40 Gold Street Residential Building

New York, New York

TECHNICAL REPORT 3

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Executive Summary

40 Gold Street is a slender 14 story residential building located in Manhattan, New York. The lateral system is comprised of 5 braced frames and 4 moment frames. In the third technical report, a detailed analysis of the lateral system was performed to confirm the design according to various criteria including strength, drift, story drift, and overturning moments.

With the aid of two preliminary analysis methods and a 3D ETABS model, the center of rigidity and relative stiffness values were first determined. According to the results, torsion effects are minimal and the center of rigidity is nearly equivalent to the center of mass.

In order to properly confirm the existing lateral design, several loading combinations were defined and applied to the ETABS model. Considering the 7 basic load combinations of ASCE7-05 section 3.2.3 and the 4 wind cases defined in ASCE7-05 figure 6.9, 38 different load combinations were applied to the model. By comparing story shear and story drift output, the controlling loading conditions in both the X and Y direction were determined to be Wind case 1 in conjunction with either ASCE7-05 equation 4: **1.2D + 1.6W+ L + .5Lr** or ASCE7-05 equation 6: **.9D + 1.6W.**

Using unfactored wind loads calculated in technical report 1, drift and story drifts under the 4 ASCE7-05 wind cases were checked for serviceability issues. Based on the ETABS output, wind case one controls, and the corresponding drift and story drifts did not exceed the allowable drift: $\Delta_{WIND} = H / 400$. 3D output data for ASCE 7-05 Load Combinations 5: **1.2D + 1.0E + 1.0L** and load combination 7: **.9D + 1.0E** were examined to verify seismic induced story drifts do not exceed the allowable drift: $\Delta_{SEISMIC} = .015hsx$. As expected, the design satisfies the stability requirement associated with seismic loading.

Based on inspection, it was determined the braced frames possess the largest overturn potential. After comparing uplift forces with the counteracting dead loads, it was determined that 8 different locations require pile caps with uplift resistance. With a low total building weight of just 4,681 kips, overturning due to lateral loads was expected to be an issue.

The last stage of design confirmation required strength checks of critical cross braces and lateral columns. Based on inspection of 3D output, braced frame BR-3 resists the largest story shears under the controlling loading condition. As a result, member checks of braced frame BR-3 were assumed to represent an overall design check of the lateral system.

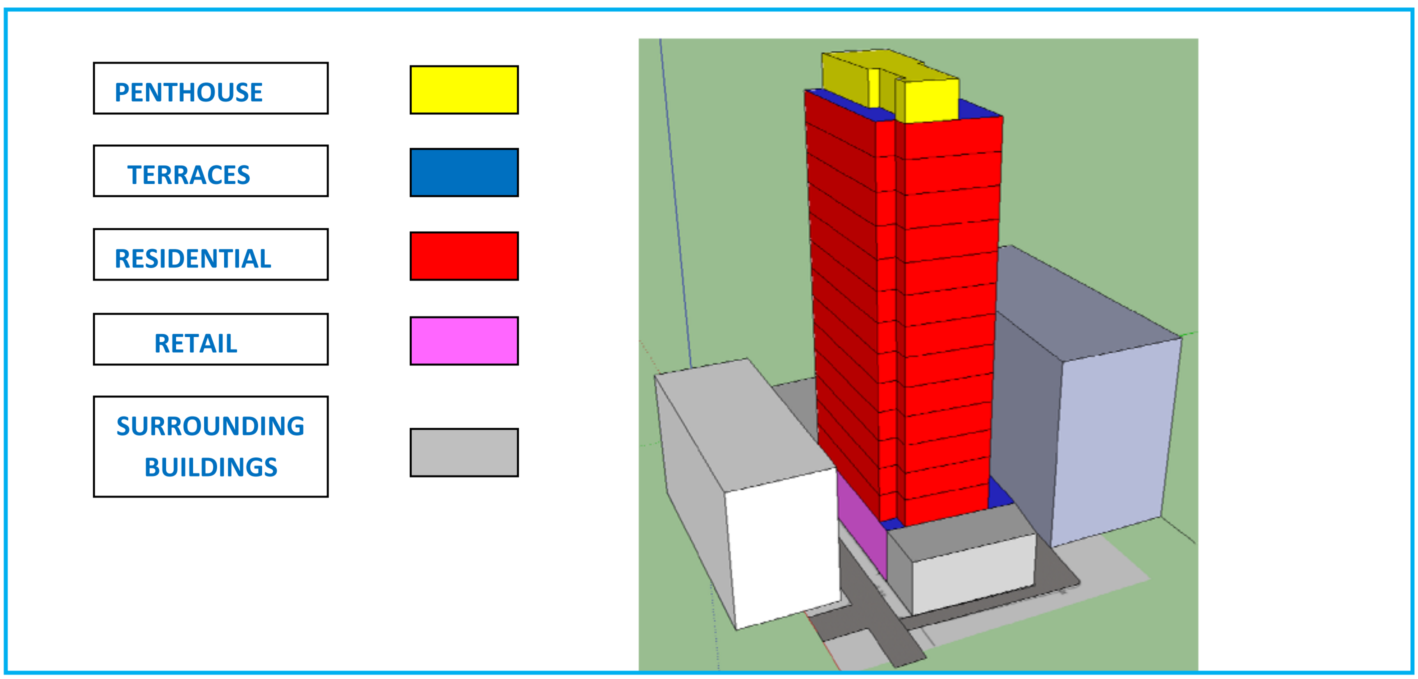
In conclusion, the existing lateral system design is adequate meeting all standard criteria and requirements. With a spread out lateral system, torsion effects are minimal. Wind loads generate the largest story shears and story drifts. The largest area of concern pertains to the overturning moment; however, appropriately designed pile caps (as shown in report) provide an easy solution to uplift issues.

Introduction

40 Gold Street is an impressive building 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 for the slender 14 story residential tower. The lowest floor, referred to as the cellar, is below grade and functions as extra retail space as well as 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 accompanied by 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 building's 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 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 rectangular manner, and the exterior shell is also 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 makes the most of a small site, and is certainly playing a major role in the successful rebuilding of Lower Manhattan.

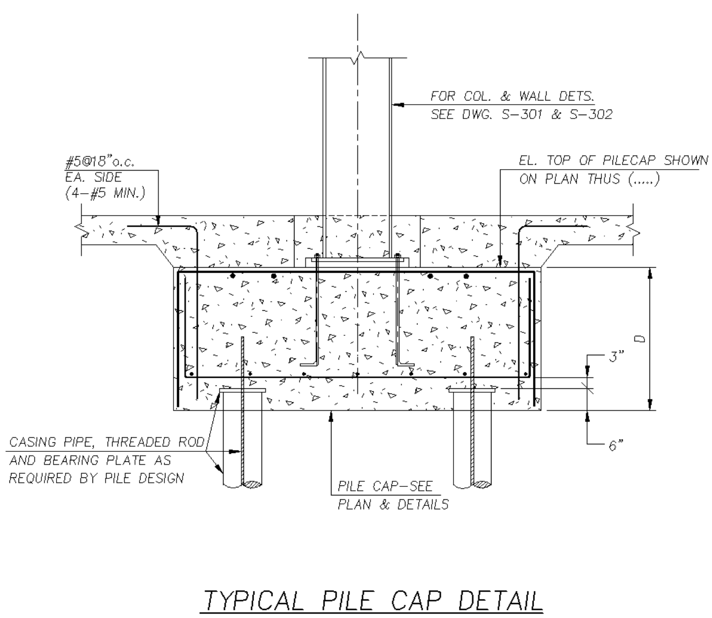
Structural System Overview

Foundations

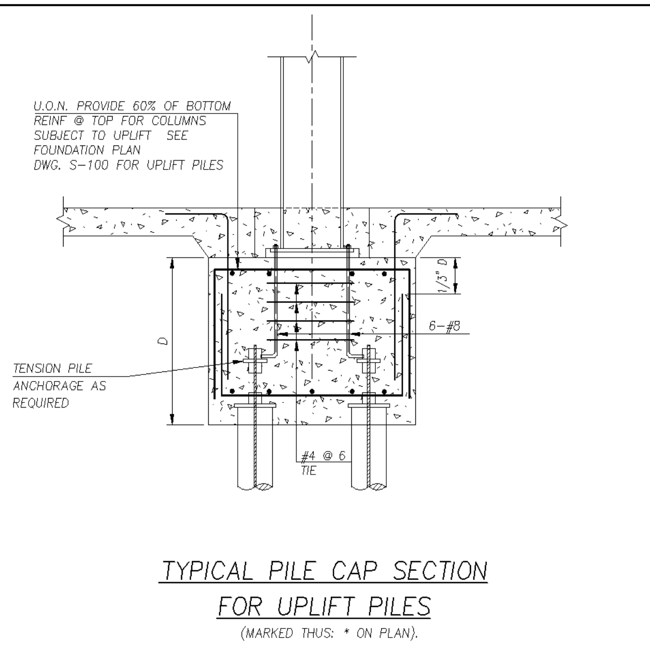
The site excavation and foundation work required a great deal of design work and creative planning compared to the average building project. As mentioned in the introduction, the site is very constricted with two existing structures against the property line, and two streets (Eden's Alley and Gold Street) are in close 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 underpin the adjacent existing structures. As a result, the depth of the various foundation components 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 (88) 75 Ton compression capacity piles that are 35' long and (13) 35 Ton compression capacity piles that are 25' long. Various pile caps are used to distribute building loads to the piles: they 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**.



S-1



S-2

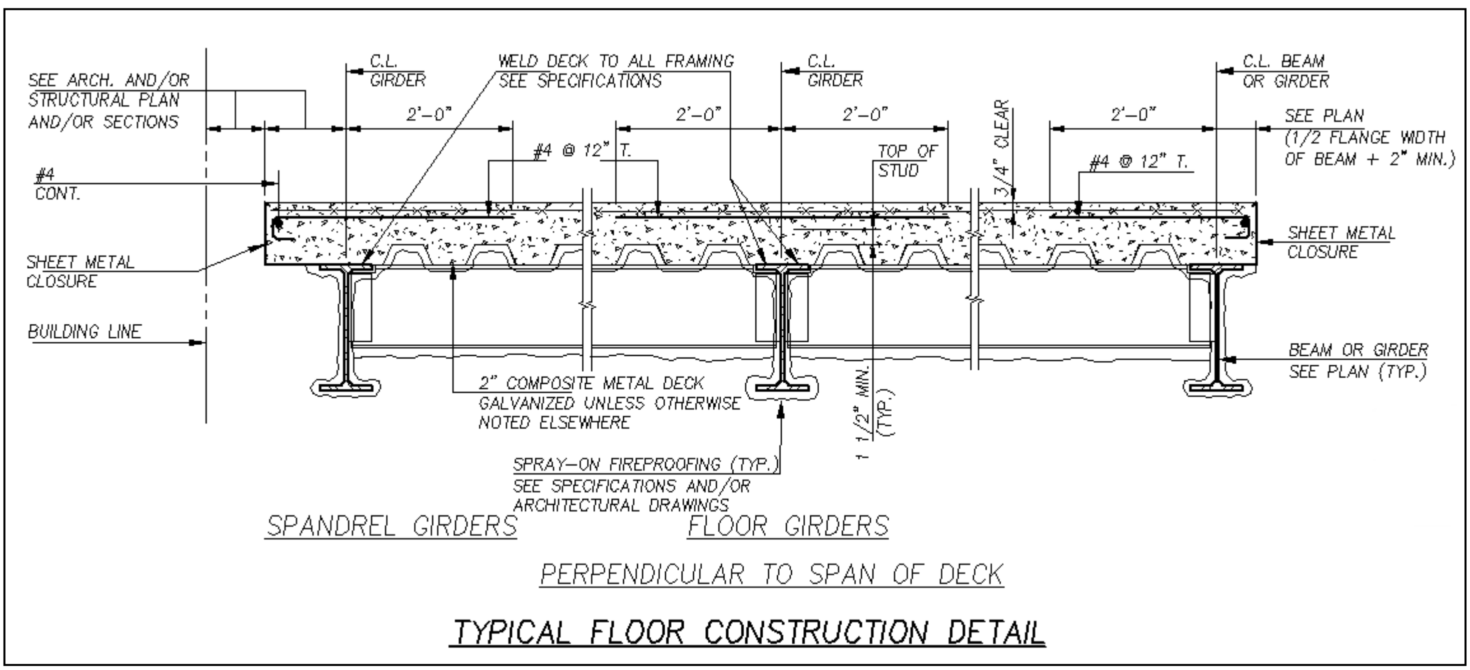
Floor System

The floor system employed in the 40 Gold Street building design is primarily slab on composite 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" bars, 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 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.

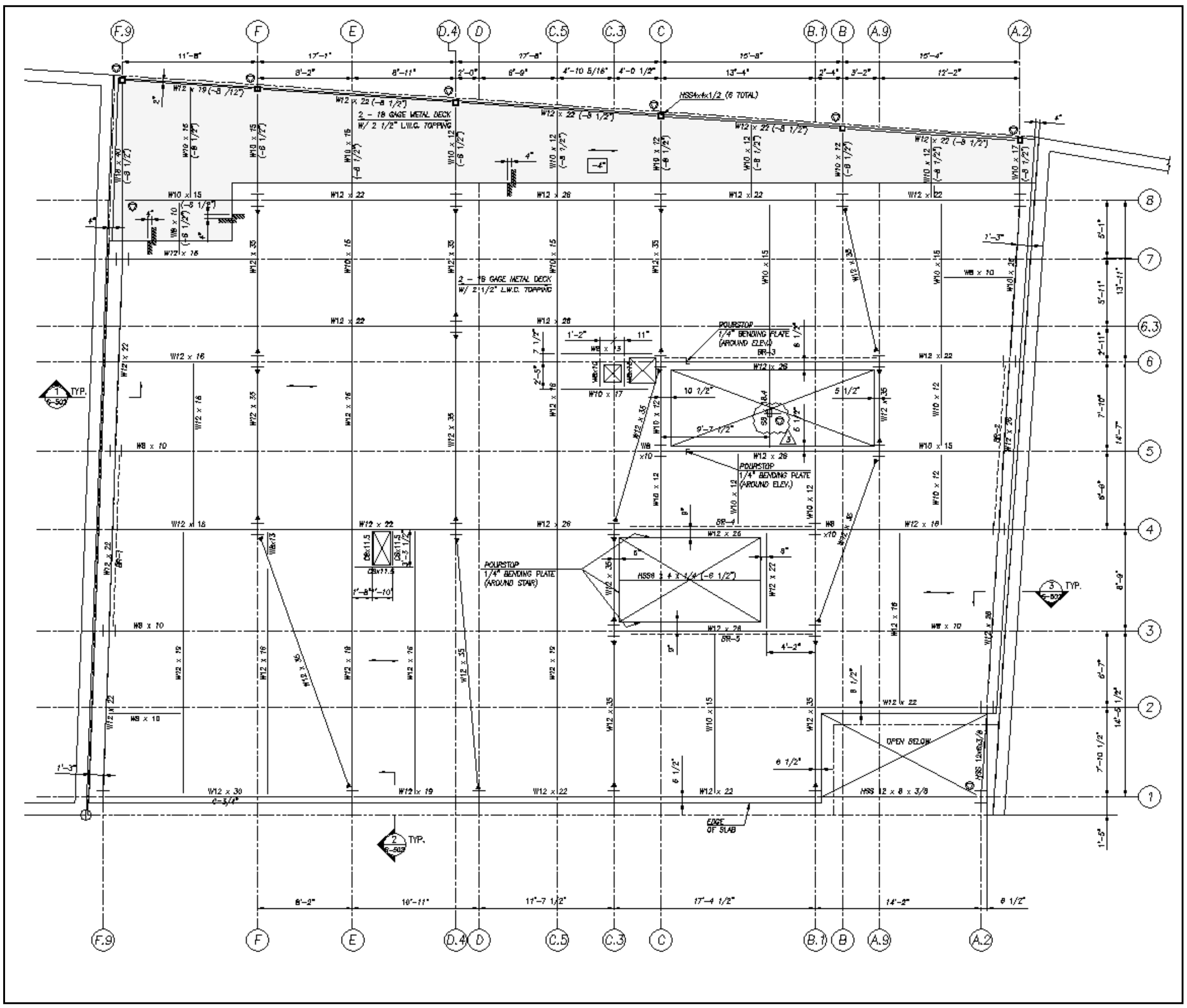
The cellar floor consists of a two-way 8" slab on grade with #5 @ 12" on center, top and bottom each way. The cellar slab rests on a 6" layer of crushed stone. More importantly, the cellar floor which is sub grade required a change in elevation as a consequence of closely surrounding structures and foundations. 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 is supported by a 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 span diagonally across the plan. 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

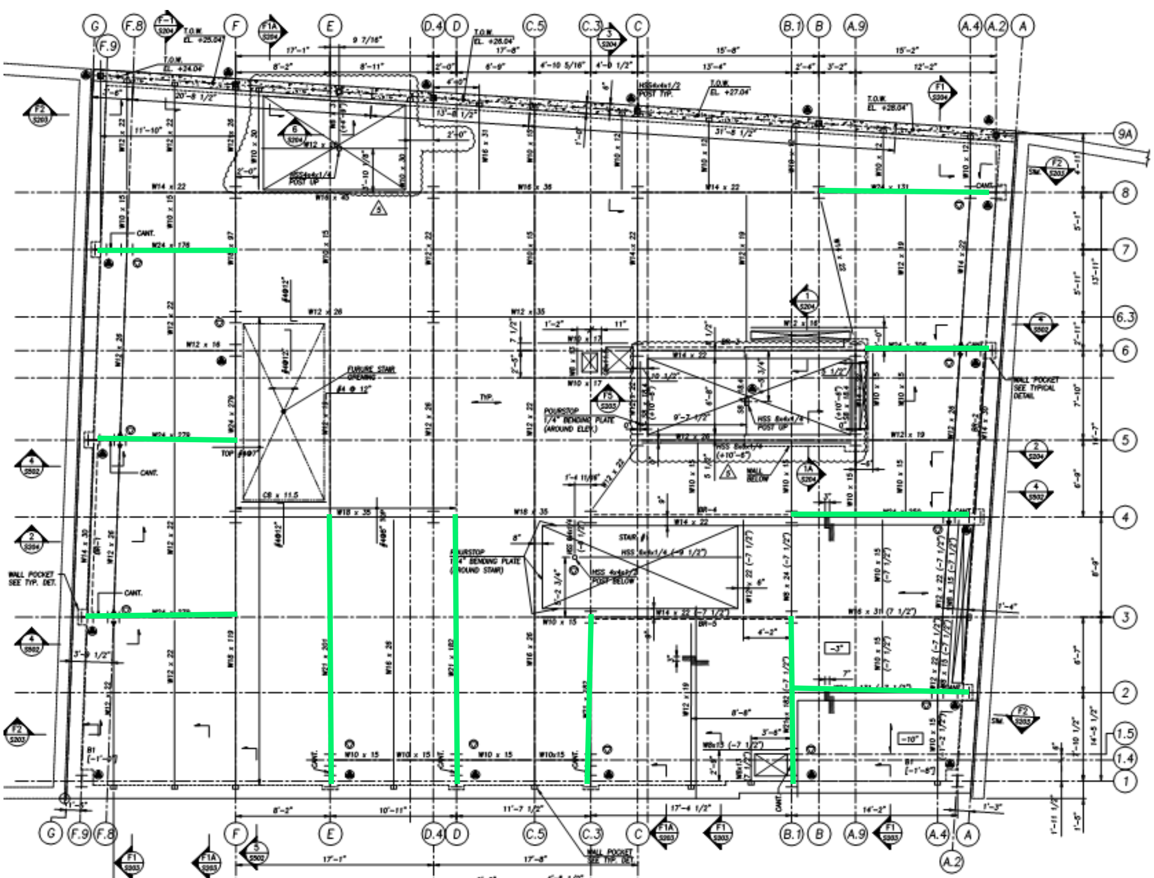
Gravity System

The gravity loads are resisted by a 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 nearly constant from level to level, but a slight reduction in size is observed near the top of the structure. The column splices are all located at 2' -6" above each finished floor. Almost all columns rise two floors. 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 large as W24x279's.

F-2

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

F-3



S-5

Highlighted Beams Cantilever outward 2 feet

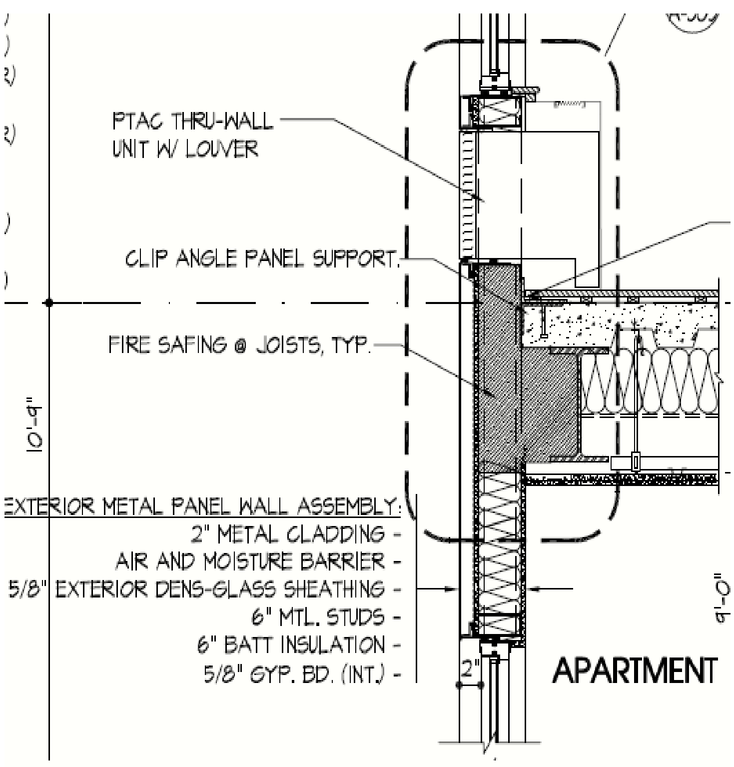
Sustainability

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 shade device.

Building Envelope

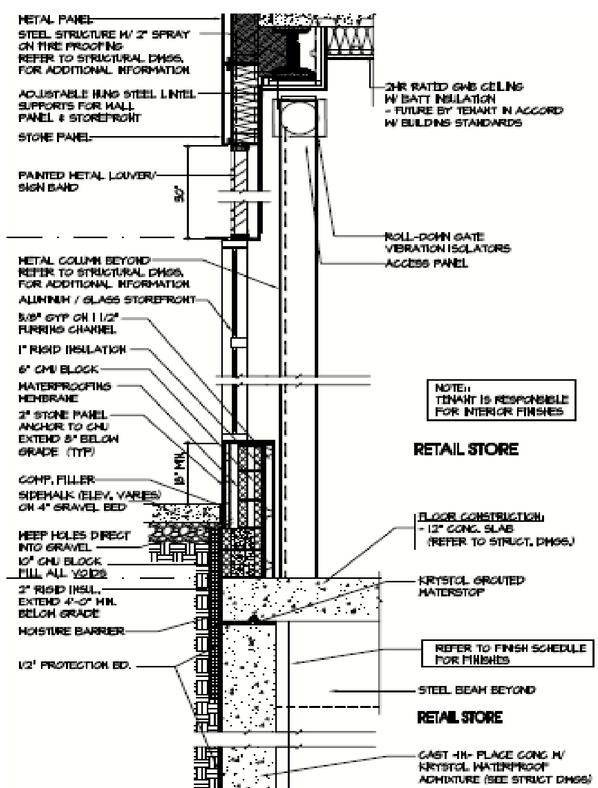
Floors 2-14 are enclosed by a basic non-bearing exterior metal panel wall assembly. The general composition of the wall shown in figure S-6 is 2" metal cladding (exterior), air and moisture barrier, 5/8" exterior dens-glass sheathing, 6" metal studs, 6" batting insulation, and 5/8" gypsum board (interior).

