technical report 1:

structural concepts and existing conditions report

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executive summary

This Structural Concepts and Existing Conditions Report summarizes the physical conditions as specified by the design engineer. Information related to the building design such as design concepts, building codes, and required loadings were all taken into consideration during lateral analysis. This report contains an overview of the floor framing, foundation system, structural slabs, lateral resisting system, bracing elements, and secondary structural systems for the Granby Tower.

While Granby Tower is part of a larger complex being constructed, I considered only tower elements in my analysis. In actuality, the tower shares the first 6 floors on the north and south-east with neighboring 6-story residential buildings and the first 7 floors with a much larger parking structure. The simplification due to this assumption will not necessarily result in a specific resultant because the surface area exposed to wind will be greater, but the shear walls in the residential buildings that will take some shear. So it can be assumed that this is a fair assumption to neglect since a more detailed lateral analysis of the residential buildings would be required to determine how they affect the lower stories of Granby Tower.

Preliminary wind and seismic lateral analyses predict that wind in the east-west directions cause the largest base shear (V_b = 2210 kips) experienced by Granby Tower. Despite the tremendous effective seismic weight of a concrete high-rise, wind was proven to control in all directions due to Norfolk's high basic wind speed (V = 110 mph).

Spot checks were carried out using gravity loads, story shear, and overturning moments due to lateral wind forces, all of which were found through gravity and lateral analysis. Spot checks of the two-way post-tensioning flat plate slab, a typical column at story 8, and the concrete shear wall core at the slab-on-grade. Design analysis proved that all elements were adequately designed for gravity and lateral forces.

introduction

The Granby Tower (*fig 1*) is a proposed mixed-use, luxury, high rise located in the downtown historic district of Norfolk, Virginia. Historically Granby Street was the premier shopping, dining, gathering and theatre corridor, and these luxuries were supplemented by the direct connection to the Elizabeth River waterfront. The conveniences of Granby Street fell out of favor in the 1960's as suburban development between Norfolk and Virginia Beach promised bargain shopping malls. Due to the decline in popularity of a very important landmark and cultural center, city officials began reviving the city center in the 1970's and are still working to regain the prestige that Granby Street held in the early 1900's.

Granby Tower will be the tallest building in Norfolk upon completion and will provide roughly 300 luxury apartments with views of downtown Norfolk and the Elizabeth River, 6 stories of parking, a roof top fitness center and pool, leasable office space. It is becoming increasingly popular in the Norfolk and Virginia Beach areas to build above parking structures for a number of reasons. One of the most obvious reasons is that you must provide parking space, and since the site has little open space for a free standing garage, the best way to maximize your profit is to utilize the lower floors for parking. The second main reason for an above ground parking structure housed within the buildings structure is due to the sandy soil conditions and high ground water table that don't allow for deep foundations. Most designs, especially heavy concrete structures, require slab on grade with deep piles to penetrate the deep Yorktown Strata layer that is buried beneath layers of unstable sand and clay.

The lateral force resisting system at Granby Tower is designed as a concrete shear wall core which helps to maximize leasable space while keeping most views unobstructed. The floor framing system is a two-way flat-plate post-tensioned slab with minimal drop panels to capitalize on floor to ceiling height. The longest span seen by the slab is 30 feet with typical bays at 26' x 30'. These design features will allow spaces to feel spacious and elegant and with a design focused on luxury, it is easy to see that Granby Tower will stand as a landmark for the city to celebrate a vibrant history and a promising future.

