)} = 22.2 / 170'8''^{(.8)} = \boxed{.363 \geq 1 \text{ Hz} \text{ FLEXIBLE}}$$

$$g_Q = g_v = \boxed{3.4}$$

$$\bar{z} = 0.6h = 0.6(170'8'') = \boxed{102.4} > z_{min} \text{ OKAY} ✓$$

$$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6} = 0.6\left(\frac{33}{102.41}\right)^{1/6} = \boxed{.4968}$$

$$L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}} = 320\left(\frac{102.4}{33}\right)^{1/3} = \boxed{466.74}$$

$$Q = \sqrt{\frac{1}{1 + 0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{.63}}} = \sqrt{\frac{1}{1 + 0.63\left(\frac{78'2\frac{1}{2}'' + 170'8''}{466.74}\right)^{.63}}}$$

$$Q = \boxed{.8380}$$

V = 110 MPH

WIND CALCULATIONS | TECHNICAL REPORT 1 | JESSE COOPER

$$\bar{V}_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}} V\left(\frac{88}{60}\right) = (.45)\left(\frac{102.4}{33}\right)^{1/4}(110)\left(\frac{88}{60}\right)$$

$$\bar{V}_{\bar{z}} = \boxed{96.3569}$$

$$N_1 = \frac{n_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}} = \frac{.363\,(466.741)}{96.3569} = \boxed{1.7583 = N_1}$$

$$R_n = \frac{7.47\,N_1}{(1 + 10.3N_1)^{5/3}} = \frac{7.47\,(1.7583)}{(1 + 10.3(1.7583))^{5/3}} = \boxed{.09615 = R_n}$$

$$R_h = \frac{1}{\eta} - \frac{1}{2\eta^2}\left(1 - e^{-2\eta}\right) \quad \text{for } \eta > 0$$

$$\eta = 4.6\,n_1\,h/\bar{V}_{\bar{z}} = \frac{4.6\,(.363)\,(170'\,8'')}{96.3569} = \boxed{2.957 = \eta}$$

$$R_h = \frac{1}{2.957} - \frac{1}{2(2.957)^2}\left(1 - e^{(-2\cdot 2.957)}\right) = \boxed{.28115 = R_h}$$

$$R_B = \frac{1}{\eta} - \frac{1}{2\eta^2}\left(1 - e^{-2\eta}\right) \quad \text{where } \eta = 4.6\,n_1\,B/\bar{V}_{\bar{z}}$$

$$\eta = \frac{4.6\,(.363)\,(78'\,2\tfrac{1}{2}'')}{96.3569} = \underline{1.3553}$$

$$R_B = \frac{1}{1.3553} - \frac{1}{2(1.3553)^2}\left(1 - e^{(-2\cdot 1.3553)}\right) = \boxed{.4837 = R_B}$$

$$R_L = \frac{1}{\eta} - \frac{1}{2\eta^2}\left(1 - e^{(-2\eta)}\right) \quad \text{where } \eta = 15.4\,n_1\,L/\bar{V}_{\bar{z}} = \underline{3.2948}$$

$$\boxed{R_L = .2575}$$

$$R = \sqrt{\frac{1}{\beta} R_n R_h R_B \left(0.53 + 0.47 R_L\right)} = \boxed{.09226 = R}$$

damping ratio

WIND CALCULATIONS	TECHNICAL REPORT 1	JESSE COOPER

NOTE: β = STRUCTURAL DAMPING

* MEASURE OF ENERGY DISSIPATION IN A VIBRATING STRUCTURE THAT RESULTS IN BRINGING THE STRUCTURE TO A QUIESCENT STATE.

IN WIND APPLICATION, DAMPING RATIOS OF 1% – 2% ARE TYPICALLY USED IN UNITED STATES FOR STEEL + CONCRETE BUILDINGS.