The sub grade spaces, also referred to as the cellar, are enclosed by a cast-in-place concrete wall. A detail of the enclosure can be seen in Figure S-7. Retail areas on the street level are enclosed by a large aluminum and glass storefront anchored to a basic CMU wall assembly which consists of 2" stone panel (exterior), waterproofing membrane, 6" CMU, 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-6

Typical Building Envelope for Residential Tower



S-7

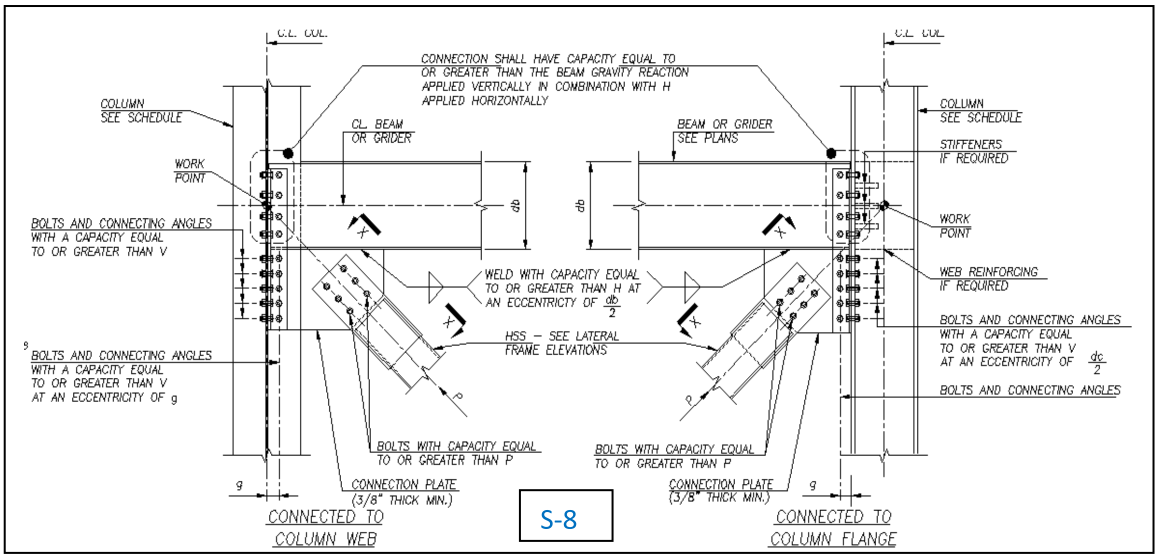
Typical Building Envelope for Ground Floor

Roof System

40 Gold Street features an ordinary flat roof, whose framing is 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 feature 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.

Lateral System

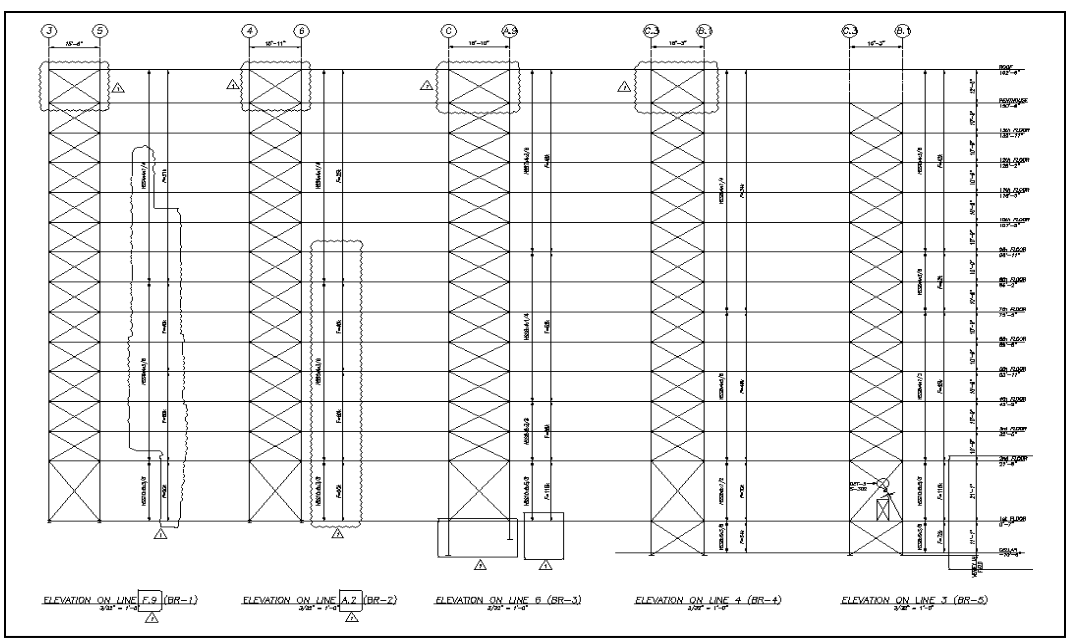
The lateral system of 40 Gold Street consists of 5 braced frames and 4 moment frames. Figure S-10 shows the moment frames, which span east to west across the building, in red. The braced frames are shown in green. The moment frames are skewed since several of the building's footings are offset to avoid disturbing the adjacent structural foundations. The moment frame along column line A.9 is skewed due to architectural constraints. Figure S-8 illustrates the typical connections and structural members that form the braced frames, and figure S-9 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. 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 variable; the center of rigidity is located near the center of mass. In consequence, the potential for torsion effect due to seismic load is lessened.



S-8

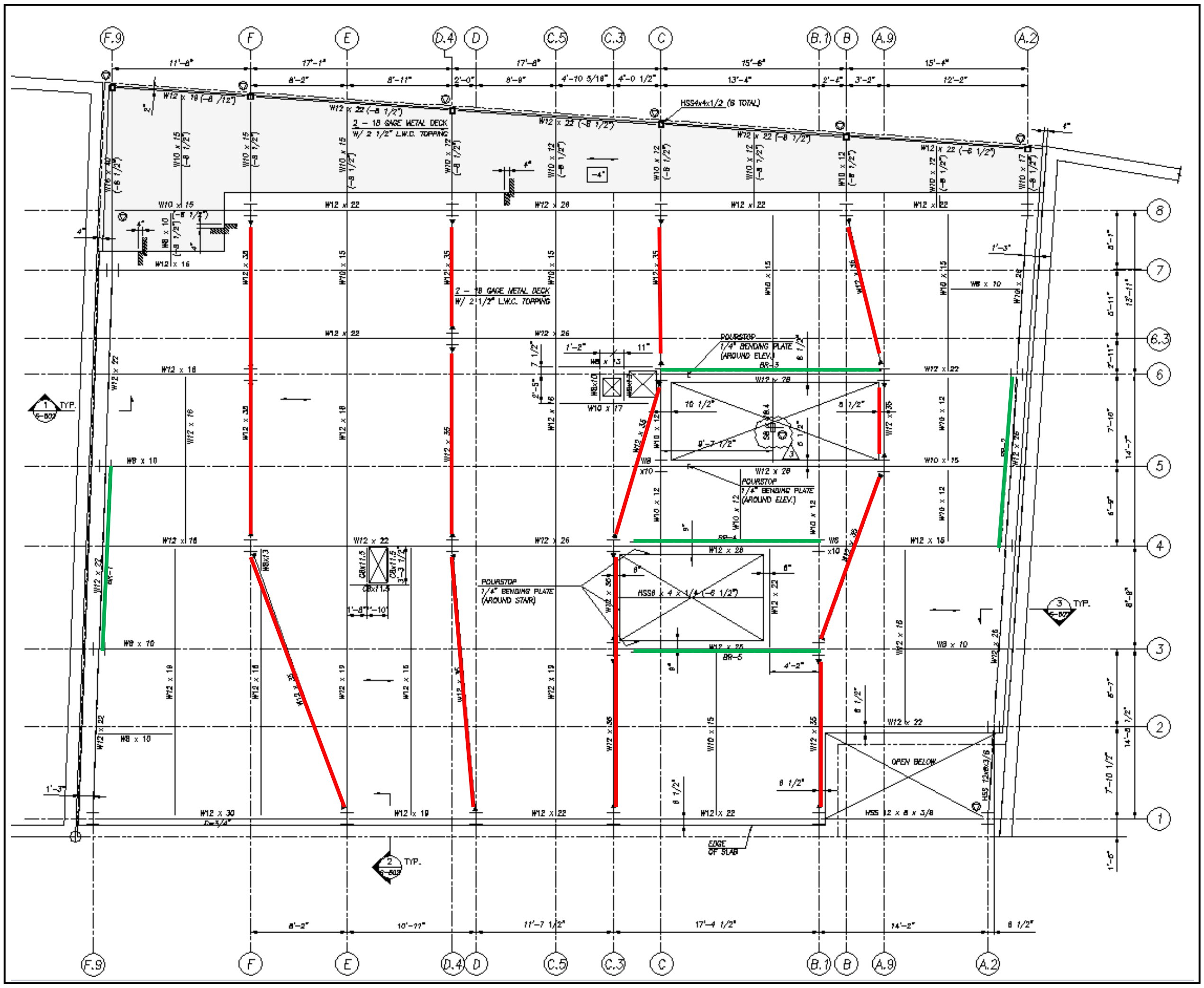
Figure S-6 details the typical brace frame.

Figure S-7 shows the five braced frames in elevation



S-9

LATERAL SYSTEM LAYOUT



S- 10

Moment Frames

Braced Frames

Codes, Design Standards:

- 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:

  American Society of Civil Engineers (ASCE), ASCE7-05

  Building Code
  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

Required Loads

Building Dead Loads were provided by the Structural Engineering Firm Severud Associates.

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

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Building live loads were determined by consulting ASCE 7. The actual design loads used by Severud Associates were verified.

Area	Actual Design Load	Thesis Design Load (ASCE 7-05)	Code/Table
Residential	40 psf	40 psf	ASCE7-05 Table 4-1
Retail	100 psf	100 psf	
Corridors	100 psf	100 psf	
Roof	60 psf	60 psf	
Terraces/Pedestrian	100 psf	100 psf	

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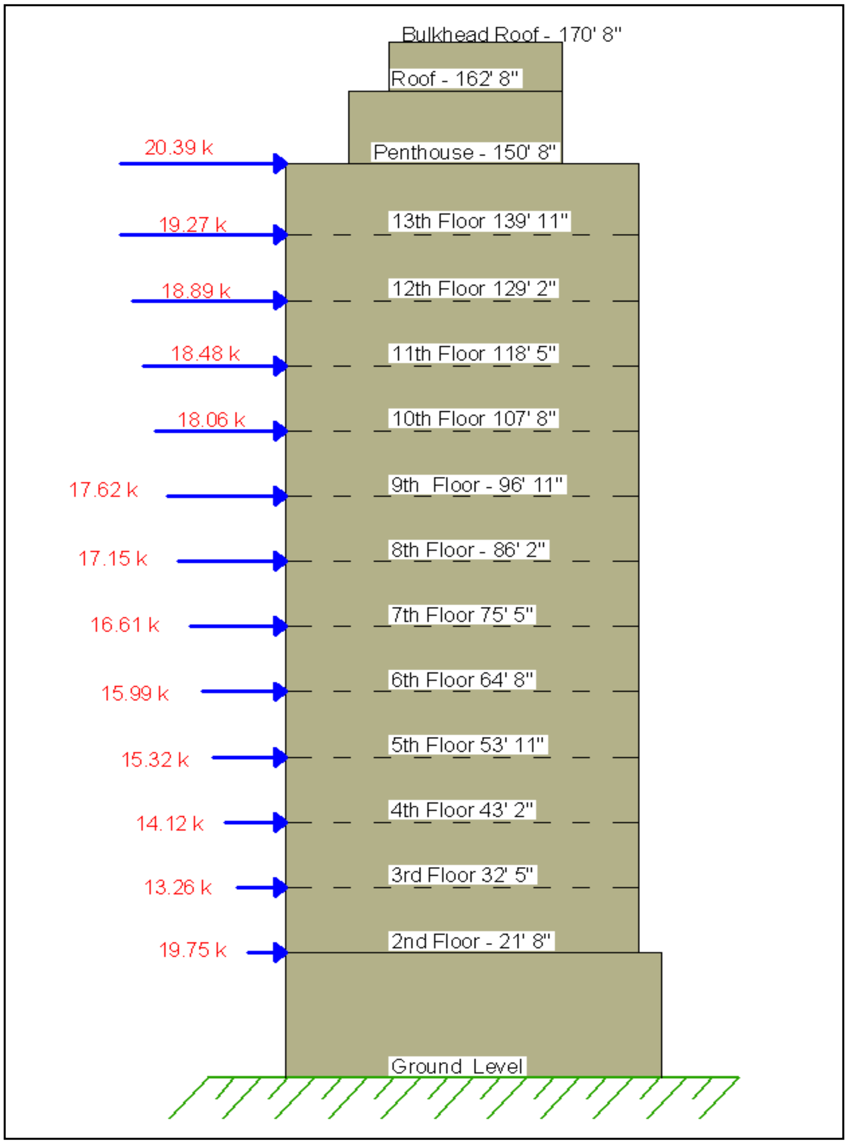
WIND LOAD CALCULTATIONS

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). The results of these wind calculations are illustrated in the following figures. Calculations can be viewed in their entirety in Appendix A. Figure **F-4** and **F-5** show the calculated story forces in the X and Y direction respectively.

East / West Wind Diagram (X Direction):



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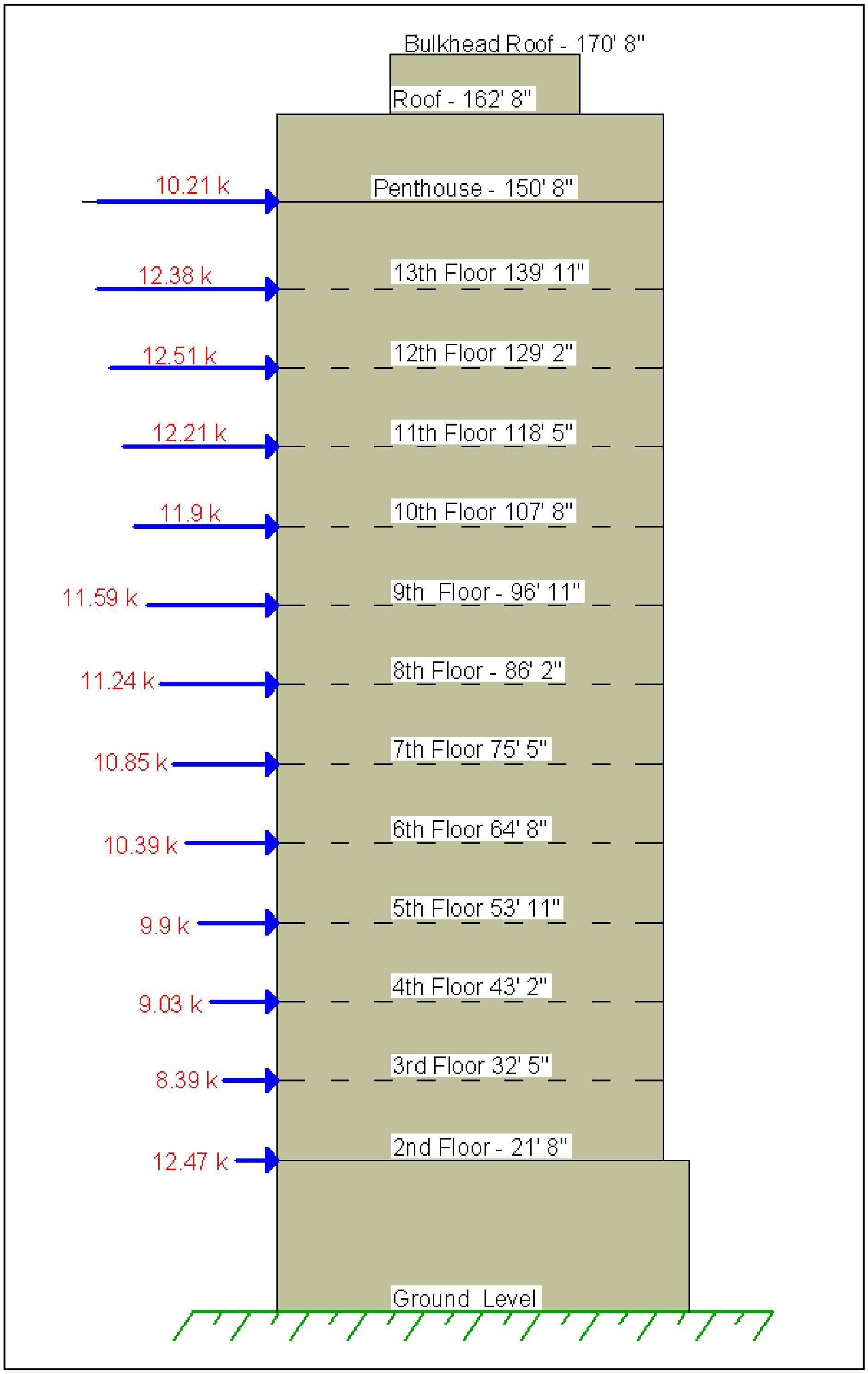
East / West Wind Diagram (X Direction):



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SEISMIC INTRO

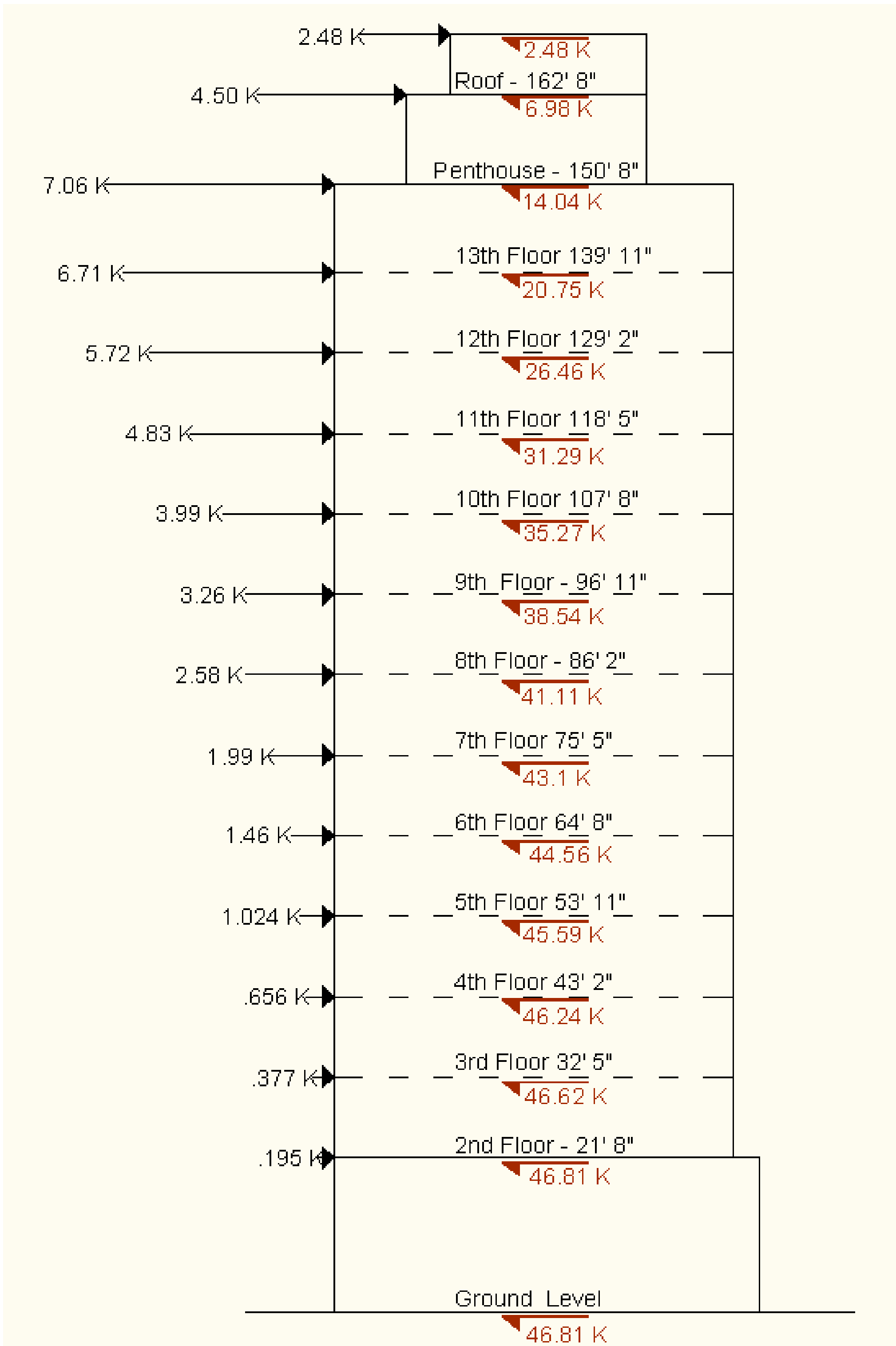
The following table T-3 and corresponding calculations found in Appendix B, were obtained in accordance with IBC 2006 Section 1613.1 and by Referencing Chapters 12 and 13 of ASCE7-05. The 40 Gold street building is a slender steel framed structure located in Manhattan, New York. To quickly summarize 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. Refer to Appendix B for all calculations and tables pertaining to the seismic load determination. Figure F-6 on the following page displays the seismic story forces.