This report will analyze the existing condition of Granby Tower to determine if the structure is adequate for gravity and lateral loads. Wind and seismic forces will be calculated for the structure and then used to analyze the capacity of the floor plate, columns, and shear walls.

fig 1 – rendering of Granby Tower

structural overview

foundation

To determine the soil bearing capacity, sixteen (16) 100 to 110-foot deep Standard Penetration Test borings were drilled within the proposed Granby Tower site. Borings were conducted in accordance with ASTM D 1586 standards and performed with rotary wash drilling procedures to analyze the soil types at 5 foot integrals. Soil tests determined that the first 20 feet of most samples consisted of silty fine sand (SM) or poorly graded fine sand (SP-SM). The next 25 feet of bore was composed of clay (CL) followed by 55 feet of poorly graded fine to coarse sand (SP-SM) and/or silty fine sand (SM). Due to the composition of the soil and location of the groundwater table (6 to 7 feet below grade), the geotechnical engineer recommended a deep pile foundation system with driven, precast, pre-stressed, concrete piles since shallow foundations would result in excessive settlements due to the extreme building weight.

To determine the feasibility and required depths of the piles, fifteen test piles were driven with and evaluated with a Pile Driving Analyzer. The analysis dictated the use of 12" square, precast, pre-stressed concrete piles (SPPC) at 80 feet deep with 100 ton capacity and 14" SPPC at 90 feet with 140 ton capacity. Roughly 1000 piles were driven throughout the site with 255-14" SPPC piles supporting the ordinary shear wall core. Due to the lateral forces seen by the shear walls, the outer 156 piles are designed for tension. The pile cap supporting the shear wall is 10 feet thick with a 28-day compressive strength (f'c) of 5000 psi and #10 and #11 reinforcing on top and bottom, while all other pile caps will be designed with an f'c of 4000 psi and # 7 and #8 reinforcing.

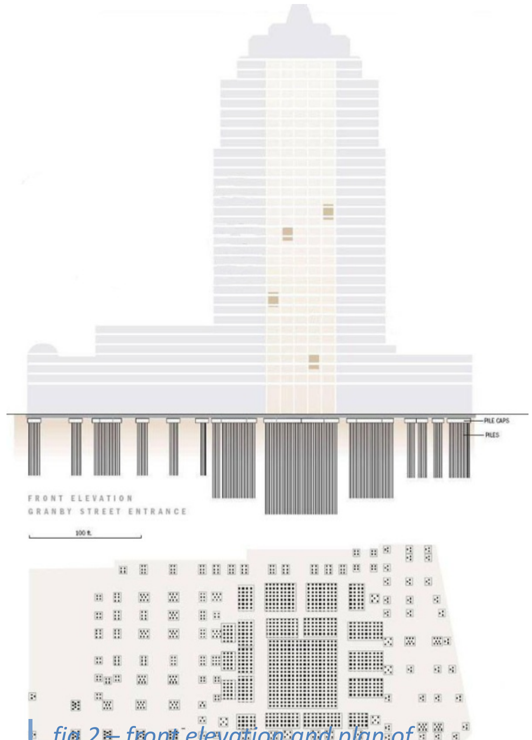


fig 2 – front elevation and plan of piles for Granby Tower. source: Abiouness, Cross and Bradshaw, Inc.

The slab on grade is 5" thick, reinforced with 6x6-W2.9xW2.9 welded wire fabric over a 10 mil polyethylene vapor barrier. The geotechnical engineer specified the slab to be placed over 4" porous fill with less than 5% passing the No. 200 sieve to act as a capillary barrier. The slab should also be "floating" in the sense that it is not rigidly connected to columns or foundations to reduce cracking.

floor system

The floor system for the Granby Tower consists of a two-way flat plate post tensioned slab designed in accordance with the Post-Tensioning Manual 6th Edition by the Post-Tensioning Institute and ACI 318-02. All slabs are designed with a 28-day compressive strength (f'c) of 5000 psi, and the first 7 levels of the tower require a 9" slab while the remaining levels are designed as an 8" slab. Tendons for post-tensioning will be ½" diameter (ø), 7-wire, low relaxation strand, fully encased in grease with a minimum sheathing thickness of 50mm.