TAKE MEAN OF RANGE: 1.5% = β

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

$$G_f = 0.925\left(\frac{1 + 1.7(.4968)\sqrt{(3.4)^2(.838)^2 + (3.788)^2(.09226)^2}}{1 + 1.7(3.4)(.4968)}\right)$$

$$\boxed{G_f = .8845}$$

$$G_R = \sqrt{2 \ln(3{,}600 n_1)} = 3.788$$

WIND CALCULATIONS | TECHNICAL REPORT 1 | JESSE COOPER

FLOWCHART 5.7

IS BUILDING ENCLOSED OR PARTIALLY ENCLOSED ? YES

PARAPET ? YES BUT
↳ PARAPET AT PENTHOUSE LEVEL, 4' HIGH.
HOWEVER TWO FLOORS RISE ABOVE PARAPET TO 170.67'

q_P MEASURED @ TOP OF PARAPET — 154.67'

$$q_P = q_{154.67'} = .00256\, K_z K_{zt} K_d V^2 I$$

$$= .00256\,(1.119341)(1.0)(.85)(110^2)(1.0)$$

$$\boxed{q_P = 29.47 \text{ PSF}}$$

LINEAR INTERPOLATION WHERE $K_z = \dfrac{154.67' - 140'}{160' - 140'}(1.13 - 1.09) + 1.09$

$K_z = 1.11934$

DETERMINE COMBINED NET PRESSURE COEFFICIENT GC_{pn} :

$GC_{pn} = +1.5$ WINDOWARD

$GC_{pn} = -1.0$ LEEWARD

DETERMINE COMBINED NET DESIGN PRESSURE ON PARAPET

$P_P = q_P\, GC_{pn} = (29.47 \text{ PSF})(1.5) = 44.207$ WINDWARD

$(29.47 \text{ PSF})(-1.0) = -29.47$ LEEWARD

LOW RISE BUILDING ? NO → h > 60' 0"

WIND CALCULATIONS | TECHNICAL REPORT 1 | JESSE COOPER

- BUILDING IS FLEXIBLE → NOT RIGID ✓

- CORRESPONDING GUST FACTOR G OR G_F FROM FLOWCHART 5.6

$G_F = .8845$ ✓

DETERMINE VELOCITY PRESSURE q_z FOR WINDWARD WALLS ALONG THE HEIGHT OF THE BUILDING AND q_h FOR LEEWARD WALLS, SIDE WALLS, ROOF. ✓

DETERMINE C_P FOR WALLS + ROOF (FIG. 6-6 or 6-8)

WINDWARD → $C_p = .8$

LEEWARD → $C_p = -0.5$ → $L/B = 56.79/78'2\frac{1}{2}" = .726 < 1$

SIDE WALLS → $C_p = -0.7$

DETERMINE INTERNAL PRESSURE COEFFICIENTS GC_{pi} (FIG 6-5)

$GC_{pi} = +0.18$
$\phantom{GC_{pi} =} -0.18$

DETERMINE DESIGN WIND PRESSURES EQ. 6-19

$$\boxed{\begin{array}{ll} P_z = q_z G_F C_P - q_h (GC_{pi}) & \text{WINDWARD} \\ \quad \text{WHERE } C_P = .8 & \\ P_z = q_h G_F C_P - q_h (GC_{pi}) & \text{LEEWARD} \\ \quad \text{WHERE } C_P = -0.5 & \end{array}}$$

SAMPLE CALCULATION

STORY 10 - WINDWARD FACE: $P_z = P_{107'8"} = (26.57)(.8845)(.8) - 30.31(.18)$

$P_z = 13.345$

JESSE COOPER

NORTH / SOUTH DIRECTION

* NOTE THE EXACT SAME PROCEDURE + PROVISIONS WERE FOLLOWED TO OBTAIN THE WIND PRESSURE VALUES FOR THE EAST / WEST DIRECTION.

SOME VALUES CHANGED DUE TO CHANGE IN BUILDING DIMENSIONS W/ RESPECT TO WIND PATH! (SEE BELOW)

NEW VALUES

$B = 56.79'$, $L = 78'\,2\tfrac{1}{2}''$

$$Q = \sqrt{\frac{1}{1 + 0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{.63}}} = \sqrt{\frac{1}{1 + .63\left(\frac{56.79 + 170'8''}{466.74}\right)^{.63}}} = \underline{.8449 = Q}$$

R_L: $\eta = \dfrac{15.4\,(.363)\,(78'\,2\tfrac{1}{2}'')}{96.3569} = 4.537$

$$R_L = \frac{1}{4.537} - \frac{1}{2(4.537)^2}\left(1 - e^{(-2 \cdot 4.537)}\right) = \underline{.1961 = R_L}$$