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- 3

Seismic Diagram



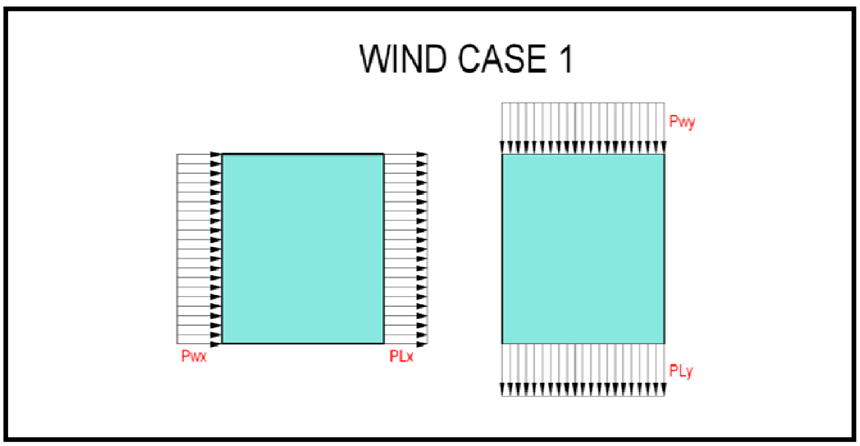
F- 6

LOAD CASES AND COMBINATIONS

In this report, strength, drift, story drift, and overturning moment checks were performed by analyzing the structure under the following ASCE7-05 (section 2.3.2) load combinations:

1. 1.4 (D + F)
2. 1.2 (D + F + T) + 1.6(L+H) + 0.5(L_r or S or R)
3. 1.2D + 1.6(L_r or S or R) + (L or .8W)
4. 1.2D + 1.6W + L + 0.5(L_r or S or R)
5. 1.2D 1.0E + L + 0.2S
6. .9D + 1.6W + 1.6H
7. .9D + 1.0E + 1.6H

Design wind pressures were applied according to the following 4 ASCE7-05 (figure 6.9) wind loading cases:



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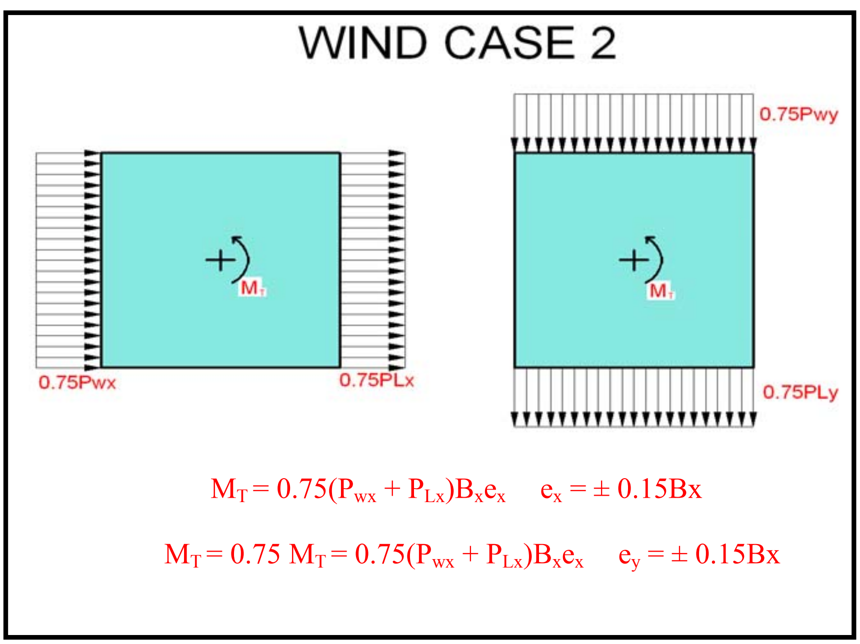
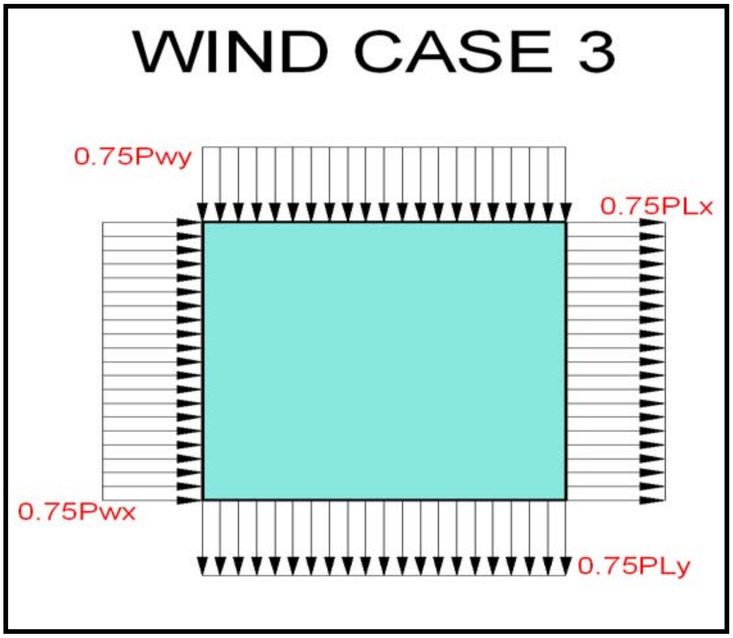
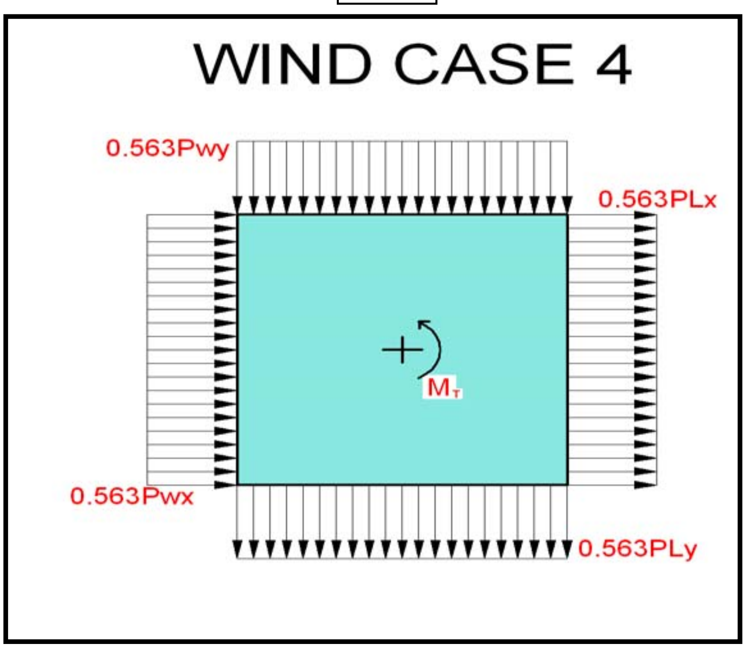


F- 8



F- 9

F- 10



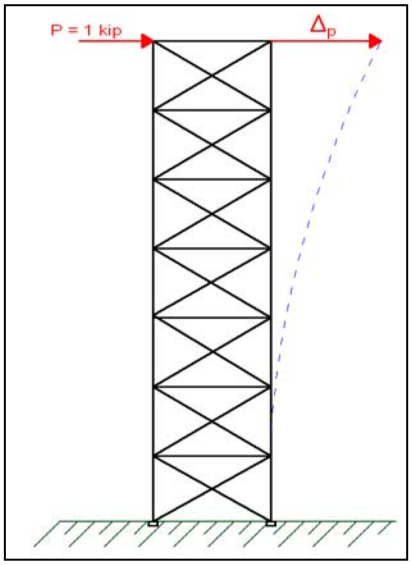
$$M_T = 0.563(P_{wx} + P_{Lx})B_x e_x + 0.563(P_w + P_{Lx})B_x e_x$$

$$e_x = \pm\, 0.15Bx \qquad e_y = \pm\, 0$$

PRELIMINARY LATERAL SYSTEM ANALYSIS

GOAL: Two approximate lateral force analysis methods were performed in order to acquire a better understanding of the lateral system in 40 Gold Street. Included in the preliminary results are approximated relative stiffness values, predicted lateral force distributions, and estimated locations for the center of rigidity (C.O.R.) and center of mass (C.O.M.). The primary purpose of these approximations is to serve as "expected" target values for comparison and verification of the 3D ETABS model output. Additionally, although the preliminary results are not exact, the values provide assistance when troubleshooting and interpreting the 3D model. Finally, the preliminary analysis results provide useful insight when determining the relevance of each load combination and case.

METHOD ONE: (See Appendix C for All supporting calculations)
Governed by the inverse relationship between displacement (Δ) and stiffness (k), method one calculates relative stiffness values with the aid of a 2D modeling procedure. First, separate SAP2000 2D models were created for each moment frame and braced frame. As shown in figure **F-11** below, a 1 kip lateral load was assigned to the top left joint of each frame. The horizontal translational displacement (U_1) of the top right joint was obtained for each frame under the 1 kip loading. Stiffness values for each frame were determined using the inverse of displacement: **$K = P / \Delta_P$** where K is in kips/inch.



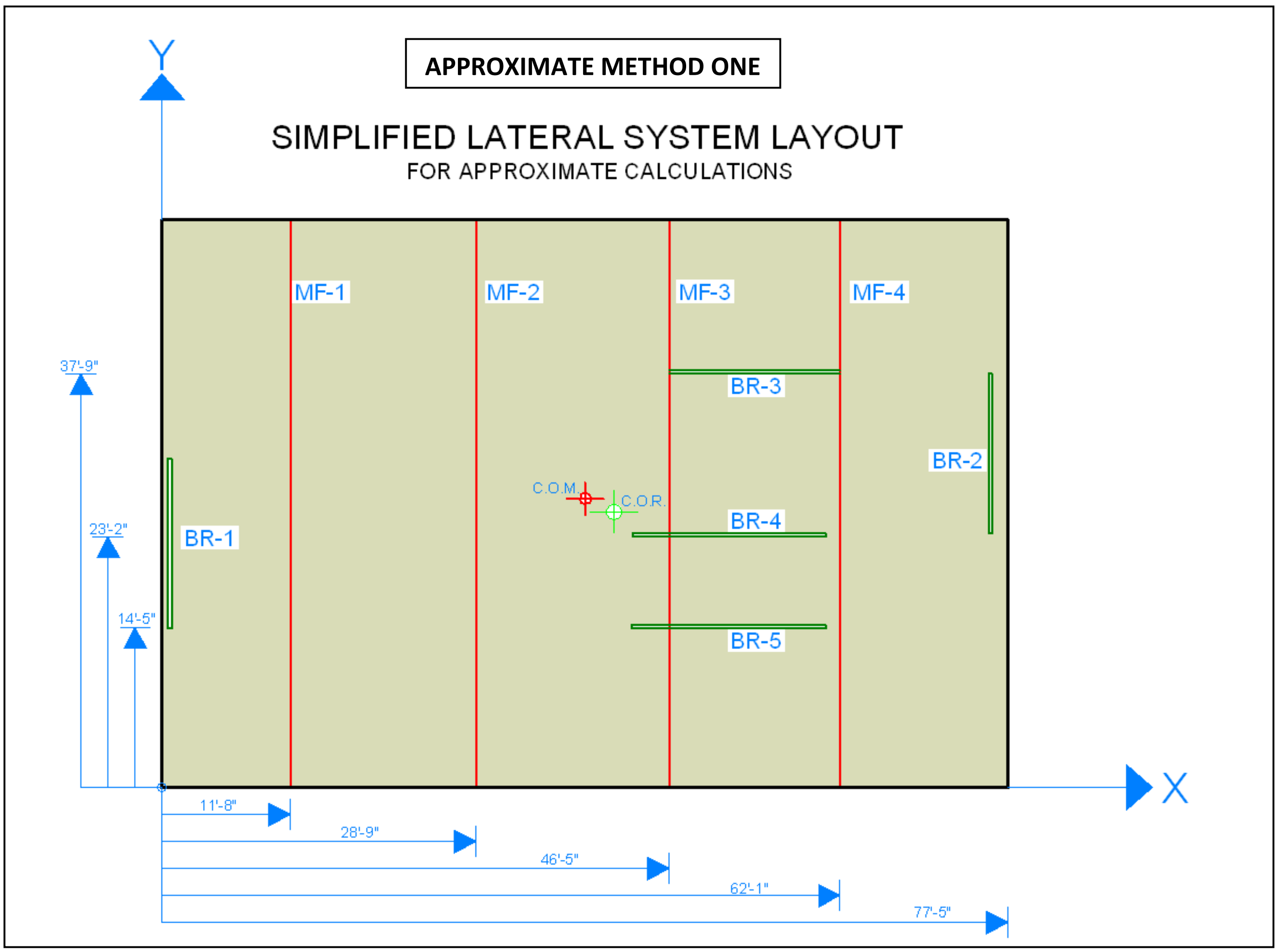
F- 11

For this procedure, the lateral system was modeled with a simplified geometry. As shown in figure **F-12**, the irregular geometries of the moment frames are not considered. Since the lateral elements only participate in resisting loads acting along a single axis parallel to its layout, modeling the moment frames as shown below greatly reduces the complexity of the calculations. Unfortunately, this introduces a source of error to the moment frame stiffness values, since each moment frame is now modeled entirely parallel to the Y direction. As discussed with Penn State Architectural Engineering faculty, this simplification is acceptable for approximate calculations and the error is expected to be minimal since moment frames are typically 10 times more flexible than brace frames of similar geometry.

As illustrated in figure **F-12**, the distance of each lateral element with respect to the specified origin (lower left corner) was determined. With these distances and the relative stiffness values, the center of rigidity was determined.

Since the building mass is evenly distributed, the center of mass was assumed equivalent to the geometric center. The X and Y eccentricities were then determined based on the relative distances between the center of rigidity and center of mass. Approximate lateral load distribution was determined by accounting for both the direct shear forces and torsion effects.

- Direct Shear Force Equation: $F_{ix\ direct} = (k_{ix} * P) / (\sum K_{ix})$ and $F_{iy\ direct} = (k_{iy} * P) / (\sum K_{iy})$
- Torsion Forces: $F_{i\ torsion} = (k_i * d_i * P_x * e_y) / (\sum k_i d_i^2)$
- Total Forces: $F_i = F_{idirect} \pm F_{itorsion}$



F- 12

Additional Comments Regarding Computer Modeling: The material and structural shapes were all properly defined and assigned. As instructed, the braces were modeled as pinned by releasing the moments in the 2-2 and 3-3 directions. Also, base supports were assigned restraints for all 6 degrees of freedom, and the beams and columns are all modeled in the correct orientation.

Summary of Method One Calculations:

METHOD ONE - CALCULATION SUMMARY									
APPROXIMATE FRAME STIFFNESS AND CENTER OF RIGIDTY CALCULATION									CENTER OF RIGIDITY COORDINATES
Story	Load Direction	Frame	Load, P (kips)	Displacement, Δ (inches)	Stiffness, Kiy (kips / in)	Stiffness Factor	Xi (in)	KiyXi (kips)	X
1		BR-1	1	0.3177	3.1476	0.134	0	0	X = ∑KiyXi / ∑Kiy = 495.73 " = 41.31'
		BR-2	1	0.2807	3.5625	0.1518	929	3309.5625	
	Y	MF-1	1	0.304	3.289	0.1401	140	460.46	
		MF-2	1	0.3269	3.059	0.1303	345	1055.355	
		MF-3	1	0.1985	5.037	0.2146	557	2805.609	
		MF-4	1	0.1861	5.373	0.2289	745	4002.885	
					∑Kiy = 23.4681			∑KiyXi = 11,633.8715	
Story	Load Direction	Frame	Load, P (kips)	Displacement, Δ (inches)	Stiffness, Kix (kips/in)		Yi (in)	KixYi (kips)	Y
1		BR-3	1	0.1674	5.9737	0.3507	453	2706.0861	Y = ∑KixYi / ∑Kix = 303.98" = 25.33'
	X	BR-4	1	0.1879	5.3219	0.312	278	1479.4882	
		BR-5	1	0.1742	5.74	0.3369	173	993.02	
					∑Kiy = 17.0356			∑KiyXi = 5178.5943	

T-4

Center of Mass (x,y) : (38’-8” , 26’-0”) **Center of Rigidity (x,y):** (41’-4”, 25’-3”) **Eccentricity (x,y):** (2’-8”,0’-7”)

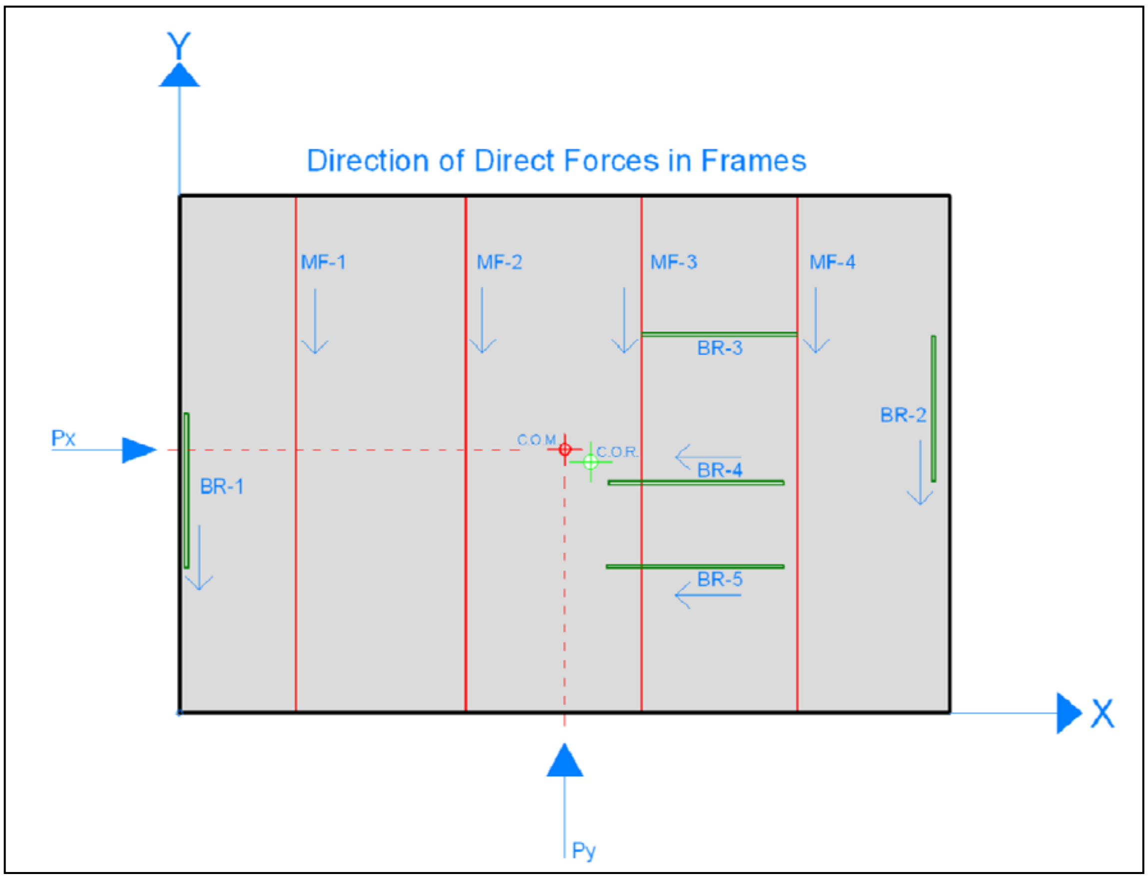
METHOD ONE - CALCULATION SUMMARY					
DIRECT SHEAR FORCES					
Story	Direction	Frame	Kix	Load	Fix = Kix * Px / ∑Kix
1	X	BR-3	5.9737	Px	.3506Px
		BR-4	5.3219	Px	.3124Px
		BR-5	5.74	Px	.3370Px
			∑Kix = 17.0356		
Story	Direction	Frame	Kiy	Load	Fiy = Kiy*Py / Kiy
1	Y	BR-1	3.1476	Py	.134Py
		BR-2	3.5625	Py	.1517Py
		MF-1	3.289	Py	.14Py
		MF-2	3.059	Py	.1302Py
		MF-3	5.037	Py	.2144Py
		MF-4	5.373	Py	.2287Py
			∑Kiy =23.5681		

T- 5

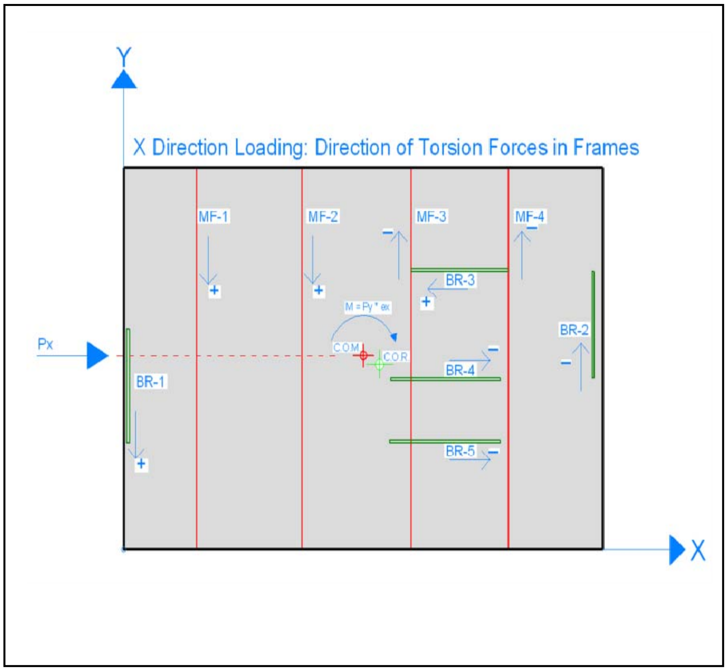
METHOD ONE - CALCULATION SUMMARY								
TORSIONAL FORCES								
Story	Direction	Frame	Kix (kips / in)	di (ft)	Load	ey (ft)	J = ∑kidi^2 (kips/in) ft^2	fit = kidiPxey / J (kips)
1	X	BR-3	5.9737	12.41667	Px	0.58333	17,444.31	.00248Px
		BR-4	5.3219	2.0833				.000371Px
		BR-5	5.74	10.8333				.002079Px
Story	Direction	Frame	Kiy (kips / in)	di (ft)	Load	ex (ft)	J = ∑kidi2 (kips/in) ft2	fit = kidiPyex / J (kips)
1	Y	BR-1	3.1476	41.333	Py	2.66667	17,444.31	.0198Py
		BR-2	3.5625	36.0833				.0197Py
		MF-1	3.289	29.6667				.0149Py
		MF-2	3.059	12.5833				.0149Py
		MF-3	5.037	5.08333				.00391Py
		MF-4	5.373	20.75				.017Py

T- 6

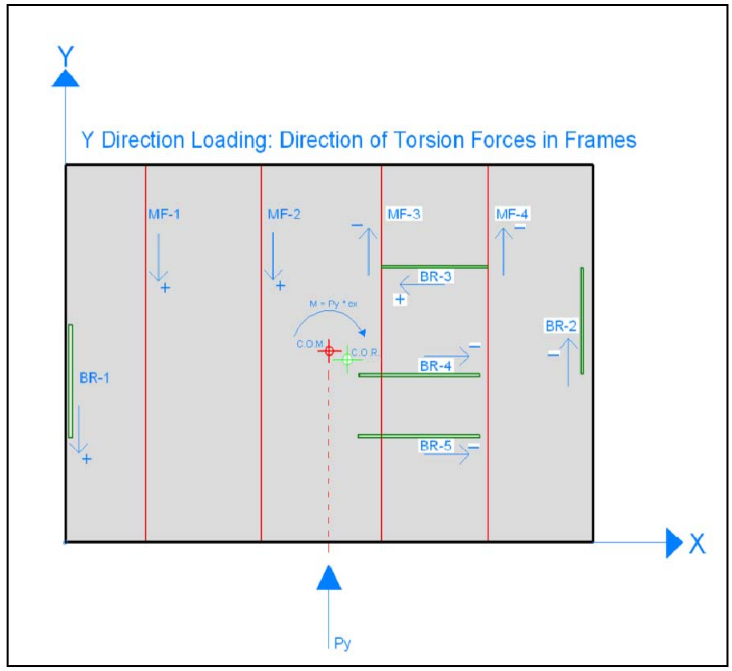
In order to properly calculate the total force distributed to each frame, both direct and torsion forces must be combined. However, as the following diagrams F-13 through F-15 demonstrate, torsion forces do not always act in the same direction as the direct forces. With the loads Px and Py applied at the center of mass, the structure pivots around the center of rigidity. Torsion forces are labeled + or – signifying how the torsion forces contribute to the total force in each frame.



F- 13



F- 14



F- 15

Method One Results - Total Forces:

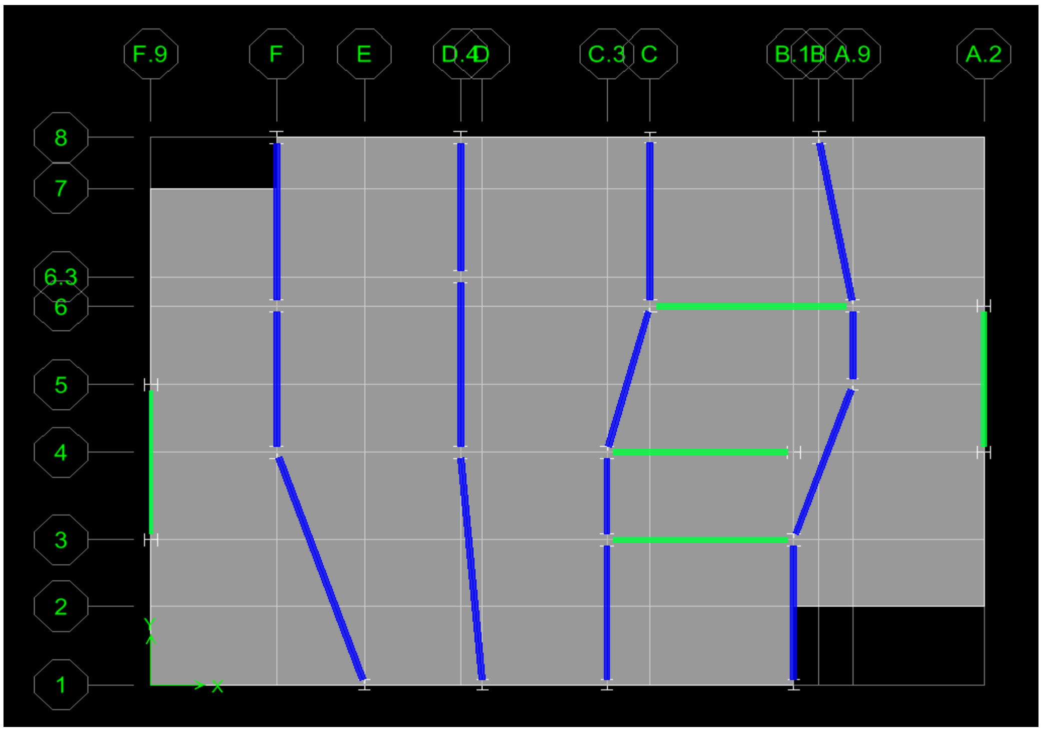
Based on the above diagrams and calculated direct and torsion forces, total forces were calculated and shown below in table T-7. Please see Appendix C for all supporting calculations.

Method One - Total Force Calculation						
First Story						
Direction	Frame	Direct Force		Torsional Force		Total Force
X	BR-3	.3506Px	+	.00248Px	=	.3531Px
	BR-4	.3124Px	-	.000371Px	=	.312Px
	BR-5	.3370Px	-	.002079Px	=	.3349Px
Y	BR-1	.134Py	+	.0198Py	=	.1538Py
	BR-2	.1517Py	-	.0197Py	=	.132Py
	MF-1	.140Py	+	.0149Py	=	.1549Py
	MF-2	.1302Py	+	.0149Py	=	.1451Py
	MF-3	.214Py	-	0.00391Py	=	.2105Py
	MF-4	.229Py	-	.017Py	=	.2117Py

T - 7

METHOD TWO:
A second, more exhaustive analysis was completed to provide a better approximation of relative stiffness values and the center of rigidity location. Using ETABS, a 3D model of the lateral system was produced which is shown in plan view in figure **F-16** below. In order to generate accurate results, the 3D model was created to be a near exact representation of the actual lateral system. Unlike method one, no simplifications to the moment frame geometries were made. However, the perimeter brace frames (BR-1 and BR-2) were modeled slightly rotated from their actual orientation in order to align with the Y axis.

PLAN VIEW OF 3D ETABS MODEL



F- 16

Shown in tables **T-8** through **T-33**, relative stiffness values were determined by analyzing 3D model output pertaining to lateral force distribution. First, 1000 kip loads were applied to the roof level center of mass, in both the X and Y direction. For each story, section cuts were used to determine the forces distributed to each lateral frame. By using rigid diaphragm modeling, the distributions of lateral forces are directly proportional to the relative stiffness of the resisting lateral elements. Knowing the relative stiffness values, the first story center of rigidity (see table **T-34**) was approximated by determining each lateral frame's distance from the specified origin and then applying the equations:

$$\overline{X} = \frac{\sum K_{iy} * X_i}{\sum K_{iy}} \qquad \overline{Y} = \frac{\sum K_{ix} * Y_i}{\sum K_{ix}}$$

Method Two – Relative Stiffness Values (X Direction):

STORY 1: 1000 Kip X Direction Loading		
LATERAL ELEMENT	Total Horizontal Force per Frame (kips)	Percent Fraction of Total Story Shear (%)
BR-3	-452.8475	45.28%
BR-4	-195.9516	19.95%
BR-5	-318.5722	31.86%
Column 1-E	-1.0534	-
Column 1-D.4	-0.7113	-
Column 1-C.3	-0.8635	-
Column 1-B.1	-0.5435	-
Column 3-F.9	-3.6871	-
Column 4-F	-1.7063	-
Column 4-D.4	-1.4096	-
Column 4-A.2	-3.9948	-
Column 5-F.9	-3.7377	-
Column 5-A.2	-1.5105	-
Column 6-F	-1.5972	-
Column 6-A.2	-4.0523	-
Column 6.3-D.4	-1.749	-
Column 8-F.9	-1.621	-
Column 8-D.4	-1.6208	-
Column 8-C	-0.9674	-
Column 8-B	-1.5461	-
Total Column Shear	-32.3715	3.24%
Total Story Shear	**-999.7428 ≈ 1000**	

T - 8

T- 9

STORY 2: 1000 Kip X Direction Loading		
LATERAL ELEMENT	Total Horizontal Force per Frame (kips)	Percent Fraction of Total Story Shear (%)
BR-3	-423.7597	42.38%
BR-4	-270.5714	27.06%
BR-5	-291.4904	29.15%
Total Column Shear	-14.1689	14.17%
Total Story Shear	-999.9904 ≈ -1000	

T-10

STORY 3: 1000 Kip X Direction Loading		
LATERAL ELEMENT	Total Horizontal Force per Frame (kips)	Percent Fraction of Total Story Shear (%)
BR-3	-460.2514	46.03%
BR-4	-256.7392	25.67%
BR-5	-280.2593	28.03%
Total Column Shear	-2.6833	0.27%
Total Story Shear	999.933 ≈ -1000	

STORY 4: 1000 Kip X Direction Loading		
LATERAL ELEMENT	Total Horizontal Force per Frame (kips)	Percent Fraction of Total Story Shear (%)
BR-3	-330.4552	33.05%
BR-4	-333.7631	33.38%
BR-5	-324.1224	32.41%
Total Column Shear	-11.654	1.17%
Total Story Shear	-999.9947 ≈ -1000	

STORY 5: 1000 Kip X Direction Loading		
LATERAL ELEMENT	Total Horizontal Force per Frame	Percent Fraction of Total Story
BR-3	-333.3764	33.38%
BR-4	-347.4635	34.75%
BR-5	-308.9626	30.89%
Total Column Shear	-10.0965	1.01%
Total Story Shear	-999.899 ≈ -1000	

T-11

T-12

STORY 6: 1000 Kip X Direction Loading		
LATERAL ELEMENT	Total Horizontal Force per Frame (kips)	Percent Fraction of Total Story Shear (%)
BR-3	-322.7331	32.27%
BR-4	-360.1843	36.02%
BR-5	-311.7727	31.18%
Total Column Shear	5.2993	0.53%
Total Story Shear	-999.989 ≈ -1000	

T-13

STORY 7: 1000 Kip X Direction Loading		
LATERAL ELEMENT	Total Horizontal Force per Frame (kips)	Percent Fraction of Total Story Shear (%)
BR-3	-370.2479	37.02%
BR-4	-245.5232	24.60%
BR-5	-368.1966	36.82%
Total Column Shear	-16.0321	1.60%
Total Story Shear	-999.9998 ≈ -1000	

T-14

STORY 8: 1000 Kip X Direction Loading		
LATERAL ELEMENT	Total Horizontal Force per Frame (kips)	Percent Fraction of Total Story Shear (%)
BR-3	-357.1115	35.71%
BR-4	-243.0985	24.31%
BR-5	-388.8792	38.89%
Total Column Shear	10.879	1.09%
Total Story Shear	-999.9682 ≈ -1000	

T-15

STORY 9: 1000 Kip X Direction Loading		
LATERAL ELEMENT	Total Horizontal Force per Frame (kips)	Percent Fraction of Total Story Shear (%)
BR-3	-417.1654	41.72%
BR-4	-221.9606	22.20%
BR-5	-343.2399	34.32%
Total Column Shear	17.6023	1.76%
Total Story Shear	-999.9682 ≈ -1000	

T-16

STORY 10: 1000 Kip X Direction Loading		
LATERAL ELEMENT	Total Horizontal Force per Frame (kips)	Percent Fraction of Total Story Shear (%)
BR-3	-411.0836	41.12%
BR-4	-210.8523	21.09%
BR-5	-362.0056	36.20%
Total Column Shear	16.0579	1.61%
Total Story Shear	-999.9994 ≈ -1000	

T-17

STORY 11: 1000 Kip X Direction Loading		
LATERAL ELEMENT	Total Horizontal Force per Frame (kips)	Percent Fraction of Total Story Shear (%)
BR-3	-401.626	40.16%
BR-4	-206.8532	20.69%
BR-5	-369.8931	36.99%
Total Column Shear	21.5988	2.26%
Total Story Shear	-999.9711 ≈ -1000	

T-18

STORY 12: 1000 Kip X Direction Loading		
LATERAL ELEMENT	Total Horizontal Force per Frame (kips)	Percent Fraction of Total Story Shear (%)
BR-3	-404.2599	40.43%
BR-4	-207.811	20.78%
BR-5	-370.6444	37.06%
Total Column Shear	17.2549	1.73%
Total Story Shear	-999.9702 ≈ -1000	

T-19

STORY 13: 1000 Kip X Direction Loading		
LATERAL ELEMENT	Total Horizontal Force per Frame (kips)	Percent Fraction of Total Story Shear (%)
BR-3	-389.6974	38.97%
BR-4	-211.3553	21.14%
BR-5	-374.4777	37.45%
Total Column Shear	24.4664	2.45%
Total Story Shear	-999.9968 ≈ -1000	

T-20

Method Two: Relative Stiffness Values (Y Direction):

STORY 1: 1000 Kip Y Direction Loading		
LATERAL ELEMENT	Total Horizontal Force per Frame (kips)	Percent Fraction of Total Story Shear (%)
BR-1	**-394.3177**	39.43%
BR-2	**-375.9383**	37.60%
MF-1		
Column 8-F	-11.6801	
Column 6-F	-15.6254	
Column 4-F	-13.8805	
Column 1-E	-7.865	
Total Sum	**-49.051**	4.91%
MF-2		
Column 8-D.4	-12.0052	
Column 6.3-D.4	-16.4687	
Column 4-D.4	-13.3399	
Column 1-D	-7.1679	
Total Sum	**-48.9817**	4.90%
MF-3		
Column 8-C	-9.3883	
Column 6-C	-16.0873	
Column 4-C.3	-15.4552	
Column 3 C.3	-17.0782	
Column 1-C.3	-9.3978	
Total Sum	**-67.4068**	6.74%
MF-4		
Column 8-B	-11.6991	
Column 6-A.9	-16.235	
Column 5-A.9	-12.5676	
Column 3-B.1	-13.9377	
Column 1-B.1	-7.1374	
Total Sum	**-61.5768**	6.16%
Other Lateral Column (3-B.1	**2.3308**	
Total Story Shear	**-999.6031 ≈ -1000**	

T-21

STORY 2: 1000 Kip Y Direction Loading		
LATERAL ELEMENT	Total Horizontal Force per Frame (kips)	Percent Fraction of Total Story Shear (%)
BR-1	-357.2846	35.73%
BR-2	-329.9131	32.91%
MF-1	-60.1366	6.14%
MF-2	-61.8115	6.18%
MF-3	-96.667	9.67%
MF-4	-93.6168	9.36%
Total Story Shear	**-999.4296 ≈ -1000**	

T-22

STORY 3: 1000 Kip Y Direction Loading		
LATERAL ELEMENT	Total Horizontal Force per Frame (kips)	Percent Fraction of Total Story Shear (%)
BR-1	-323.7154	32.37%
BR-2	-287.4911	28.75%
MF-1	-72.8961	7.29%
MF-2	-77.7233	7.77%
MF-3	-122.0524	12.21%
MF-4	-115.4226	11.54%
Total Story Shear	-999.3009 ≈ -1000	

T-23

STORY 4: 1000 Kip Y Direction Loading		
LATERAL ELEMENT	Total Horizontal Force per Frame (kips)	Percent Fraction of Total Story Shear (%)
BR-1	-303.0885	30.31%
BR-2	-267.7516	26.78%
MF-1	-83.7577	8.38%
MF-2	-89.3822	8.94%
MF-3	-126.5951	12.66%
MF-4	-128.763	12.88%
Total Story Shear	-999.3381 ≈ -1000	

T-24

STORY 5: 1000 Kip Y Direction Loading		
LATERAL ELEMENT	Total Horizontal Force per Frame (kips)	Percent Fraction of Total Story Shear (%)
BR-1	-252.7339	25.27%
BR-2	-212.0862	21.21%
MF-1	-103.9855	10.40%
MF-2	-112.2172	11.22%
MF-3	-158.7852	15.88%
MF-4	-159.3664	15.94%
Total Story Shear	-999.1744 ≈ -1000	

T-25

STORY 6: 1000 Kip Y Direction Loading		
LATERAL ELEMENT	Total Horizontal Force per Frame (kips)	Percent Fraction of Total Story Shear (%)
BR-1	-225.4545	22.55%
BR-2	-183.4697	18.35%
MF-1	-115.406	11.54%
MF-2	-120.09	12.01%
MF-3	-176.4774	17.65%
MF-4	-178.1965	17.82%
Total Story Shear	-999.0941 ≈ -1000	

T-26

STORY 7: 1000 Kip Y Direction Loading		
LATERAL ELEMENT	Total Horizontal Force per Frame (kips)	Percent Fraction of Total Story Shear (%)
BR-1	-180.4632	18.05%
BR-2	-140.6739	14.07%
MF-1	-133.2412	13.32%
MF-2	-137.8584	13.79%
MF-3	-203.5901	20.36%
MF-4	-203.2989	20.33%
Total Story Shear	-999.1257 ≈ -1000	

T-27

STORY 8: 1000 Kip Y Direction Loading		
LATERAL ELEMENT	Total Horizontal Force per Frame (kips)	Percent Fraction of Total Story Shear (%)
BR-1	-155.4693	15.55%
BR-2	-128.6807	12.68%
MF-1	-145.2901	14.53%
MF-2	-155.927	15.59%
MF-3	-208.0247	20.80%
MF-4	-205.5248	20.55%
Total Story Shear	-998.9166 ≈ -1000	

T-28

STORY 9: 1000 Kip Y Direction Loading		
LATERAL ELEMENT	Total Horizontal Force per Frame (kips)	Percent Fraction of Total Story Shear (%)
BR-1	-168.7346	16.87%
BR-2	-118.3451	11.83%
MF-1	-144.973	14.50%
MF-2	-111.2871	11.23%
MF-3	-224.7073	22.47%
MF-4	-224.7048	22.47%
Total Story Shear	-992.7519 ≈ -1000	

T-29

STORY 10: 1000 Kip Y Direction Loading		
LATERAL ELEMENT	Total Horizontal Force per Frame (kips)	Percent Fraction of Total Story Shear (%)
BR-1	-180.6402	18.06%
BR-2	-123.7505	12.38%
MF-1	-130.162	13.02%
MF-2	-119.5793	11.60%
MF-3	-221.5623	22.16%
MF-4	-223.071	22.31%
Total Story Shear	-998.7653 ≈ -1000	

T-30

STORY 11: 1000 Kip Y Direction Loading		
LATERAL ELEMENT	Total Horizontal Force per Frame (kips)	Percent Fraction of Total Story Shear (%)
BR-1	-165.4824	16.55%
BR-2	-106.6289	10.66%
MF-1	-136.8503	13.69%
MF-2	-124.069	12.41%
MF-3	-232.3705	23.24%
MF-4	-233.3079	23.33%
Total Story Shear	-998.709 ≈ -1000	

T-31

STORY 12: 1000 Kip Y Direction Loading		
LATERAL ELEMENT	Total Horizontal Force per Frame (kips)	Percent Fraction of Total Story Shear (%)
BR-1	-161.9929	16.20%
BR-2	-114.7709	11.48%
MF-1	-146.4793	14.65%
MF-2	-120.4197	12.04%
MF-3	-230.6843	23.07%
MF-4	-223.9039	22.39%
Total Story Shear	-998.251 ≈ -1000	

T-32

STORY 13: 1000 Kip Y Direction Loading		
LATERAL ELEMENT	Total Horizontal Force per Frame (kips)	Percent Fraction of Total Story Shear (%)
BR-1	-129.77284	12.98%
BR-2	-62.3307	6.23%
MF-1	-128.3839	12.84%
MF-2	-171.0502	17.11%
MF-3	-248.5248	24.85%
MF-4	-258.4535	25.85%
Total Story Shear	-998.516 ≈ -1000	

T-33

CENTER OF RIGIDITY CALCULATION FOR STORY 1						
METHOD 2						
Direction	Frame	Frame Load Applied Kips)	Total Horizontal Force per Frame (kips)	Percentage of Total Story Shear (%) K	Distance (Di) From Origin (ft.)	Center of Rigidity
X	BR-3	1000	-452.847	45.28%	37.75	Y = ∑kDi = 26.55'
	BR-4		-195.9516	19.60%	23.16667	
	BR-5		-318.5722	31.86%	15.4166667	
Y	BR-1	1000	-394.3177	39.43%	0	X = ∑kDi = 38.04'
	BR-2		-375.9383	37.59%	77.4166667	
	MF-1		-49.051	4.91%	11.666667	
	MF-2		-48.981	4.90%	28.75	
	MF-3		-67.4068	6.74%	46.416667	
	MF-4		-61.5768	6.16%	62.0833333	

T-34

PRELIMINARY ANALYSIS – RESPONSE AND CONCLUSIONS:

Method one involved an efficient but highly approximated set of calculations. Lateral system geometry was simplified and the center of mass was assumed equivalent to the buildings geometric center. According to results, the moment frames and brace frames have similar stiffness values which were unexpected. Typically, braced frames are 10 times stiffer than moment frames of similar geometry. As is evident in figure **F-16**, the moment frames are approximately 3 to 4 times wider than the brace frames. As a result of a wide stance and the simplification of moment frame geometry to linear layouts along the Y axis, the stiffness values are magnified.

In method two, the lateral system was modeled exactly according to the as built drawings. Relative stiffness values appear accurate and adhere to the generalization that braced frames are approximately 10 times stiffer than moment frames of similar geometry. As shown in table **T-21**, when applying a 1000 kip load at the roof level, the 1st story brace frames BR-1 and BR-2 account for 39.43% and 37.6% of the load respectively. As expected, the moment frames only accounted for 5 – 6 % of the load each. This force distribution is directly proportional to relative stiffness values and confirms that the brace frames are the dominant lateral system elements. Accuracy of method two results can be confirmed with several basic observations. First, tables **T-22** through **T-23** reveal that stiffness values for moment frames 3 and 4 exceed the stiffness values for moment frames 1 and 2. Not only are MF-3 and MF-4 more linear, but they are composed of 4 bays as opposed to 3 bays. With more structure, MF-3 and MF-4 have greater cross sectional area and larger moments of inertia, two characteristics that increase stiffness values.

At this stage, all observations and preliminary analysis results suggest torsion effects play a minimal role in controlling the design. Both preliminary methods yielded a center of mass and center of rigidity at nearly equivalent locations. Therefore, the eccentricity (moment arm) is small resulting in low torsion effects. Also, the rectangular shape and evenly spaced out lateral system (not concentrated at core) should significantly reduce torsion forces. Mathematically, torsion appears minimal when examining tables **T-8** through **T-33** of method 2 in which the total forces in each lateral frame amount to the corresponding story shear of 1000 kips. In addition, method 1 average torsion forces were reported as .0025Px in the X direction and .015Py in the Y direction. However, in order to thoroughly understand the lateral system, wind cases with torsion effects (2 and 4) are included in the final analysis.

Considering the 7 ASCE7-05 load combinations and 4 wind cases, the following 38 loading conditions were considered in the report:

1-3:	Load Combination 3,4,6: Wind Case 1, X Direction
4-6:	Load Combination 3,4,6: Wind Case 1, Y Direction
7-9:	Load Combination 3,4,6: Wind Case 2, X Direction, $+e_y$
10-12:	Load Combination 3,4,6: Wind Case 2, X Direction, $-e_y$
13-15:	Load Combination 3,4,6: Wind Case 2, Y Direction + e_y
16-18:	Load Combination 3,4,6: Wind Case 2, Y Direction – e_y
19-21:	Load combination 3,4,6: Wind Case 3
22-24:	Load Combination 3,4,6: Wind Case 4, $+e_x$ and $+e_y$
25-27:	Load Combination 3,4,6: Wind Case 4, $+e_x$ and $–e_y$
29-31:	Load Combination 3,4,6: Wind Case 4, $-e_x$ and $–e_y$
32-34:	Load Combination 3,4,6: Wind Case 4, $-e_x$ and + e_y
35:	Load Combination 5
36:	Load Combination 7

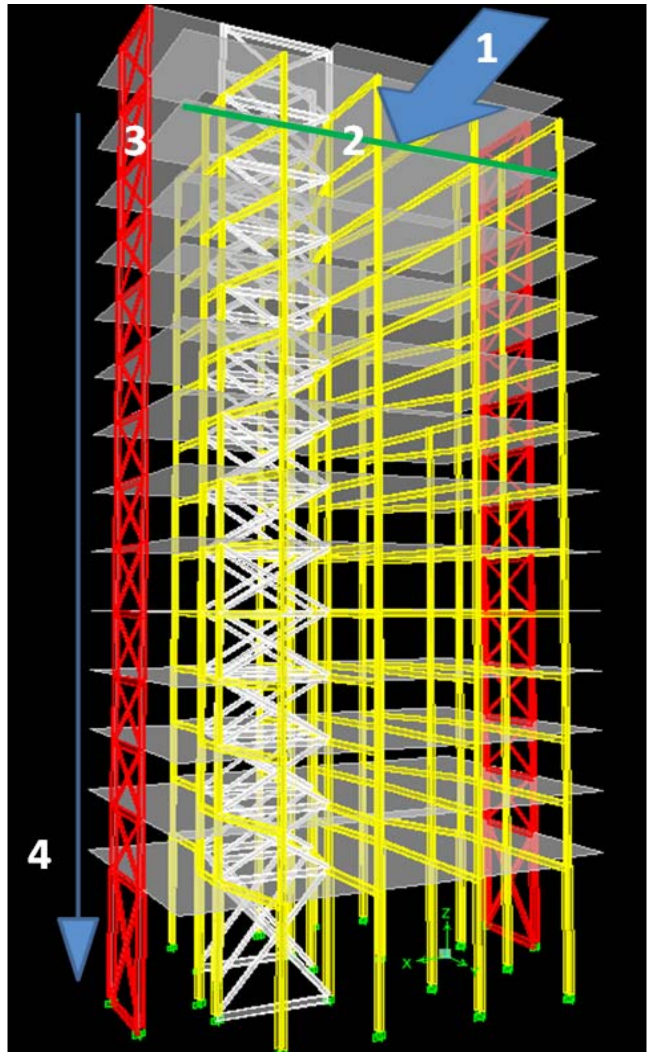
LATERAL LOAD PATH SHCEMATIC

Step One: Horizontal wind pressures act perpendicularly to the building envelope.