R_B: $\eta = \dfrac{4.6\,(.363)\,(56.79)}{96.3569} = \boxed{.9841}$

$$R_B = \frac{1}{.9841} - \frac{1}{2(.9841)^2}\left(1 - e^{(-2(.9841))}\right) = \underline{.2605 = R_B}$$

$$R = \sqrt{\tfrac{1}{1.5}(.09615)(.28115)(.2605)(.53 + .47(.1461))}$$

$\underline{R = .07179}$

$$G_f = .925\left(\frac{1 + 1.7(.4968)\sqrt{(3.4)^2(.8449)^2 + (3.788)^2(.07179)^2}}{1 + 1.7(3.4)(.4968)}\right)$$

$G_f = .8877$

NO PARAPET IN THIS DIRECTION

DETERMINE C_p: $\boxed{\text{WINDWARD: } C_p = .8}$

$L/B = 78'2\tfrac{1}{2}'' / 56.79' = 1.377 > 1.0$

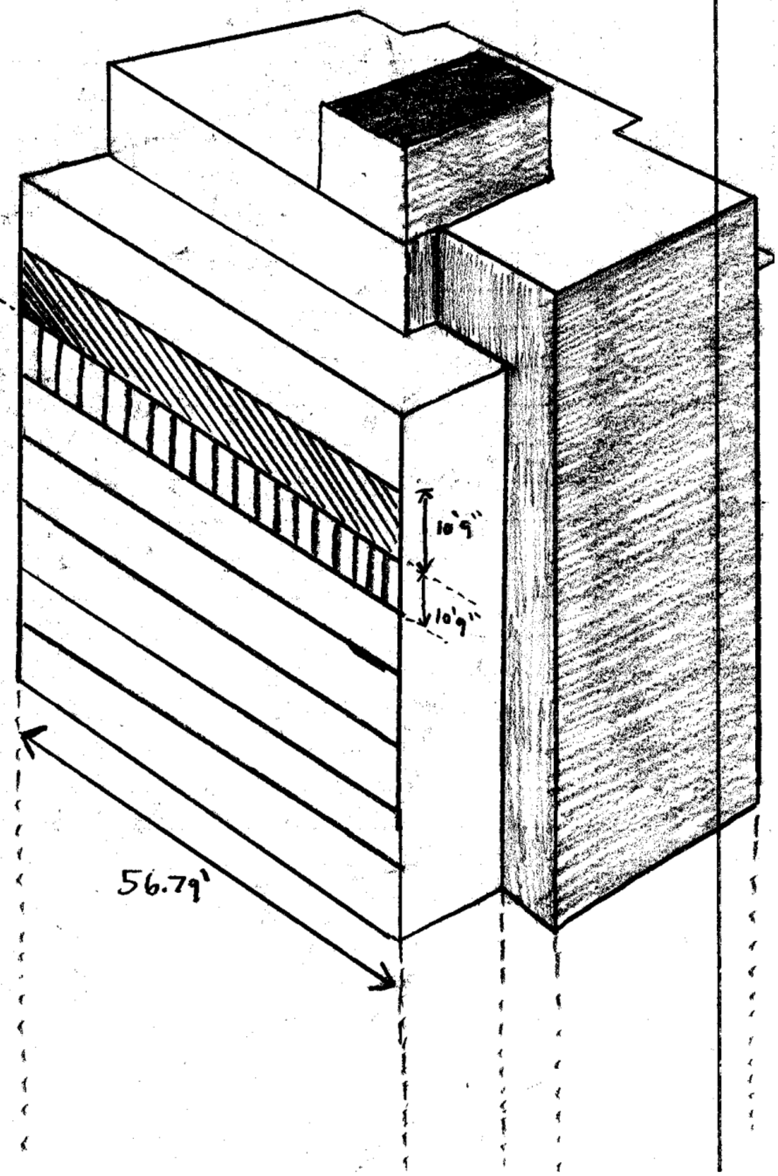
LINEAR INTERPOLATION: $\boxed{C_p = -.42 \text{ LEEWARD}}$