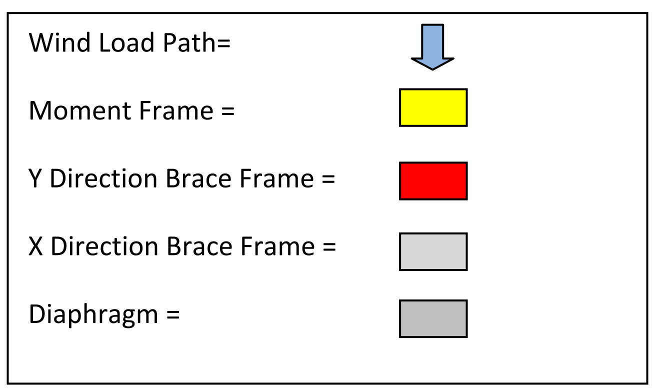
Step Two: Wind pressures are distributed based on tributary areas, from the building envelope to the connected horizontal stiff elements.

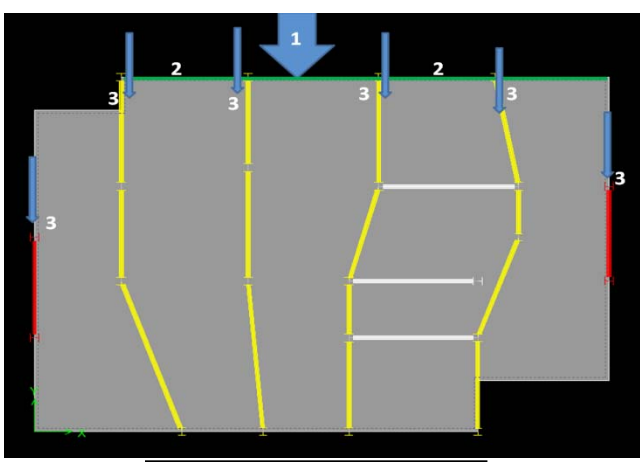
Step Three: In the case of 40 Gold Street, the floor structures are connected to the building envelope and collect the lateral load. Behaving as a rigid diaphragm, the floor structures transfer lateral loads to vertical bearing elements based on relative stiffness. 40 Gold street is designed with a lateral system comprised of 5 braced frames and 4 Moment frames. The frames only participate in resisting loads that act parallel to their axis of orientation. In the adjacent diagrams **F-17** and **F-18**, the red frames, are braced frames and resist the greatest percentage of lateral loads due to their high rigidity relative to the moment frames (yellow).

Step Four: Once loads reach the frames, they are transferred down to the supporting foundation. Constructed with rigidly connected joints that resist rotation, moment frames collect lateral forces and transfer them down a vertical plane down to the foundation. The braced frames have cross bracing providing a diagonal path to transfer loads down the frame and ultimately to the supporting foundation.



F-17 3D Schematic - Y Direction Loading





Plan View Schematic **F-18**

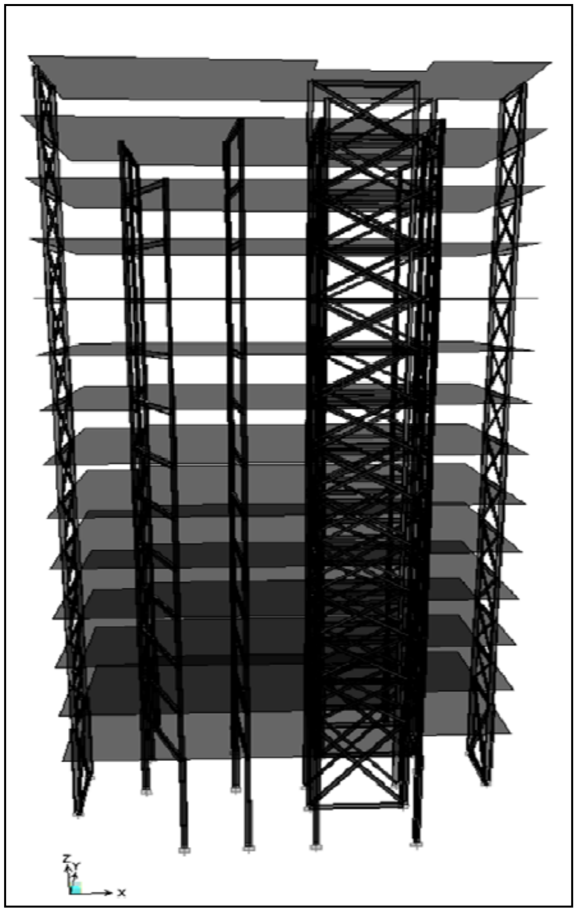
3D ETABS Model Analysis – Output Data

In the final stage of analysis, 3D model output for the 38 previously listed loading conditions were carefully examined. As shown in table T-35, ETABS computed the center of rigidity and center of mass for each story. Methods one and two yielded similar values verifying the output data.

During the modeling process, careful attention was given to the diaphragm modeling. Uniform dead loads calculated in technical report 1 (see appendix B) due to the weight of beams, slab, deck, and superimposed loads dead loads were applied to each diaphragm. In doing so, overturning moment and column strength checks could be easily completed with the aid of the 3D model.

A mass was assigned to each diaphragm. The unit conversion required mass = PSF / (12^3 * 32.2* 1000). After running analysis, the first mode period of vibration in the X, Y, and Z directions were recorded. (P-delta effects were not considered). Although the actual periods of vibration are unknown, the values below are reasonable suggesting there are no major modeling errors.

T_Y = 2.12 sec **T_Z = 2.03 sec** **T_X = 1.80 sec**



F-19

T-35

3D ETABS MODEL OUTPUT						
	CENTER OF MASS (in)		CENTER OF RIGIDITY (in)		Eccentricity (in)	
STORY	X	Y	X	Y	ex	ey
2	457.499	333.021	454.581	309.942	2.918	23.079
3	457.499	333.021	457.439	307.33	0.06	25.691
4	457.499	333.021	457.44	307.372	0.059	25.649
5	457.499	333.021	456.769	305.275	0.73	27.746
6	457.499	333.021	456.619	304.506	0.88	28.515
7	457.499	333.021	456.902	304.401	0.597	28.62
8	457.499	333.021	457.817	304.996	-0.318	28.025
9	457.499	333.021	459.697	305.3	-2.198	27.721
10	457.499	333.021	462.38	306.265	-4.881	26.756
11	457.499	333.021	465.181	306.802	-7.682	26.219
12	457.499	333.021	467.728	307.066	-10.229	25.955
13	457.499	333.021	469.65	307.174	-12.151	25.847
Penthouse	457.499	333.021	471.279	307.334	-13.78	25.687
Roof	445.973	292.035	471.073	309.181	-25.1	-17.146

Wind Drifts - Serviceability Check:

Confirmation of the lateral system requires serviceability checks. Using unfactored wind loads shown in figures F-4 and F-5, story drifts and total building drifts were recorded and examined for each of the four ASCE7-05 wind cases. Wind case 1 controlled in both the X and Y directions. Tables T-36 and T-37, show controlling story drift and total drift values which were compared to the allowable drift: $\Delta_{WIND} = H / 400$.

Actual Wind Drift (Unfactored Loads): N-S Direction							
CONTROLLING WIND CASE: ASCE7-05 Wind Case 1 - X Direction							
Story	Story Height (in.)	Story Drift (in.)	Allowable Story Drift $\Delta wind = H / 400$ (in.)	Serviceability Check Actual < Allowable	Total Drift (in.)	Allowable Total Drift $\Delta wind = H/400$ (in.)	Serviceability Check Actual < Allowable
Roof	1945	0.002762	0.36	OK	0.03306	4.8625	OK
Penthouse	1801	0.002874	0.3225	OK	0.030298	4.5015	OK
13	1672	0.002927	0.3225	OK	0.027424	4.18	OK
12	1543	0.002958	0.3225	OK	0.024497	3.8575	OK
11	1414	0.002959	0.3225	OK	0.021539	3.535	OK
10	1285	0.002923	0.3225	OK	0.01858	3.2125	OK
9	1156	0.002877	0.3225	OK	0.015657	2.89	OK
8	1027	0.002745	0.3225	OK	0.01278	2.5675	OK
7	898	0.002489	0.3225	OK	0.010035	2.245	OK
6	769	0.00225	0.3225	OK	0.007546	1.9225	OK
5	640	0.001949	0.3225	OK	0.005296	1.6	OK
4	511	0.001541	0.3225	OK	0.003347	1.2775	OK
3	382	0.0012	0.3225	OK	0.001806	0.955	OK
2	253	0.000606	0.6325	OK	0.000606	0.6325	OK

T-36

Actual Wind Drift (Unfactored Loads): E-W Direction							
CONROLLING WIND CASE: ASCE7-05 Wind Case 4, and eccentricity combination: + ex and - ey							
Story	Story Height (in.)	Story Drift (in.)	Allowable Story Drift $\Delta wind = H / 400$ (in.)	Serviceability Check Actual < Allowable	Total Drift (in.)	Allowable Total Drift $\Delta wind = H/400$ (in.)	Serviceability Check Actual < Allowable
Roof	1945	0.001967	0.36	OK	0.024354	4.8625	OK
Penthouse	1801	0.001902	0.3225	OK	0.022387	4.5015	OK
13	1672	0.001989	0.3225	OK	0.020485	4.18	OK
12	1543	0.002067	0.3225	OK	0.018496	3.8575	OK
11	1414	0.002134	0.3225	OK	0.016429	3.535	OK
10	1285	0.002156	0.3225	OK	0.014295	3.2125	OK
9	1156	0.002153	0.3225	OK	0.012139	2.89	OK
8	1027	0.001998	0.3225	OK	0.009986	2.5675	OK
7	898	0.001877	0.3225	OK	0.007988	2.245	OK
6	769	0.001706	0.3225	OK	0.006111	1.9225	OK
5	640	0.001505	0.3225	OK	0.004405	1.6	OK
4	511	0.001256	0.3225	OK	0.0029	1.2775	OK
3	382	0.00104	0.3225	OK	0.001644	0.955	OK
2	253	0.000604	0.6325	OK	0.000606	0.6325	OK

T-37

Seismic Story Drift- Stability Check:

3D output data for ASCE 7-05 Load Combinations 6 (1.2D + 1.0E + 1.0L) and 7 (.9D + 1.0E) were examined to verify seismic induced story drifts do not exceed the allowable .015hsx. Table T-38, displays the actual and allowable drift values proving the lateral system is properly designed. Therefore, it is fair to assume the 40 Gold Street structure will not sustain any permanent damage due to small or moderate seismic activity. More importantly, in the event of severe seismic activity, structural failure will be avoided; however, the seismic induced stresses will exceed the yield strength of various structural members resulting in inelastic deformation (permanent damage). Since the building has a low overall building weight and is located in New York City, an area of little seismic activity, seismic story drifts did not come close to exceeding the allowable drift. Please see Appendix B for seismic load calculations.

Governing Equation: $\Delta_{SEISMIC} = .015H_{SX}$

TABLE 12.12-1 ALLOWABLE STORY DRIFT, Δ_a[a,b]

Structure	Occupancy Category		
	I or II	III	IV
Structures, other than masonry shear wall structures, 4 stories or less with interior walls, partitions, ceilings and exterior wall systems that have been designed to accommodate the story drifts.	$0.025h_{sx}$[c]	$0.020h_{sx}$	$0.015h_{sx}$
Masonry cantilever shear wall structures [d]	$0.010h_{sx}$	$0.010h_{sx}$	$0.010h_{sx}$
Other masonry shear wall structures	$0.007h_{sx}$	$0.007h_{sx}$	$0.007h_{sx}$
All other structures	$0.020h_{sx}$	$\boxed{0.015h_{sx}}$	$0.010h_{sx}$

Actual and Allowable Seismic Story Drift:

Seismic Story Drifts (inches)							
Story	Story Height (in)	X Direction			Y Direction		
		Actual	Allowable = .015H_{sx}	Check actual < allowable	Actual	Allowable = .015H_{sx}	Check actual < allowable
Roof	1945	0.00067	2.16	OK	0.000639	2.16	OK
Penthouse	1801	0.000724	1.935	OK	0.000579	1.935	OK
13	1672	0.000742	1.935	OK	0.000632	1.935	OK
12	1543	0.000763	1.935	OK	0.000692	1.935	OK
11	1414	0.000777	1.935	OK	0.000755	1.935	OK
10	1285	0.000776	1.935	OK	0.000796	1.935	OK
9	1156	0.000769	1.935	OK	0.00082	1.935	OK
8	1027	0.00073	1.935	OK	0.000774	1.935	OK
7	898	0.000657	1.935	OK	0.000737	1.935	OK
6	769	0.000587	1.935	OK	0.00734	1.935	OK
5	640	0.0005	1.935	OK	0.000581	1.935	OK
4	511	0.000389	1.935	OK	0.000475	1.935	OK
3	382	0.000295	1.935	OK	0.00038	1.935	OK
2	253	0.000141	3.795	OK	0.00021	3.795	OK

T-38

Design Confirmation – Spot Checks

To properly confirm the existing lateral design, critical structural members were analyzed for strength requirements. Major areas of concern included both column shear checks and isolated uplift forces due to overturning moments on each lateral frame.

Before any design checks can be completed, the controlling loading conditions had to be determined. Considering all 38 loading conditions, story shear and drift output data was compared. In each direction, the largest story shears and story drifts resulted from the same two loading conditions. Wind case 1 with both ASCE7-05 load combination 4: 1.2D + 1.6W + L and load combination 6: .9D + 1.6W. Both load combinations include a 1.6 wind scale factor resulting in a large lateral load and story shears. Table T-39 shows the total story shear for the controlling load conditions.

Total Story Shear		
Controlling Load Combinations: 12 and 23		
Story	Vx (kips)	Vy (kips)
13	-32.62	-16.34
12	-63.46	-36.14
11	-93.68	-56.16
10	-123.25	-75.7
9	-152.14	-94.74
8	-180.34	-113.28
7	-207.78	-131.26
6	-234.35	-148.62
5	-259.94	-165.25
4	-284.45	-181.09
3	-307.04	-195.54
2	-328.26	-208.96
1	-359.86	-228.91

T-39

With the aid of the section cut technique, story shears in each lateral frame were determined for the controlling loading conditions. Before proceeding, the story shears of each frame were totaled (see green columns of table T-40) and compared to the total story shear values above. The values are nearly equivalent suggesting once again that torsion is minimal and output data is sensible. Story shears were converted to story forces and heights. As shown in table T-42, uplift forces were determined by dividing the overturning moment by the base dimension. At this point, wind case 1 with load combination 6 is most important for the overturning because it includes the .9 dead load scale factor. In order to determine if uplift forces have impact on the foundation, the uplift force values had to be compared to the corresponding magnitude of the accumulated dead load at the same corresponding base column (see figure F-20).

Story Shear For Each Lateral Frame:

	Story Shear Per Lateral Frame										
	X Direction				Y Direction						
story	BR-3	BR-4	BR-5	Total Vx (some shear unaccounted for: other columns)	BR-1	BR-2	MF-1	MF-2	MF-3	MF-4	Total Vy (some shear unaccounted for: other columns)
13	-16.0881	-0.1095	-15.37	-31.5676	15.0654	17.4169	-7.8632	-10.6261	-15.0028	-15.2417	-16.2515
12	-29.097	-8.0166	-24.5386	-61.6522	5.2617	7.0195	-9.7417	-8.1391	-15.478	-14.9769	-36.0545
11	-41.2991	-16.1409	-34.5191	-91.9591	-2.4222	0.7449	-10.2153	-9.2877	-17.3773	-17.506	-46.7759
10	-53.942	-24.1668	-43.2944	-121.4032	-10.6433	-6.7109	-10.6946	-9.9977	-18.5721	-18.974	-75.5926
9	-67.5185	-32.1547	-50.5988	-150.272	-16.0246	-11.8664	-13.3094	-10.2461	-21.648	-21.5366	-94.6311
8	-67.0383	-41.9963	-69.4974	-178.532	-20.3288	-17.9158	-14.8486	-15.9775	-21.8996	-22.1951	-113.1654
7	-78.4174	-48.7961	-77.4	-204.6135	-29.169	-24.9884	-14.6335	-15.3484	-23.1963	-23.8283	-131.1639
6	-74.3586	-82.0364	-76.9499	-233.3449	-39.388	-34.7354	-14.0926	-14.7987	-22.3013	-23.1848	-125.316
5	-82.0916	-90.5018	-85.2436	-257.837	-48.1333	-43.3221	-13.8918	-15.1206	-21.9622	-22.7039	-165.1339
4	-86.2344	-99.5055	-95.8794	-281.6193	-60.4674	-56.0745	-12.2027	-13.1669	-19.0558	-20.0205	-180.9878
3	-127.3893	-88.7406	-90.0917	-306.2216	-68.4241	-63.4477	-11.5803	-12.477	-19.9553	-19.5363	-195.4207
2	-125.9348	-101.333	-98.5057	-325.7736	-79.4139	-75.8152	-9.9098	-10.3447	-16.711	-16.6706	-208.8652
1	-152.4079	-78.4999	-118.489	-349.3963	-91.5772	-89.4733	-9.9834	-10.0712	-14.0141	-13.7097	-228.8289

T-40

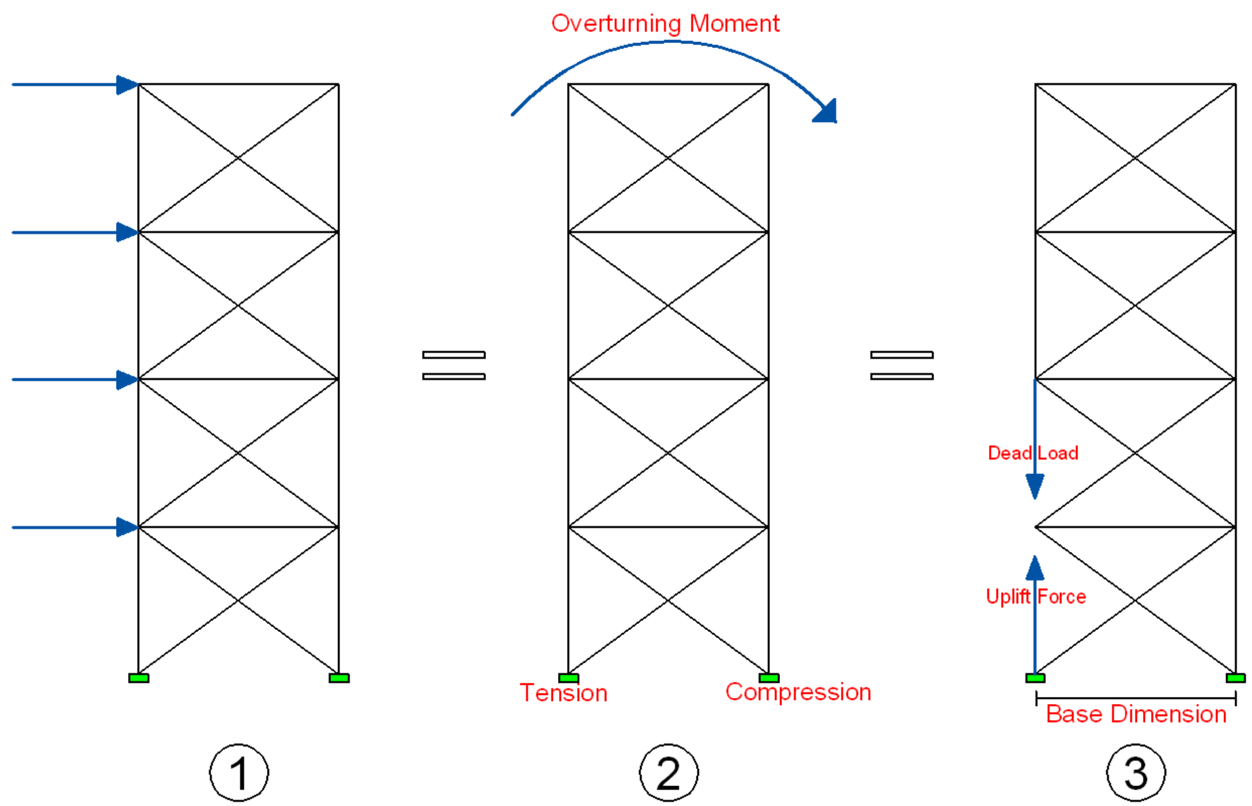
Story Forces For Each Lateral Frame:

	Story Forces Per Frame								
	X Direction			Y Direction					
story	BR-3	BR-4	BR-5	BR-1	BR-2	MF-1	MF-2	MF-3	MF-4
P	16.0881	0.1095	15.37	-15.0654	-17.4169	7.8632	10.6261	15.0028	15.2417
13	13.0089	7.9071	9.1686	9.8037	10.3974	1.8785	-2.487	0.4752	-0.2648
12	12.2021	8.1243	9.9805	7.6839	6.2746	0.4736	1.1486	1.8993	2.5291
11	12.6429	8.0259	8.7753	8.2211	7.4558	0.4793	0.71	1.1948	1.468
10	13.5765	7.9879	7.3044	5.3813	5.1555	2.6148	0.2484	3.0759	2.5626
9	-0.4802	9.8416	18.8986	4.3042	6.0494	1.5392	5.7314	0.2516	0.6585
8	11.3791	6.7998	7.9026	8.8402	7.0726	-0.2151	-0.6291	1.2967	1.6332
7	-4.0588	33.2403	-0.4501	10.219	9.747	-0.5409	-0.5497	-0.895	-0.6435
6	7.733	8.4654	8.2937	8.7453	8.5867	-0.2008	0.3219	-0.3391	-0.4809
5	4.1428	9.0037	10.6358	12.3341	12.7524	-1.6891	-1.9537	-2.9064	-2.6834
4	41.1549	-10.7649	-5.7877	7.9567	7.3732	-0.6224	-0.6899	0.8995	-0.4842
3	-1.4545	12.5925	8.414	10.9898	12.3675	-1.6705	-2.1323	-3.2443	-2.8657
2	26.4731	-22.8332	19.9828	12.1633	13.6581	0.0736	-0.2735	-2.6969	-2.9609

T-41

Overturning Moment:

As expected, significant uplift forces were only associated with the braced frames. Due to the large story forces and short base dimensions, the braced frames have a greater tendency to overturn than the moment frames. The diagram below is a visual representation of the process and calculations required to check for overturning moment in a braced frame. All the frames have uplift forces present in the columns; however, the accumulated dead force load counteracts the uplift force. In order to determine if the foundation needs to be designed for uplift, the magnitudes of the dead load and uplift force for each area of interest were compared (see table T-42).



Step One: Determine Story Forces Per Lateral Frame

Step Two: Calculate Overturning Moment Per Lateral Frame

Step Three: Determine Uplift force and Dead Load at Point of Interest

F-20

Overturning Moment Results:

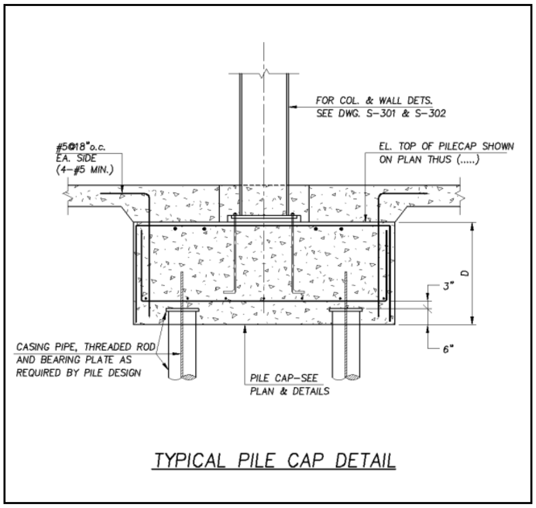
The entire calculation process is summarized in the table below. Several areas require uplift resistance. These locations are highlighted in figure S-11. The existing foundation system includes several pile caps designed for uplift forces. The typical detail of the pile cap is shown in figure S-12.