WIND CALCULATIONS	TECHNICAL REPORT I	JESSE COOPER	10

SAMPLE CALCULATION OF F_x VALUES DUE TO WIND

NORTH/SOUTH DIRECTION

FLOOR 12



= 14.412 PSF

= 13.92 PSF

F_x @ FLOOR 12 = F_{12}

$$F_{12} = \left(\frac{10'9''}{2} \times 56.79'\right)(14.412\ \text{psf}) + \left(\frac{10'9''}{2} \times 56.79'\right)(13.92\ \text{PSF})$$

F_{12} = 8,650.67 lbs

$\boxed{F_{12} = 8.7\ \text{KIPS}}$

APPENDIX B – SEISMIC CALCULATIONS

SEISMIC CALCULATIONS	TECHNICAL REPORT 1	JESSE COOPER

DETERMINING SEISMIC LOADS

FLOWCHART 6.1 – CONSIDERATION OF SEISMIC DESIGN REQUIREMENTS

11.1.2 EVERY STRUCTURE, AND PORTION THEREOF, INCLUDING NONSTRUCTURAL COMPONENTS, SHALL BE DESIGNED AND CONSTRUCTED TO RESIST THE EFFECTS OF EARTHQUAKE MOTIONS

- IS THE STRUCTURE A DETACHED ONE-OR-TWO FAMILY DWELLING? NO
- AGRICULTURAL STORAGE STRUCTURE? NO
- STRUCTURE REQUIRE SPECIAL CONSIDERATION? NO
- SEISMIC REQUIREMENTS – ASCE/SEI 7-05 MUST BE CONSIDERED

FLOWCHART 6.2 SEISMIC GROUND MOTION VALUES (11.4)

- DETERMINE THE PARAMETERS S_s and S_1 from the 0.2 and 1.0s SPECTRAL RESPONSE ACCELERATIONS ON FIG. 22-1 through 22-14 (11.4.1)

$S_s = 35 = \boxed{.35}$

$S_1 = 6.2 = \boxed{.062}$

IS $S_s \leq 0.15$ and $S_1 \leq 0.04$
NO NO

IS THE STRUCTURE SEISMICALLY ISOLATED OR DOES IT HAVE DAMPING SYSTEMS ON SITES w/ $S_1 \geq .6$. NO

$S_1 = .062 < .6$

$\boxed{A}$ DETERMINE THE SITE CLASS OF THE SOIL IN ACCORDANCE w/ 11.4.2 + CHPT. 20 (11.4.2 SEE FLOWCHART 6.3)

IS THE SITE CLASSIFIED AS SITE CLASS F?

TABLE 20.3-1 BASED ON upper 100ft of site profile

SEISMIC CALCULATIONS | TECHNICAL REPORT 1 | JESSE COOPER

DETERMINING SITE CLASS
• REVIEW GEOTECHNICAL REPORT ✓

ACCORDING TO GEOTECHNICAL REPORT
FILL - STRATUM CONSISTS OF HETEROGENEOUS MIXTURE OF FINE TO COARSE SAND, GRAVEL, SILT, BRICK / CONCRETE FRAGMENTS.
MEDIUM - DENSE CONDITION

IMMEDIATELY BELOW FILL - EXTENDS DOWN BELOW BORING PENETRATION
→ $\boxed{\text{SILTY SANDS}}$ - MEDIUM

ROBUST LIQUEFACTION ANALYSIS DUE TO POOR SITE CONDITIONS
→ ULTIMATELY → CONCLUDED LIQUEFACTION IS UNLIKELY DURING SEISMIC GROUND SHAKING EVENT.