Overturning Moments (ft-kips)										
story	Story Height (ft.)	X Direction			Y Direction					
		BR-3	BR-4	BR-5	BR-1	BR-2	MF-1	MF-2	MF-3	MF-4
P	150.083	2414.55	16.43409	2306.776	-2261.06	-2613.98	1180.133	1594.797	2251.665	2287.52
13	139.333	1812.569	1101.72	1277.489	1365.979	1448.701	261.737	-346.521	66.21104	-36.8954
12	128.583	1568.983	1044.647	1283.323	988.0189	806.8069	60.89691	147.6904	244.2177	325.1993
11	117.833	1489.751	945.7159	1034.02	968.7169	878.5393	56.47736	83.66143	140.7869	172.9788
10	107.083	1453.812	855.3683	782.1771	576.2457	552.0664	280.0006	26.59942	329.3766	274.4109
9	96.333	-46.2591	948.0709	1820.559	414.6365	582.7569	148.2758	552.123	24.23738	63.43528
8	85.583	973.8575	581.9473	676.3282	756.5708	605.2943	-18.4089	-53.8403	110.9755	139.7742
7	74.833	-303.732	2487.471	-33.6823	764.7184	729.3973	-40.4772	-41.1357	-66.9755	-48.155
6	64.083	495.5538	542.4882	531.4852	560.4251	550.2615	-12.8679	20.62832	-21.7305	-30.8175
5	53.333	220.948	480.1943	567.2391	657.8146	680.1237	-90.0848	-104.197	-155.007	-143.114
4	42.583	1752.499	-458.402	-246.458	338.8202	313.973	-26.5037	-29.378	38.30341	-20.6187
3	31.833	-46.3011	400.8571	267.8429	349.8383	393.6946	-53.177	-67.8775	-103.276	-91.2238
2	21.0833	558.1403	-481.399	421.3034	256.4425	287.9578	1.551731	-5.76628	-56.8596	-62.4255
Total Overturning Moment per frame		12344.37	8465.113	10688.4	5737.166	5215.592	1747.553	1776.784	2801.925	2830.069
Base Dimension: B		18'10"	16'-3"	16'-3"	15'-6"	15'-11"	52'-2"	54'-8"	53'-0"	53'-8"
Uplift Force in Exterior Column: $F_{uplift} = M_{overturn} / B$		655.96	520.93	657.72	370.14	327.68	3.346	32.5	52.85	52.73
Accumulated Dead Load		282.252	275.66	275.66	213.4	202.6	Moment Frames: No uplift expected. Low story forces and large base dimension resulted in a very low uplift force. Dead load will exceed the uplift force.			
Dead Load Factor		0.9	0.9	0.9	0.9	0.9				
Factored Dead Load		254.03	248.094	248.094	192.1	182.34				
Net Force At Base Column		401.93	272.836	409.626	178.04	145.34				

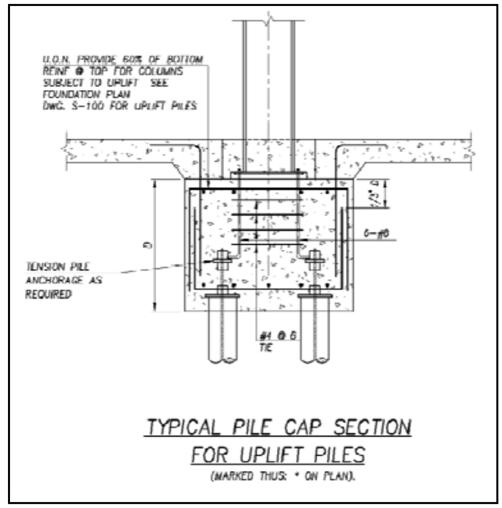
T-42

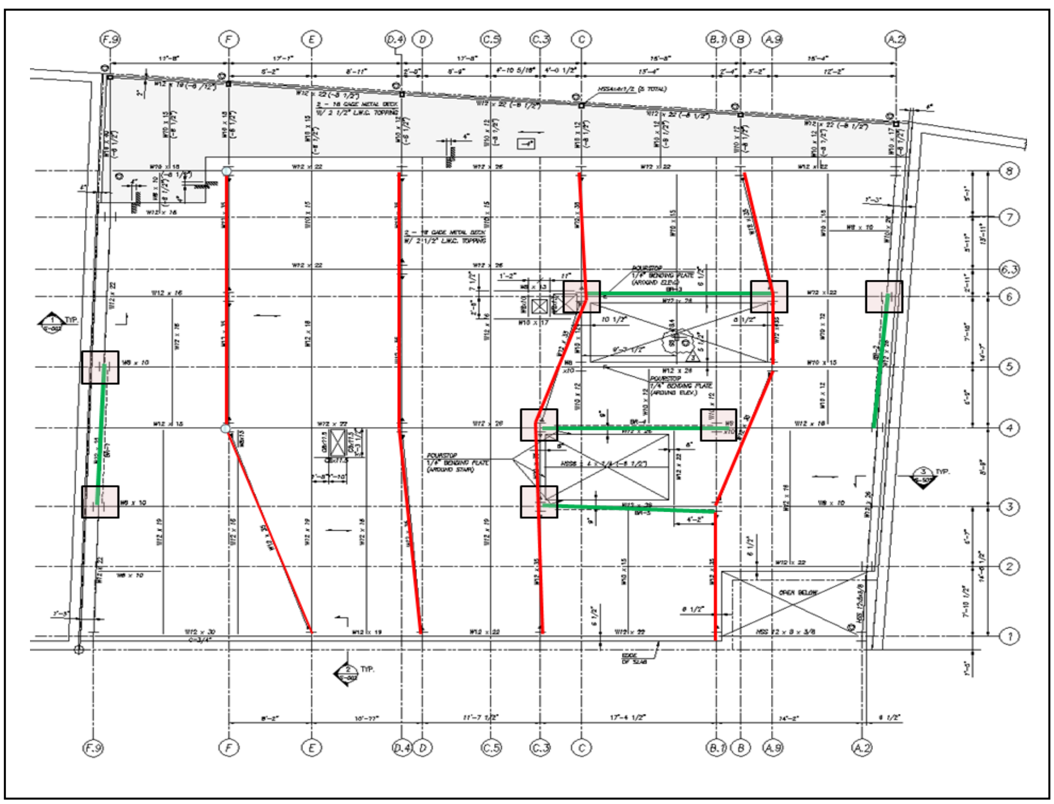
Uplift Forces:

As confirmed by the above calculations, the existing foundation of 40 Gold Street consists of several pile caps (shown in figure below) that are designed for uplift resistance. Pictured in figure, a solution to the uplift issue are pile caps with tension anchored piles. #4 Ties are also used at 6” on center. The tension piles resist uplift forces by transferring the load to the surrounding ground by way of friction. The micro piles are small circular HSS section shapes injected with grout. As shown in the detail below, steel reinforcement is used to resist the tension loads induced by uplift. To further demonstrate the impact of uplift on foundation design, the pile cap design that does not resist uplift is shown below.



S-12





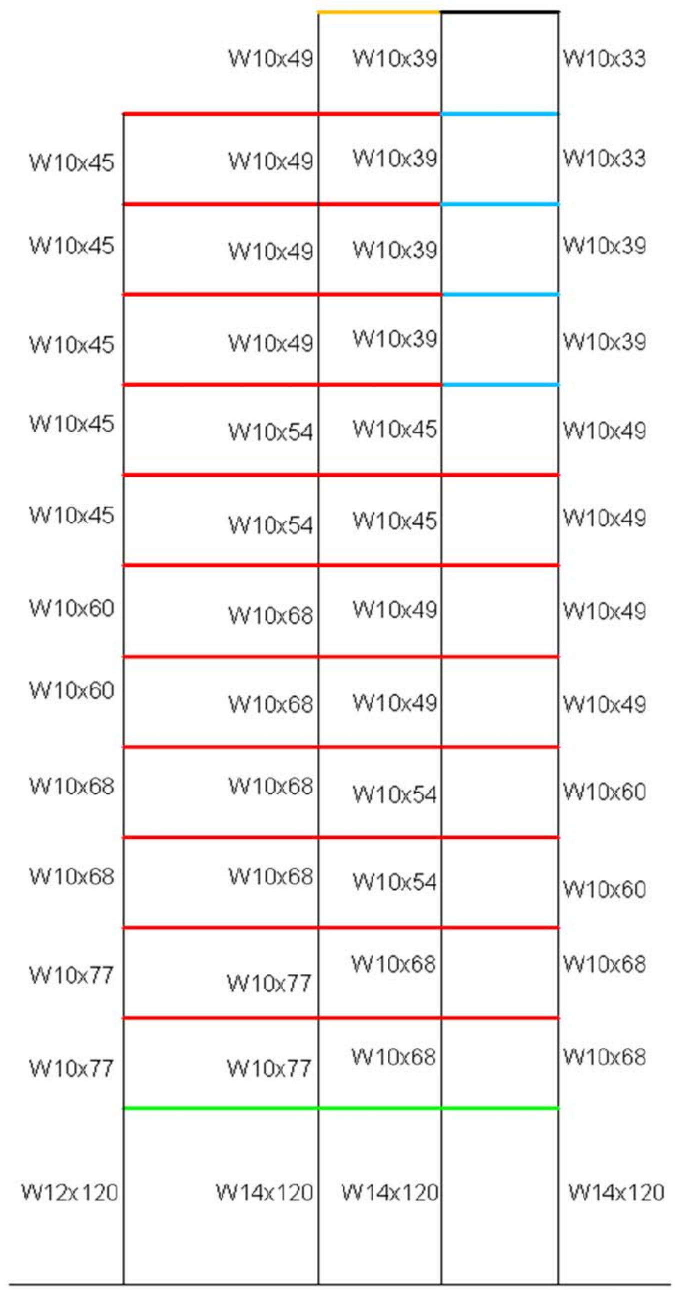
S-11 Plan View – Pile Caps Designed For Uplift

Verification of Output Data – Hand Calculations (Portal Method)

Before proceeding, the portal method and an approximate method for determining story drifts was completed. Moment frame MF-1 was chosen for the analysis. The goal of these calculations was to provide another set of data to verify the ETABS output. Using ETABS and the section cut technique, story shears were determined for MF-1 under the controlling loading condition. For the following calculations, several assumptions were made:

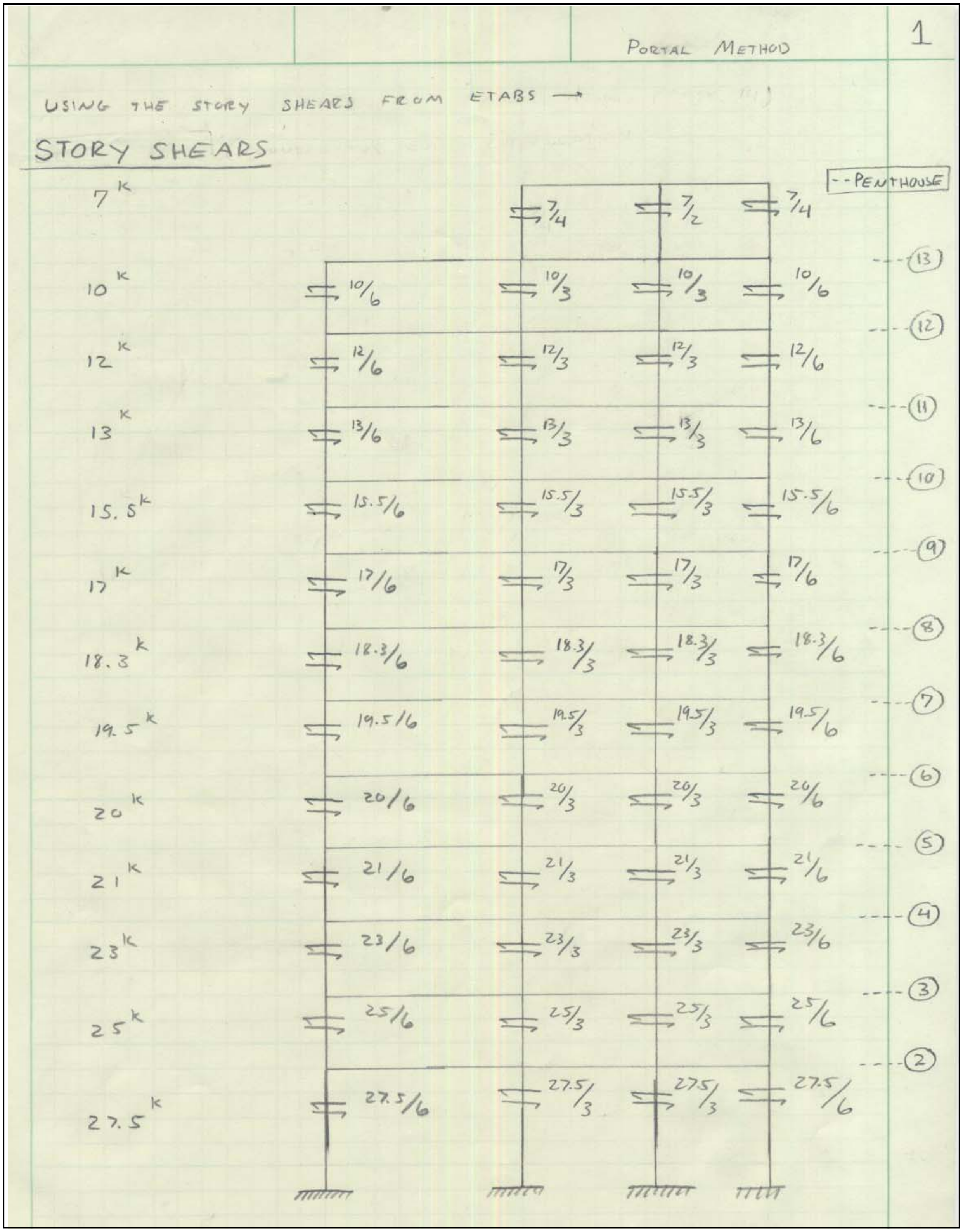
1.) Inflection points at mid-height of columns and mid-span of beams
2.) External columns carry ½ the shear of internal columns
3.) Axial Deformations are neglected

The diagram **F-21** below represents Moment Frame MF-1 with column and beam sizes shown.

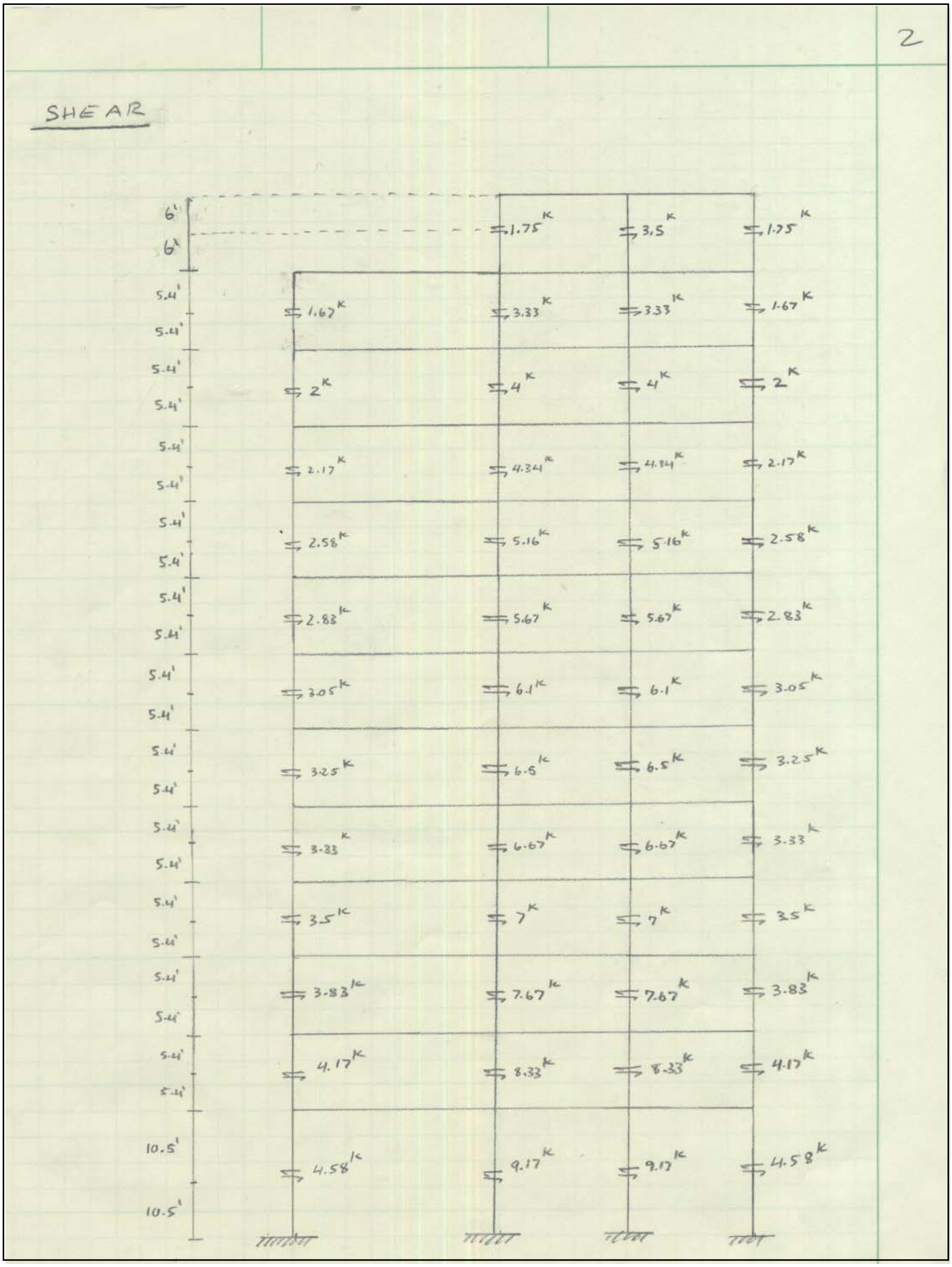


F-21

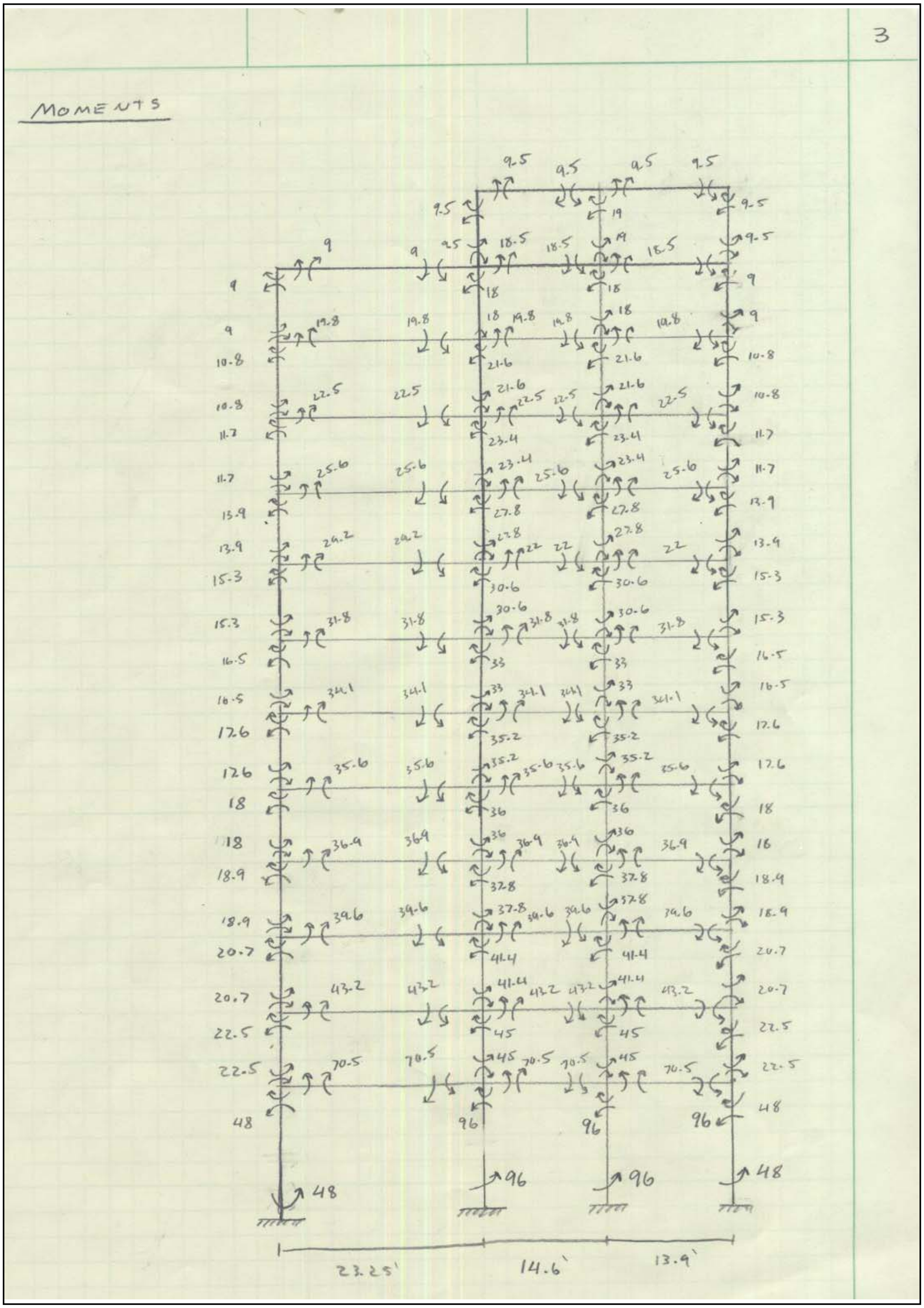
Step One: The story shears were distributed to the columns according to assumption two above.



Step Two: Important Dimensions were determined for calculation of moments.



<u>Step Three</u>: Column and beam moments were calculated for the entire moment frame M-1.



Step Four: Approximate story drifts were calculated using the column and beam moments above.

5

ESTIMATE LATERAL DISPLACEMENTS

$$U_1 = \frac{(M_1 \text{ column})(\text{column height} \times 12)^2}{6 E I_{column}} + \frac{(\text{column height} \times 12)(M_1 \text{ Beam})(L_{Beam} \times 12)}{12 (E)(I_{BEAM})}$$

$$U_1 = \frac{(48)(21 \times 12)^2}{6(29000)(1070)} + \frac{(21 \times 12)(70.5)(23 \times 12)}{(12)(29000)(285)} = \boxed{.0477 \text{ inches}}$$

$$U_2 = U_1 + \frac{(22.5)(10.75 \times 12)^2}{6(29000)(455)} + \frac{(10.75 \times 12)(23 \times 12)(43.2 + 70.5)}{12(29000)(238)} = \underline{.101 \text{ inches}}$$

$- .0707$ in

$$2^{ND} \text{ STORY DRIFT} = U_2 - U_1 = \boxed{.0536 \text{ inches}}$$

$$U_3 = U_2 + \frac{(20.7)(10.75 \times 12)^2}{6(29000)(455)} + \frac{(10.75)(12)(23 \times 12)(34.6 + 43.2)}{12(29000)(238)} =$$

$$U_3 = .101 + .0399 = .1409$$

$$3^{RD} \text{ STORY DRIFT} = U_3 - U_2 = .1409 - .101 = \boxed{.0399 \text{ inches}}$$

$$U_4 = U_3 + \frac{(18.4)(10.75 \times 12)^2}{6(29000)(394)} + \frac{(10.75)(12)(23 \times 12)(39.6 + 36.9)}{12(29000)(238)}$$

$$U_4 = .1409 + .0372 = .178$$

$$4^{TH} \text{ STORY DRIFT} = U_4 - U_3 = \boxed{.0372 \text{ inches}}$$

$$U_5 = U_4 + \frac{(18)(10.75 \times 12)^2}{6(29000)(394)} + \frac{(10.75)(12)(23 \times 12)(35.6 + 36.9)}{12(29000)(238)}$$

$$U_5 = .178 + .0355 = \underline{\underline{.214}}$$

$$5^{TH} \text{ STORY DRIFT} = \boxed{.0355 \text{ inches}}$$

Step Four Continued:

6

$$U_6 = U_5 + \frac{(17.6)(10.75\times 12)^2}{6(29000)(341)} + \frac{(10.75)(12)(23\times 12)(35.6+34.1)}{12(29000)(238)} = \underline{.249 \text{ in}}$$

6^{TH} STORY DRIFT $= \boxed{.0349}$

$$U_7 = U_6 + \frac{(16.5)(10.75\times 12)^2}{6(24000)(341)} + \frac{(10.75)(12)(23\times 12)(34.1+31.8)}{12(29000)(238)} = \underline{.2819 \text{ in}}$$

7^{TH} STORY DRIFT $= \boxed{.0329}$

$$U_8 = U_7 + \frac{(15.3)(10.75\times 12)^2}{6(24000)(248)} + \frac{10.75(12)(23\times 12)(31.8+24.2)}{12(29000)(238)} = \underline{.314 \text{ in}}$$

8^{TH} STORY DRIFT $= \boxed{.0321}$

$$U_9 = U_8 + \frac{(13.9)(10.75\times 12)^2}{6(24000)(248)} + \frac{10.75(12)(23\times 12)(29.2+25.6)}{12(29000)(238)} = \underline{.343 \text{ in}}$$

9^{TH} STORY DRIFT $= \boxed{.0289}$

$$U_{10} = U_9 + \frac{(11.7)(10.75\times 12)^2}{6(29000)(248)} + \frac{10.75(12)(23\times 12)(25.6+22.5)}{12(29000)(238)} = \underline{.368 \text{ in}}$$

10^{TH} STORY DRIFT $= \boxed{.0251}$

$$U_{11} = U_{10} + \frac{(10.8)(10.75\times 12)^2}{6(29000)(248)} + \frac{10.75(12)(23\times 12)(19.8+22.5)}{12(29000)(238)} = \underline{.3903 \text{ in}}$$

11^{TH} STORY DRIFT $= \boxed{.0223}$

$$U_{12} = U_{11} + \frac{(9)(10.75\times 12)^2}{6(29000)(248)} + \frac{10.75(12)(23\times 12)(19.8+9)}{12(29000)(238)} = \underline{.4062 \text{ in}}$$

12^{TH} STORY DRIFT $= .0159$

Results: Hand calculations yielded larger drift values than observed in the 3D ETABS output; however, the drift values calculated here do not exceed the allowable total drift or story drift. Several approximations and assumptions listed above were made during the calculations that could explain the difference in results.

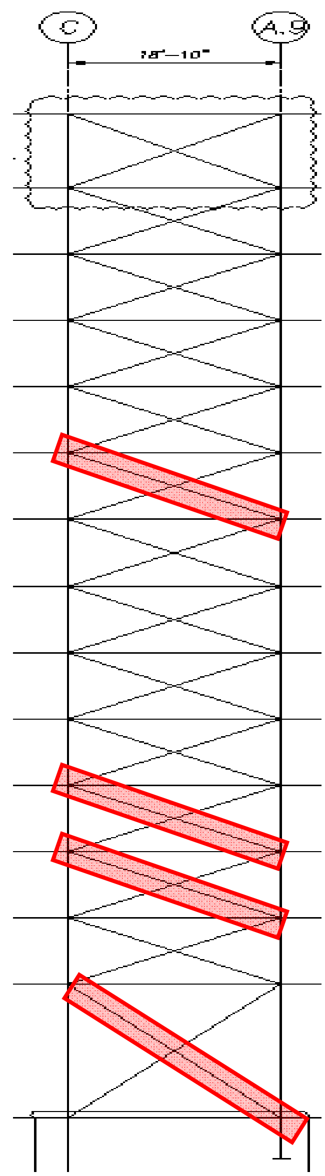
Spot Check – Braces

The following calculations are design spot checks for the cross braces in braced frame 3. As shown in table **T-41**, the largest story shears are found in braced frame 3. Therefore, by confirming the design of these critical members shown in figure **F-22**, it is fair to assume all other braces are appropriately designed.

Similar to truss members, the cross braces are required to carry tension under some conditions and compression under others. Often times in practice, it is more economical to provide extra tension members and to assume the members will buckle under compression instead of carrying load. Such an assumption allows braces to be designed as tension-only members resulting in braces with smaller cross sectional areas.

Since the compression strength is often much less than the tension strength, and because assumptions made during the actual design are not known, design checks will be completed for the compressive forces resulting from the controlling loading condition: Wind Case 1 combined with Load Combination 4 (1.2D + 1.6W + L + .5 Lr).

1	<u>Member Size</u>: HSS7x4x3/8 <u>Compressive Load</u>:
2	<u>Member Size</u>: HSS8x4x1/4 <u>Compressive Load</u>:
3	<u>Member Size</u>: HSS8x6x3/8 <u>Compressive Load</u>:
4	<u>Member Size</u>: HSS10x6x5/8 <u>Compressive Load</u>:

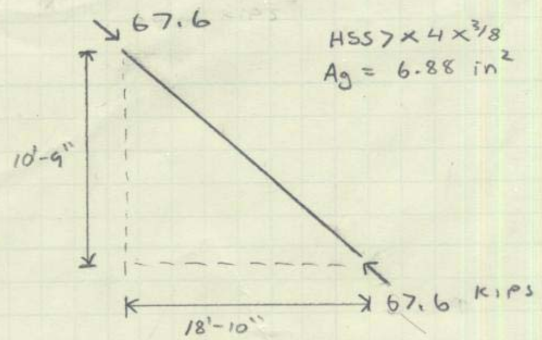


F-22

Spot Check – Brace 1:

1

SPOT CHECK – BRACE 1



HSS $7 \times 4 \times 3/8$

$A_g = 6.88 \text{ in}^2$

STEP ONE: PIN / PIN $K = 1.0$

STEP TWO: $L = \sqrt{10.75^2 + 18.83^2}$

$L = 21.685'$

STEP THREE: TABLE 1-11

$r_x = 2.46$

STEP FOUR: $\frac{KL}{r} \times 12 =$

$= \frac{1.0(21.685)}{2.46} \times 12 = 105.78$

STEP FIVE: COMPARE $\frac{KL}{r} \times 12$ TO $4.71\sqrt{\frac{E}{F_y}}$

$4.71\sqrt{29000/50} = 113 > 105.78$

INELASTIC

STEP SIX: $F_e = \frac{\pi^2 E}{(KL/r \times 12)^2} = 25.58$

STEP SEVEN: $F_{CR} = 50\left(.658^{\wedge 50/25.58}\right) = 22.06$

STEP EIGHT: $\phi P_n = \phi F_{CR} A_g$

$= .9(22.06)(6.88) = 136.6 > P_u = 67$ OKAY

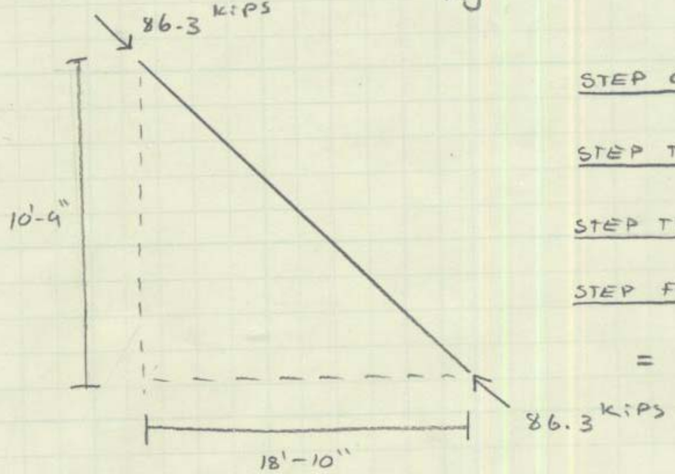
Spot Check – Brace 2:

2

SPOT CHECK - BRACE 2

HSS 8x4 x 1/4

$A_g = 5.24\ in^2$



STEP ONE: PIN / PIN $K = 1.0$

STEP TWO: $L = 21.685'$

STEP THREE: TABLE 1-11: $r_x = 2.85$

STEP FOUR: $\frac{KL}{r} \times 12 =$

$= \frac{1.0(21.685) \times 12}{2.85} = 91.305$

STEP FIVE: COMPARE TO $4.71\sqrt{E/F_y} = 113$

$91.305 < 113 \therefore$ INELASTIC

STEP SIX: $F_e = \frac{\pi^2 E}{(KL/r \times 12)^2} = \frac{\pi^2(29000)}{(91.305)^2} = 34.33$

STEP SEVEN: $F_{cr} = (.658 ^{F_y/F_e}) F_y$

$= (.6588 ^{50/34.33}) 50$

$= 27.23$

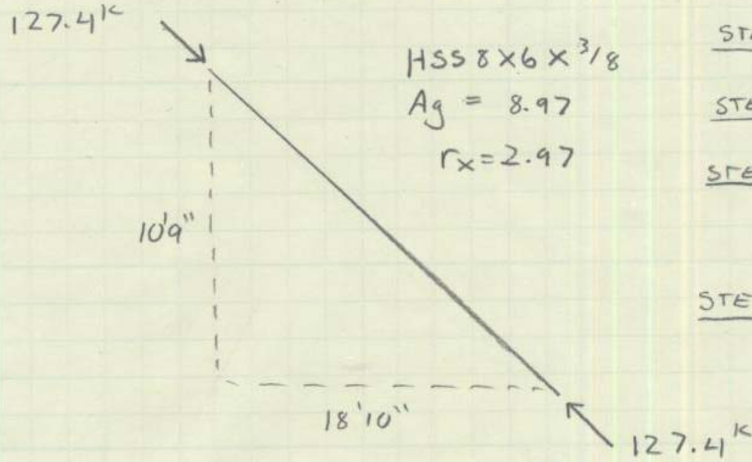
STEP EIGHT $\phi P_n = \phi F_{cr} A_g = (.9)(27.23)(5.24) = 128.4^k$

$\boxed{\phi P_n = 128.4^k > P_u = 86.3^k \quad \text{OKAY}}$

Spot Check – Brace 3:

3

SPOT CHECK – BRACE 3



HSS 8X6X3/8

A_g = 8.97

r_x = 2.97

STEP ONE: PIN/PIN K = 1.0

STEP TWO: L = 21.685'

STEP THREE: TABLE 1-11

$r_x = 2.97$

STEP FOUR: $\frac{KL}{r} \times 12 =$

$= \frac{1.0\,(21.685)}{2.97} \times 12 = 87.616$

STEP FIVE: $4.71\sqrt{E/F_y} = 113 > 87.616$

INELASTIC

STEP SIX: $F_e = \frac{\pi^2 E}{(KL/r \times 12)^2} = \frac{\pi^2 (29{,}000)}{87.616^2} = 37.28$

STEP SEVEN: $F_{CR} = (.658^{\wedge\, 50/37.28})\,50 = 28.5$

STEP EIGHT: $\phi P_n = \phi F_{CR} A_g = .9(28.5)(8.97) = 230.27$

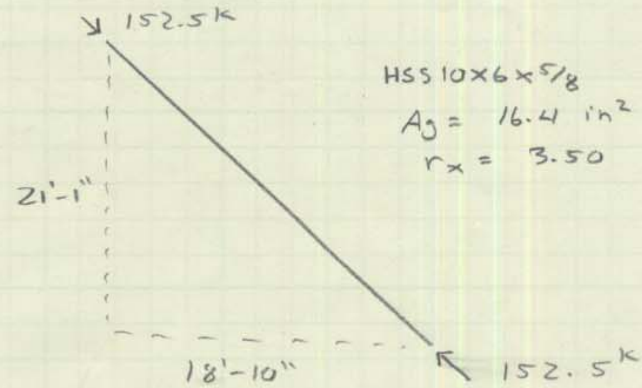
$\phi P_n = 230.27 > P_u = 127.4^k$

OKAY ✓

Spot Check – Brace 4:

4.

SPOT CHECK – BRACE 4



HSS 10x6x5/8
$A_g = 16.4 \text{ in}^2$
$r_x = 3.50$

STEP ONE: PIN / PIN $\therefore$ $K = 1.0$

STEP TWO: $L = \sqrt{21'\text{-}1''^2 + 18'\text{-}10''^2} = 28.27'$

STEP THREE: TABLE 1-11 : $r = 3.50$

STEP FOUR: $\frac{KL}{r} \times 12 = \frac{1.0\,(28.27) \times 12}{3.50} = 96.93$

STEP FIVE: $4.71\sqrt{E/F_y} = 113 > 96.93$ INELASTIC

STEP SIX: $F_e = \frac{\pi^2 \cdot E}{(96.93)^2} = 30.4$

STEP SEVEN: $F_{CR} = F_y\left(.658^{\,50/30.41}\right) = 25.156$

STEP EIGHT: $\phi P_n = \phi F_{CR} A_g = .9\,(25.156)(16.41)$

$$\boxed{\phi P_n = 371.3^k > P_u = 152.5^k \quad \text{OKAY} \checkmark}$$

Spot Check – Column:

A strength check for a first story lateral column in brace frame 3 was performed. The design check involves analyzing the column under flexural and compressive loads due to the controlling load condition: 1.2D + 1.6W + L + .5Lr.

Critical Column Check

$\boxed{D:}$ (53 psf) × (15' × 20') × (14 floors) = 222,600 lbs

→ 222.6 kips + 8000 lbs for column weight

→ D = 230.6 kips

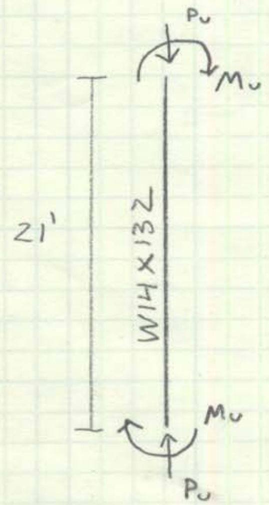
$\boxed{L:}$ (40 psf) × (15 × 20 × 14) = 168 kips

$\boxed{Lr:}$ 12 kips

$P_u = 1.2D + L + L_R = 1.2(230.6) + 168 + 12$

$P_u = 456.72$ kips

M_u : 75 ft-k due to 1.6 w lateral load



AISC MANUAL

TABLE 6-1 $KL = 21'(r_x/r_y) = 21(1.67) = 35.07'$

$p \times 10^3 = 1.51$

$b \times 10^3 = 1.27$

$P_u(1.51 \times 10^{-3}) + M_u(1.27 \times 10^{-3}) =$

$456.72(1.51 \times 10^{-3}) + 75(1.27 \times 10^{-3}) =$

$= .785 < 1.0$

OKAY ✓

Conclusions:

In the third technical report of the 40 Gold Street building, an in depth analysis of the lateral system was completed to verify that the design meets all code requirements. Confirmation of design required careful examination of critical lateral members to verify all strength, drift, story drift, and overturning requirements are satisfied. To accurately confirm the design, a 3D model of the existing lateral system was created using ETABS. However, instead of exclusively relying on the 3D model, the analysis process began with two methods of preliminary analysis.

Method one incorporated the use of 2D computer modeling to determine relative stiffness values of each frame based on displacement from a 1 kip load. Preliminary analysis method one involved significant simplification of the moment frame geometries. Consequently, the results are not accurate but provide valuable insight regarding the behavior of the lateral system. According to the preliminary data, the center of rigidity and center of mass are nearly equivalent, and torsion effects are minimal.

With only a few approximations and simplifications, the second method of preliminary analysis yielded realistic relative stiffness values. Based on distribution of lateral forces in the 3D model, the braced frames are the dominant lateral force resisting elements and are approximately 7 to 8 times stiffer than the moment frames. Braced frames are traditionally about 10 times stiffer than moment frames of similar geometry. However, the moment frames are much wider than the braced frames providing additional rigidity. Method two also estimated the center of rigidity to be located close to the center of mass.

In order to properly confirm the existing lateral design, several loading combinations were defined and applied to the ETABS model. Considering the 7 basic load combinations of ASCE7-05 section 3.2.3 and the 4 wind cases defined in ASCE7-05 figure 6.9, 38 different load combinations were applied to the model. By comparing story shear and story drift output, the controlling loading conditions in both the X and Y direction was determined to be Wind case 1 in conjunction with either ASCE7-05 equation 4: **1.2D + 1.6W+ L + .5Lr** or ASCE7-05 equation 6: **.9D + 1.6W.**

Using unfactored wind loads calculated in technical report 1, drift and story drifts under the 4 ASCE7-05 wind cases were checked for serviceability issues. Based on the ETABS output, wind case one controls, and the corresponding drift and story drifts did not exceed the allowable drift: $\Delta_{WIND} = H / 400$.
3D output data for ASCE 7-05 Load Combinations 5: **1.2D + 1.0E + 1.0L** and load combination 7: **.9D + 1.0E** were examined to verify seismic induced story drifts do not exceed the allowable drift: $\Delta_{SEISMIC} = .015hsx$. As expected, the design satisfies the stability requirement associated with seismic loading.

A second method of drift analysis was employed to provide another source of design confirmation. Using the portal method and an approximate displacement method, moment frame MF-1 was analyzed to determine story drifts induced by the controlling loading condition mentioned above. Although hand calculations yielded values slightly larger than observed in the 3D output, the approximated story drifts do not exceed the allowable limit.

Based on inspection, it was determined the braced frames possess the largest overturn potential. Not only do the braced frames resist the largest story forces, but their small base dimensions magnify the uplift forces resulting from the overturn moments. Therefore, each braced frame was analyzed under the controlling loading condition. After comparing uplift forces with the counteracting dead loads, it was determined that 8 different locations require pile caps with uplift resistance. The existing foundation agrees With a low total building weight of just 4,681 kips, overturning due to lateral loads was expected to have impact on the foundation.

The final stage of design confirmation required strength checks of critical cross braces and lateral columns. Based on inspection, braced frame BR-3 resists the largest story shears under the controlling loading condition. As a result, member checks of braced frame BR-3 were assumed to represent an overall design check of the lateral system.

Based on the results of this report, it can be concluded that the existing lateral system design is adequate meeting all standard criteria and requirements. With a spread out lateral system, torsion effects are minimal. Wind loads generate the largest story shears and story drifts. The largest area of concern pertains to the overturning moment; however, appropriately designed pile caps (as shown in report) provide an easy solution to uplift issues.

APPENDIX A – WIND LOAD CALCULTATIONS

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). The results of these wind calculations are illustrated in the following figures and tables.

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-43

T-44

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"

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-45

Calculated Wind Pressures for the North / South Direction

B = 56' 9-1/2"   L = 78' 9-1/2"

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-46

Table T-45 Is Illustrated in Diagram F-23

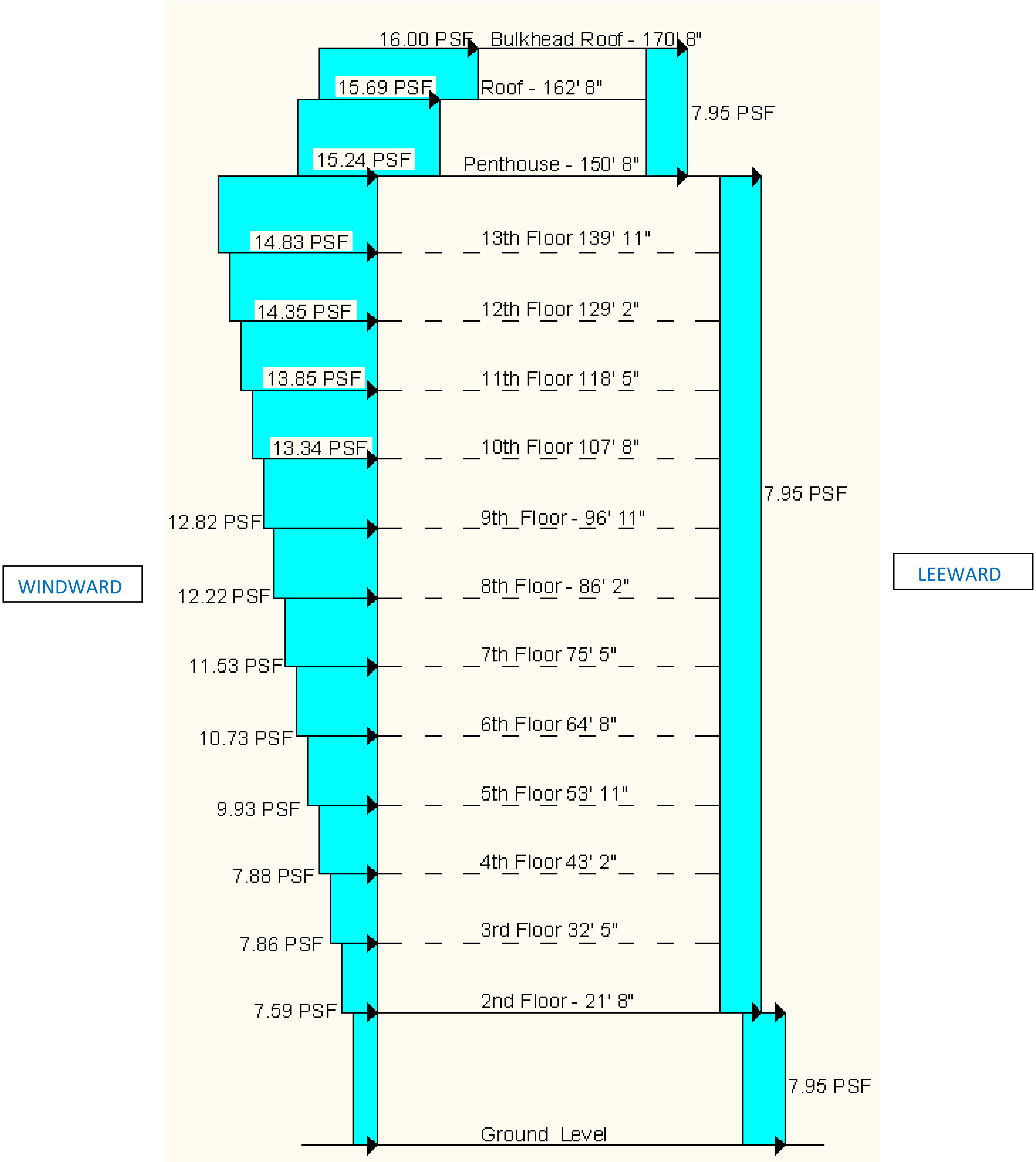
Table T-46 Is Illustrated in Diagram F-24

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-47

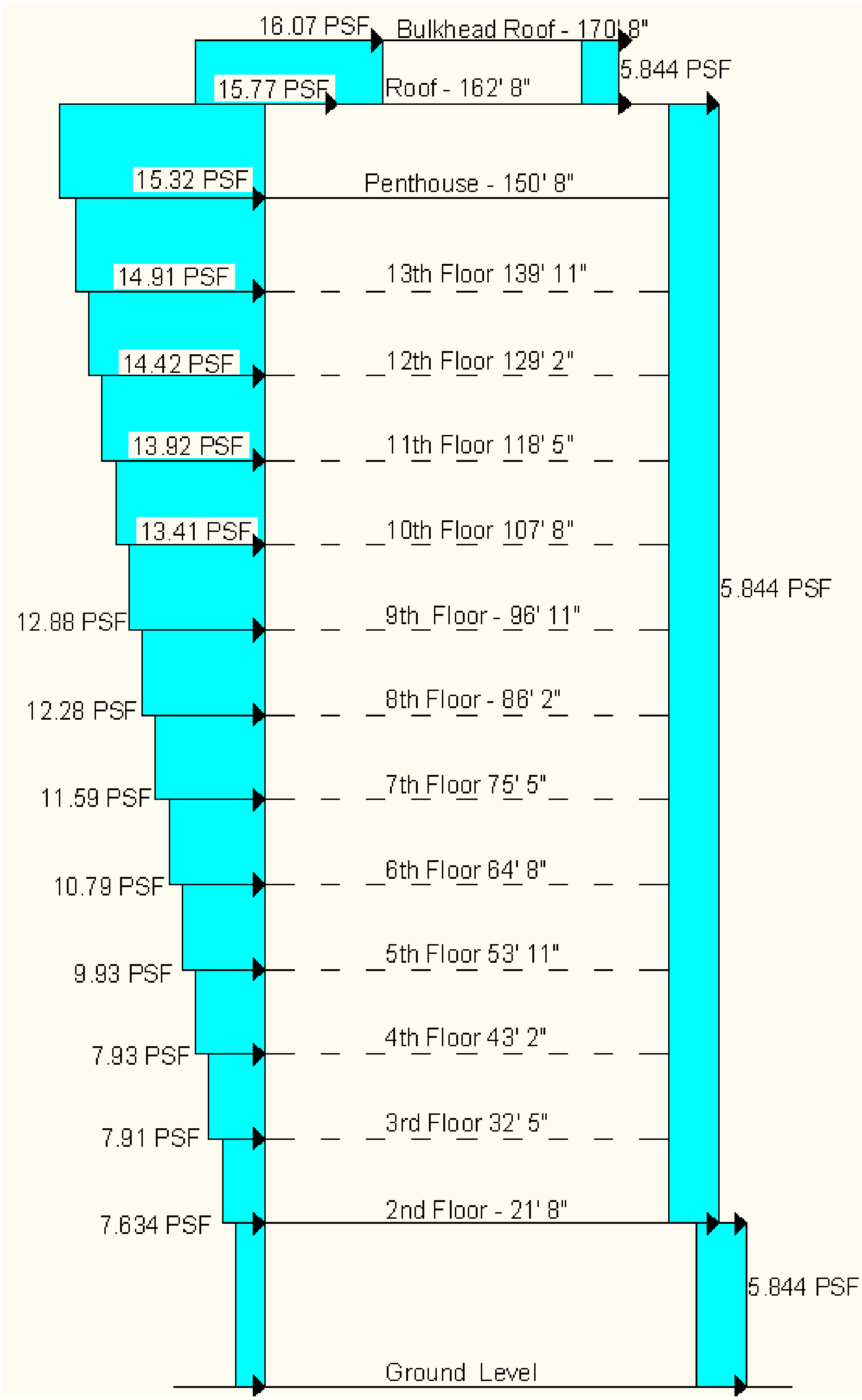
Wind Pressure Diagram for the East / West Diagram



F-23

Wind Pressure Diagram for the North/South Direction

WINDWARD



LEEWARD

F-24

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

LINEAR INTERPOLATION USING TABLE 6-3 CASE 2 EXPOSURE B

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

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

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

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

$K_{100} = 0.99$

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

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

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

<u>Sample Calculation</u> – 8TH FLOOR

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

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

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.