SITE CLASS - CONSERVATIVE → SITE D

DETERMINE S_{MS} and S_{M1} BY EQS. 11.4-1 AND 11.4-2

$$S_{MS} = F_a S_S = 1.52\,(.35) = \boxed{0.532 = S_{MS}}$$

$$S_{M1} = F_v S_1 = 2.4\,(.062) = \boxed{0.1488 = S_{M1}}$$

TABLE 11.4-1 → $\boxed{F_a = 1.52}$ (LINEAR INTERPOLATION BELOW)
TABLE 11.4-2 → $\boxed{F_v = 2.4}$

USE STRAIGHT LINE INTERPOLATION FOR INTERMEDIATE VALUES OF S_S

$S_S = .35$

$$\begin{Bmatrix} S_S = .25 \longrightarrow \text{SITE D} \rightarrow F_a = 1.6 \\ S_S = .5 \longrightarrow \text{SITE D} \rightarrow F_a = 1.4 \end{Bmatrix}$$

$$1.6 - \frac{(0.35 - 0.25)}{(0.5 - 0.25)}\left(1.6 - 1.4\right) = 1.52 = F_a$$

SEISMIC CALCULATIONS | TECHNICAL REPORT 1 | JESSE COOPER

DETERMINE $\boxed{S_{DS}}$ AND $\boxed{S_{D1}}$ BY EQS 11.4.3 AND 11.4-4

$$\underline{S_{DS}} = 2S_{MS}/3 = 2(.532)/3 = \underline{.3547}$$

$$\underline{S_{D1}} = 2S_{M1}/3 = 2(.1488)/3 = \underline{.0992}$$

FLOWCHART 6.4

OCCUPANCY = II

$S_1 \geq 0.75$? NO

STRUCTURE IS ASSIGNED TO $\boxed{\text{SDC E}}$

SDC = B OR C ? NO

"IS THE STRUCTURE AN OCCUPANCY I or II OF LIGHT FRAME CONSTRUCTION THAT IS ≤ 3 STORIES ?" NO

≤ 2 STORIES ? NO

DETERMINE $\boxed{T_S = S_{D1}/S_{DS}} = .0992/.3547 = \underline{.2797}$

DETERMINE STRUCTURE FUNDAMENTAL PERIOD $\boxed{T}$

$$T_a = C_t h_n^x \qquad h_n = 193.01 - 6.34 = 186.67$$

TABLE 12.8-2

BRACED FRAMES → $.03 = C_t$, $x = .75$

$$T_a = (.03)(186.67)^{.75} = 1.515$$

$\boxed{T_a \approx T = 1.515}$ $\qquad 3.5T_s = 3.5(.2797) = \underline{.97895}$

$\boxed{\text{IS } T < 3.5T_s \text{ ?}}$ NO

THE FOLLOWING ANALYSIS PROCEDURES CAN BE USED:
- MODAL RESPONSE SPECTRUM ANALYSIS (12.9)
- SEISMIC RESPONSE HISTORY PROCEDURES (CHAPT. 16)

* NOTE: DUE TO THE POOR SITE CONDITIONS AND THE SDC, THE MODAL RESPONSE SPECTRUM ANALYSIS SHOULD BE PERFORMED.

HOWEVER FOR THE PURPOSES OF THIS THESIS PROJECT, PERFORMING THE ANALYSIS IS NOT PRACTICAL.

ANALYTICAL PROCEDURES WILL THEREFORE BE CONDUCTED ACCORDING TO THE EQUIVALENT LATERAL FORCE PROCEDURE.

RESPONSE MODIFICATION COEFFICIENT R FROM TABLE 12.2-1

DUAL SYSTEM W/ MOMENT FRAMES CAPABLE OF RESISTING @ LEAST 25% OF PRESCRIBED SEISMIC FORCES.

- STEEL ECCENTRICALLY BRACED FRAMES

$R^a = 8$

SYSTEM OVERSTRENGTH FACTOR: $\Omega_0{}^g = 2\tfrac{1}{2}$

DEFLECTION AMPLIFICATION FACTOR $4 = C_d{}^b$

FIGURE 22-15 $\rightarrow T_L = 6$

$T > T_L$? NO

$$C_s = \frac{S_{D1}}{T(R/I)} \leq \frac{S_{DS}}{(R/I)}$$

$$C_s = \frac{.6992}{1.515(8/1)} = .00818 < .01 \quad \therefore \quad \boxed{C_s = .01}$$

TOTAL BUILDING WEIGHT CALCULATED = $\boxed{41,681,330 \text{ lbs}}$
AND
WEIGHT / FLOOR ✓

DETERMINE BASE SHEAR V by EQ. 12.8-1 $\boxed{V = C_s W}$

$C_s = .00818 \rightarrow C_s = .01$

$V = .01\ (4{,}681.33) = \boxed{46.81^{\text{kips}} = V}$

$T = 1.515 \leq 0.5$ sec ? NO $T \geq 2.5$ sec ? NO

* FOR STRUCTURES HAVING A PERIOD BETWEEN 0.5 AND 2.5, K SHALL BE 2 OR SHALL BE DETERMINED BY LINEAR INTERPOLATION BETWEEN 1 AND 2.

$k = 2$

- DETERMINE LATERAL SEISMIC FORCE Fx @ EACH LEVEL.

$$\boxed{F_x = \frac{W_x\, h_x^{\,k}}{\sum w_i\, h_i^{\,k}}\, V =}$$

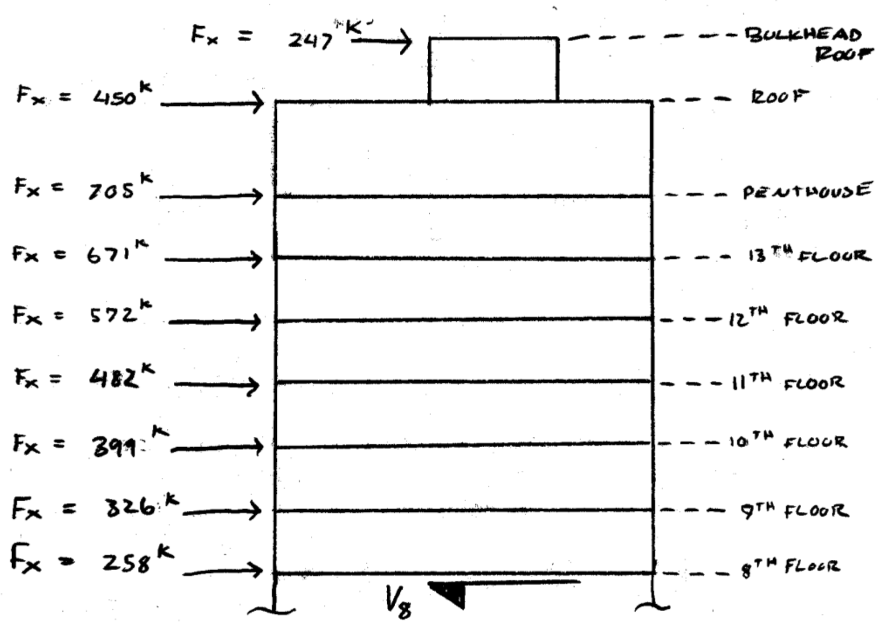
Sample calc

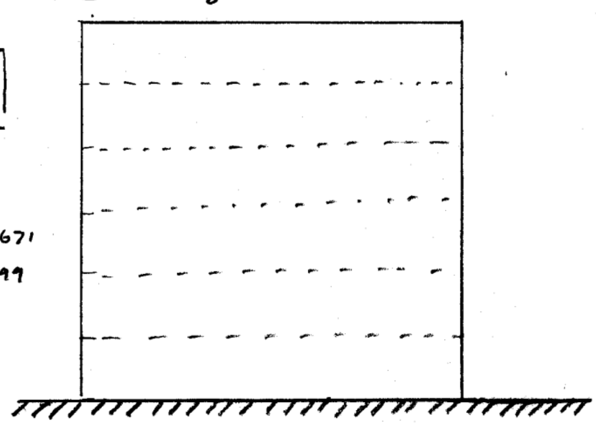
7^{TH} FLOOR $\rightarrow$ $F_7 = \dfrac{(301.493)(75.4)^2}{(40{,}388{,}195.37)}(4{,}681.3) = 198.8$ kip.

SEISMIC CALCULATIONS | TECHNICAL REPORT I | JESSE COOPER | 6

DETERMINE SEISMIC DESIGN STORY SHEAR V_x BY EQUATION 12.8-13

$$V_x = \sum_{i=x}^{n} F_i$$





SAMPLE CALCULATION OF STORY SHEAR

8TH STORY:

$$V_8 = 247 + 450 + 705 + 671 + 572 + 482 + 399 + 326 + 258$$

$$\boxed{V_8 = 4110 \text{ KIPS}}$$

ALL V_x VALUES ARE RECORDED IN FIGURE