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

$$n_1 = 22.2 / h^{(.8)} = 22.2 / 170'8''^{(.8)} = \boxed{.363 \geq 1 \, H_z \text{ NOT FLEXIBLE}}$$

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

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

$$I_{\bar{z}} = c\left(\tfrac{33}{\bar{z}}\right)^{1/6} = 0.6\left(\tfrac{33}{102.4}\right)^{1/6} = \boxed{.4968}$$

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

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

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

V = 110 MPH

WIND CALCULATIONS | TECHNICAL REPORT 1 | JESSE COOPER

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

$$G_f = 0.925\left(\frac{1 + 1.7(.4968)\sqrt{(3.4)^2(.838)^2 + (3.788)^2(.09226)^2}}{1 + 1.7(3.4)(.4468)}\right)$$

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

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

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}:

$GC_{pn} = +1.5$ WINDWARD

$GC_{pn} = -1.0$ LEEWARD

DETERMINE COMBINED NET DESIGN PRESSURE ON PARAPET

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

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

$GC_{pi} = +0.18$
$\quad\quad\;\; -0.18$

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

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

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

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

$$R_L = \frac{1}{4.537} - \frac{1}{2(4.537)^2}\left(1 - e^{(-2 \cdot 4.537)}\right) = .194 = R_L$$

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

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

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

$R = .07179$

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

$G_f = .8877$

NO PARAPET IN THIS DIRECTION

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

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

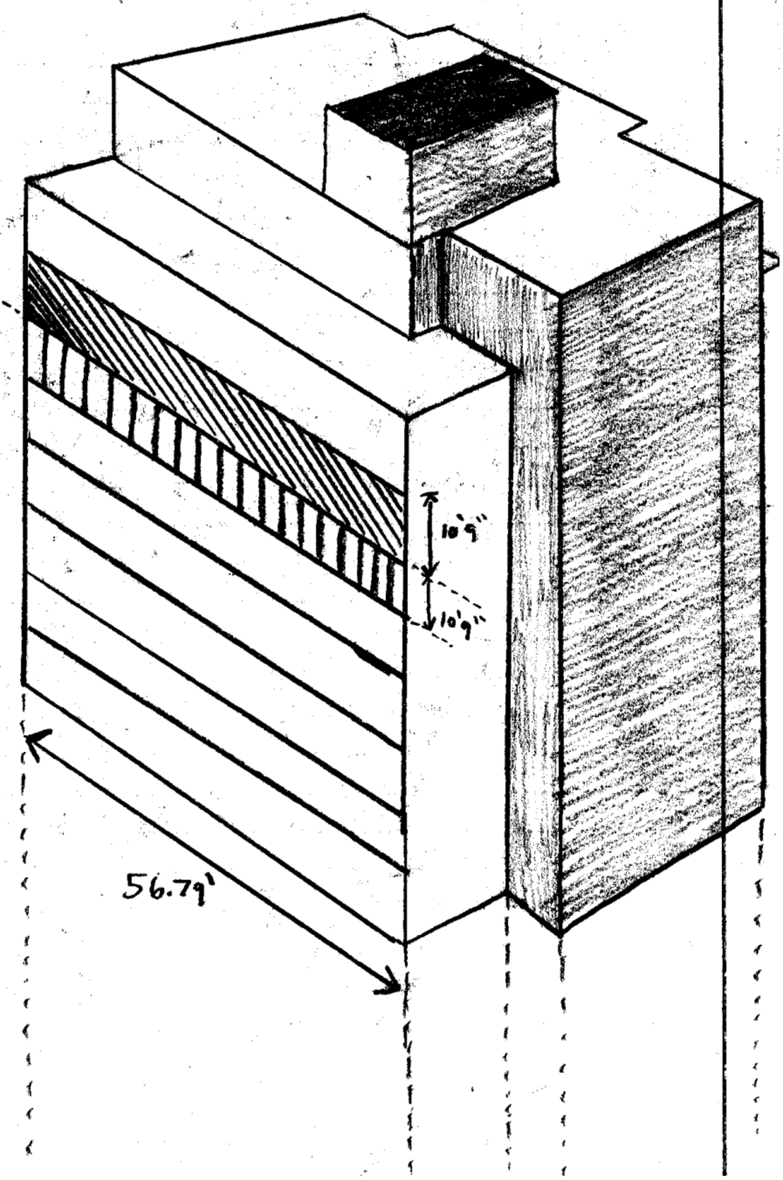
LINEAR INTERPOLATION: $\boxed{C_p = -.42 \text{ LEEWARD}}$

WIND CALCULATIONS	TECHNICAL REPORT I	JESSE COOPER	10

SAMPLE CALCULATION OF F_x VALUES DUE TO WIND

NORTH/SOUTH DIRECTION

FLOOR 12



(diagonal hatch) = 14.412 PSF

(vertical hatch) = 13.92 PSF

F_x @ FLOOR 12 = F_{12}

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

$$F_{12} = 8{,}650.67\ lbs$$

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

APPENDIX B - SEISMIC CALCULATIONS

The following tables **T-48 – T-56** , and corresponding calculations, 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-48 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-48

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-48 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-49

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-50

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-50 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-51

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-52

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-53

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-54

Table T-54 Represents the Culmination of Tables T51-T53

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-55

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-56

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

$S_1 = .062 \leq .6$

$\boxed{A}$ 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

$S_S = .35$

$$\left\{\begin{array}{l} S_S = .25 \longrightarrow \text{SITE D} \rightarrow F_a = 1.6 \\ S_S = .5 \longrightarrow \text{SITE D} \rightarrow F_a = 1.4 \end{array}\right\}$$

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

SEISMIC CALCULATIONS | TECHNICAL REPORT 1 | JESSE COOPER

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

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

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

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

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 → $.03 = C_t$, $x = .75$

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

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

$\boxed{\text{IS } T < 3.5T_s \text{ ?}}$ 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: $\Omega_0^{\,g} = 2\frac{1}{2}$

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

FIGURE 22-15 → $T_L = 6$

$T > T_L$? NO

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

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

TOTAL BUILDING WEIGHT CALCULATED = 4,681,330 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 ≤ 0.5 sec? NO T ≥ 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.

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

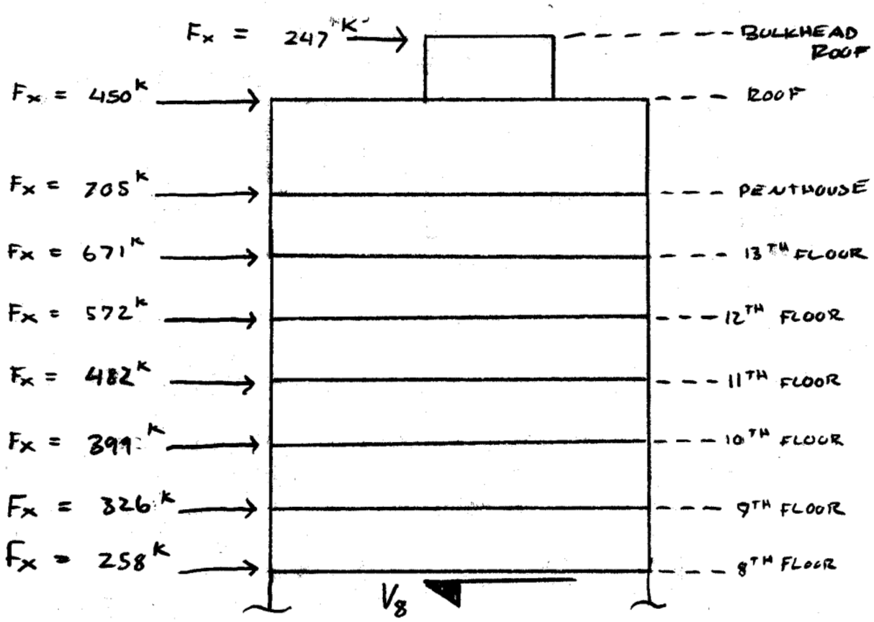
Sample calc

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

SEISMIC CALCULATIONS | TECHNICAL REPORT I | JESSE COOPER | 6

DETERMINE SEISMIC DESIGN STORY SHEAR V_x BY EQUATION 12.8-13

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

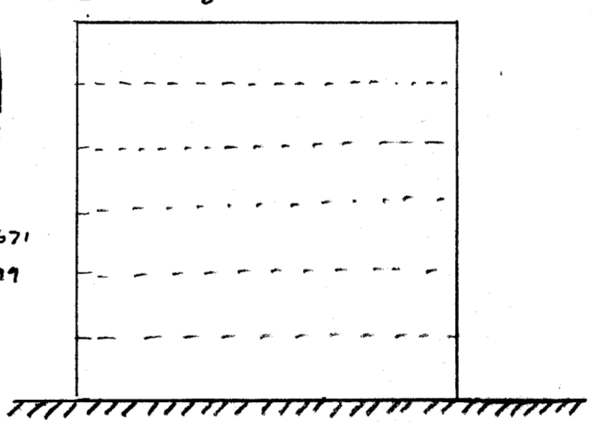


SAMPLE CALCULATION OF STORY SHEAR

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 ☐

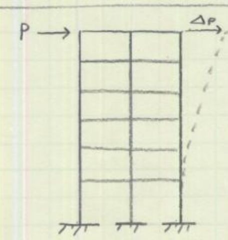
APPENDIX C – APPROXIMATE HAND CALCULTATIONS (METHOD ONE)

TECH 3 | APPROX CALCULATIONS | 1

APPROXIMATE CALCULATIONS

CALCULATING APPROXIMATE STIFFNESS FOR EACH LATERAL FRAME

$K = \frac{P}{\Delta_P}$ WHERE P = 1 KIP load



BR = BRACE FRAME

MF = MOMENT FRAME

BR-1 : $K = \frac{1}{.3177} = 3.1476$ K/in

BR-2 : $K = \frac{1}{.2807} = 3.5625$ K/in

BR-3 : $K = \frac{1}{.1674} = 5.9737$ K/in

BR-4 : $K = \frac{1}{.1879} = 5.3219$ K/in

BR-5 : $K = \frac{1}{.1742} = 5.74$ K/in

MF-1 : $K = \frac{1}{.3041} = 3.289$ K/in

MF-2 : $K = \frac{1}{.3269} = 3.059$ K/in

MF-3 : $K = \frac{1}{.1985} = 5.037$ K/in

MF-4 : $K = \frac{1}{.1861} = 5.373$ K/in

CALCULATING CENTER OF RIGIDITY

$$\bar{X} = \frac{\sum K_{iy} X_i}{\sum K_{iy}}$$

BR-1 : $K_{iy} X_i = (3.1476)(0) = 0$

BR-2 : $K_{iy} X_i = (3.5625)(77'-5'') = (3.5625)(929 \text{ in}) = 3309.5625$

MF-1 : $K_{iy} X_i = (3.289)(11'-8'') = (3.289)(140 \text{ in}) = 460.46$

MF-2 : $K_{iy} X_i = (3.059)(28'-9'') = (3.059)(345 \text{ in}) = 1055.355$

MF-3 : $K_{iy} X_i = (5.037)(46'-5'') = (5.037)(557 \text{ in}) = 2805.609$

MF-4 : $K_{iy} X_i = (5.373)(62'-1'') = (5.373)(745 \text{ in}) = 4002.885$

TECH 3 | APPROXIMATE CALCULATIONS | 2

$$\bar{X} = \frac{3309.5625 + 460.46 + 1055.355 + 2805.609 + 4002.885}{3.4176 + 3.5625 + 3.289 + 3.059 + 5.037 + 5.373}$$

$\bar{X} = 495.73'' = \underline{41.31'}$ (from specified origin)

$$\bar{Y} = \frac{\sum k_{ix} Y_i}{\sum k_{ix}}$$

BR-3: $k_{ix} Y_i = (5.9737)(37'9'') = (5.9737)(453 \text{ in}) = \underline{2{,}706.0861}$

BR-4: $k_{ix} Y_i = (5.3219)(23'-2'') = (5.3219)(278 \text{ in}) = \underline{1{,}479.4882}$

BR-5: $k_{ix} Y_i = (5.74)(14'-5'') = (5.74)(173 \text{ in}) = \underline{993.02}$

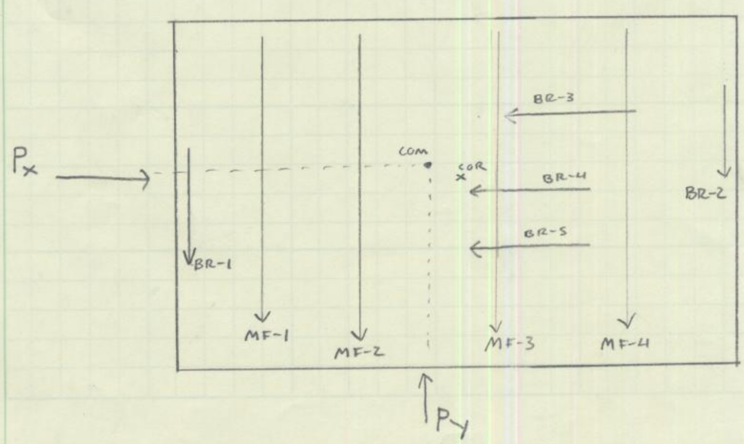
$$\bar{Y} = \frac{2706.0861 + 1{,}479.4882 + 993.02}{5.9737 + 5.3219 + 5.74} = 303.986 \text{ in} = \underline{25.33'}$$

THESE COORDINATES: (41.31', 25.33') REPRESENT THE LOWER FLOORS C.O.R. THE HIGHER FLOORS MAY HAVE A SLIGHTLY DIFFERENT C.O.R. DUE TO CHANGES IN AMOUNT AND SIZE OF MOMENT AND BRACE FRAMES.

THE FOLLOWING CALCULATIONS REPRESENT APPROXIMATE LATERAL FORCE DISTRIBUTION FOR THE LOWER FLOORS.

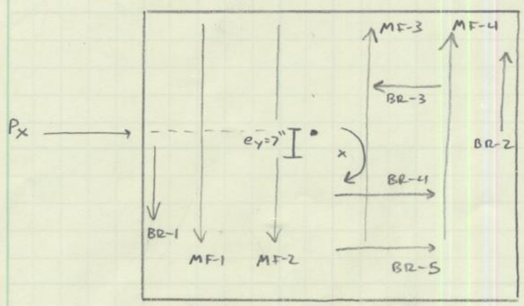
ECCENTRICITY: WHERE C.O.M. = (38'8", 26'-0")
C.O.R. = (41'4", 25'-3")
e = (2'-8", 0'-7")

DIRECTION OF FORCES

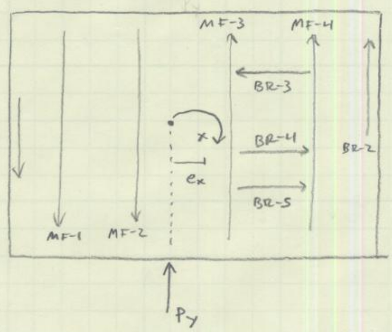


3

DIRECTION OF X TORSIONAL FORCES (DUE TO Px)



$e_y = 0'-7"$



$e_x = 2'-8"$

DIRECT SHEAR FORCES (X DIRECTION)

BR-3 $$F_{ix} = \frac{K_{ix}}{\sum k_{ix}} P_x = \frac{5.9737 \cdot P_x}{5.9737 + 5.3219 + 5.74} = .3506\, P_x$$

BR-4 $$F_{ix} = \frac{5.3219\ P_x}{5.9737 + 5.3219 + 5.74} = .3124\, P_x$$

BR-5 $$F_{ix} = \frac{5.74\ P_x}{5.9737 + 5.3219 + 5.74} = .3370\, P_x$$

4

TORSION EFFECT (X- DIRECTION) ~ (SHOULD BE MINIMAL DUE TO SMALL ECCENTRICITY)

d_i = distance from C.O.R. TO EACH WALL

BR-1 d_i = 41'-4" − 0 = 41'-4"

BR-2 d_i = 77'-5" − 41'4" = 36'-1"

BR-3 d_i = 37'-9" − 25'-3" = 12.416667'

BR-4 d_i = 25'-3" − 23'-2" = 2'-1"

BR-5 d_i = 25'-3" − 14'-5" = 10' 10"

MF-1 d_i = 41'-4" − 11'-8" = 29'-8"

MF-2 d_i = 41'-4" − 28'-9" = 12'-7"

MF-3 d_i = 46'-5" − 41'4" = 5'-1"

MF-4 d_i = 62'-1" − 41'4" = 20'-9"

$$J = \sum k_i d_i^2 = (5.9737)(12.416)^2 + (5.3219)(2'-1")^2 + (5.74)(10'-10")^2$$
$$+ (3.1476)(41'-4")^2 + (3.5625)(36'-1")^2 + (3.289)(29'-8")^2$$
$$+ (3.059)(12'-7")^2 + (5.037)(5'-1")^2 + (5.373)(20'-9")^2$$

J = 17,444.31 (k/in) ft^2

BR-3: $$f_{it} = \frac{k_i d_i P_x e_y}{\sum k_i d_i^2} = \frac{(5.9737)(12.41667)(P_x)(.5833)}{17,444.31}$$

$= .00248\, P_x$

BR-4 $$f_{it} = \frac{(5.3219)(2'-1")P_x(.5833)}{17,444.31} = .000371\, P_x$$

BR-5 $$f_{it} = \frac{(5.74)(10'-10")P_x(.58333)}{17,444.31} = .002079\, P_x$$

5

TOTAL FORCES X DIRECTION

$F_i = F_{i\,DIRECT} \pm F_{i\,TORSION}$

* NOTE: THE $\pm$ DEPENDS ON WHAT SIDE OF THE C.O.R. THE LATERAL ELEMENT OF INTEREST IS.

BR-3 $\longrightarrow$ $F_i = .3506\,P_x + .00248\,P_x = \boxed{.3531\,P_x}$

BR-4 $\longrightarrow$ $F_i = .3124\,P_x - .000371\,P_x = \boxed{.312\,P_x}$

BR-5 $\longrightarrow$ $F_i = .3370\,P_x - .002079\,P_x = \boxed{.3349\,P_x}$

DIRECT FORCES (Y DIRECTION)

$F_{iy} = \dfrac{k_{iy}}{\sum k_{iy}} P_y \qquad \sum k_{iy} =$

BR-1 $= \dfrac{3.1476}{23.4681} \times P_y = .1341\,P_y$

BR-2 $= \dfrac{3.5625}{23.4681} \times P_y = .1517\,P_y$

MF-1 $= \dfrac{3.289}{23.4681} \times P_y = .140\,P_y$

MF-2 $= 3.059/23.4681 \times P_y = .1302\,P_y$

MF-3 $= 5.037/23.4681 \times P_y = .2144\,P_y$

MF-4 $= 5.373/23.4681 \times P_y = .22874\,P_y$

TORSION FORCES (Y DIRECTION)

BR-1 : $f_{it} = (3.1476)(41'\text{-}4")(P_y)(2'\text{-}8")/17{,}444.31 = .0198\,P_y$

BR-2 : $f_{it} = (3.5625)(36'\text{-}1")(P_y)(2'\text{-}8")/17{,}444.31 = .0197\,P_y$

MF-1 : $f_{it} = (3.289)(29'\text{-}8")(P_y)(2'\text{-}8")/17{,}444.31 = .0149\,P_y$

MF-2 : $f_{it} = (3.059)(12'\text{-}7")(P_y)(2'\text{-}8")/17{,}444.31 = .0149\,P_y$

MF-3 : $f_{it} = (5.037)(5'\text{-}1")(P_y)(2'\text{-}8")/17{,}444.31 = .00391\,P_y$

MF-4 : $f_{it} = (5.373)(20'\text{-}4")(P_y)(2'\text{-}8")/17{,}444.31 = .017\,P_y$

6

TOTAL FORCES IN Y- DIRECTION

BR-1 $.134\ P_y + .0198\ P_y = \boxed{.1538\ P_y}$

BR-2 $.1517 P_y - .0197\ P_y = \boxed{.132\ P_y}$

MF-1 $.140\ P_y + .0149\ P_y = \boxed{.1549\ P_y}$

MF-2 $.1302 P_y + .0149\ P_y = \boxed{.1451 P_y}$

MF-3 $.2144 P_y - .00391\ P_y = \boxed{.21049\ P_y}$

MF-4 $.22874 P_y - .017\ P_y = \boxed{.21174\ P_y}$