Maximum sag for tendons will be 5 ½" and supported by chairs or bolsters. Post-tensioning will occur when the concrete has reached 75% of its designed f'c, and all of the uniform tendons shall be stressed before banded tendons. Uniform tendons are even distributed through the north-south (long) direction with a maximum span of 26' while banded tendons run east-west (short direction) along column lines with a maximum span of 30'.

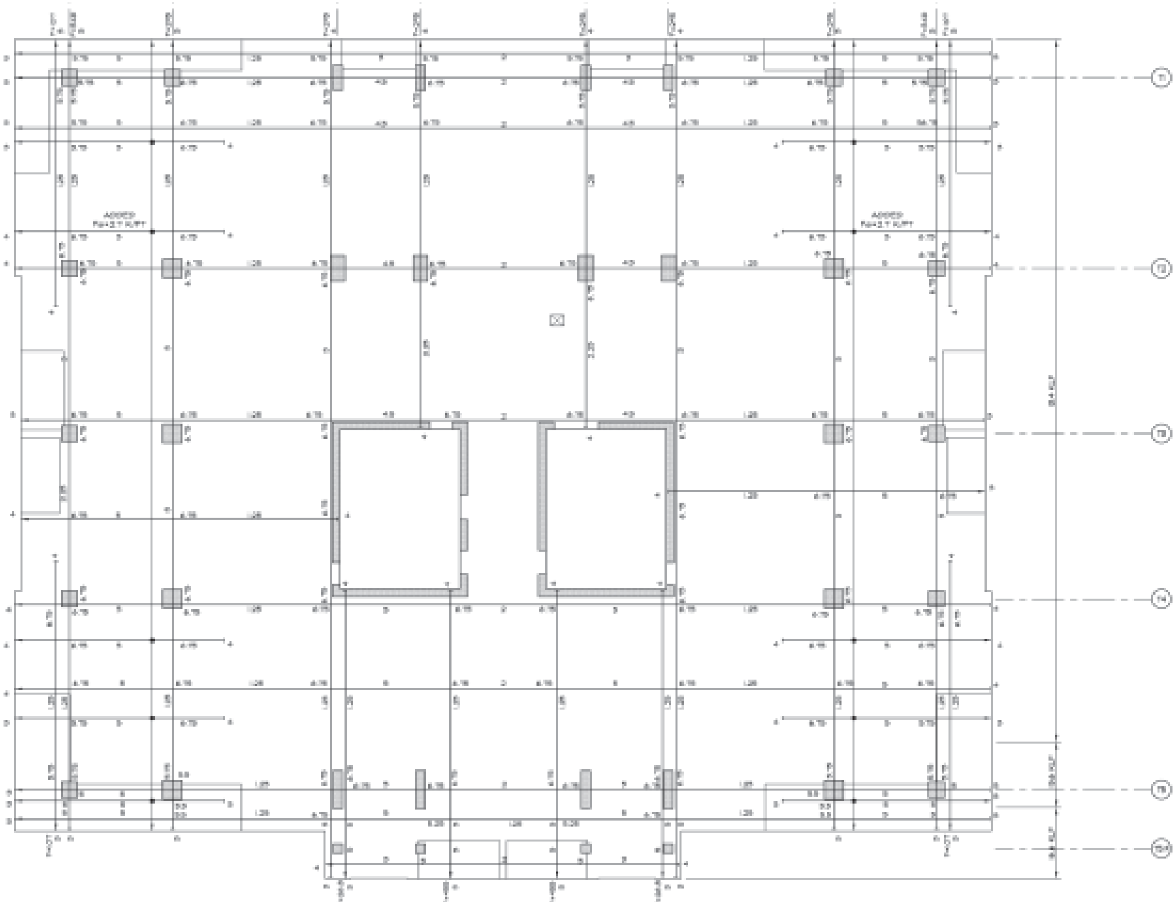


fig 3 – typical post-tensioning plan for levels 8 through 12. Plan and True North →**N**

columns

Gravity columns are laid out on a fairly regular grid with the largest bay at 26'x30'. Roughly 32 columns run the full building height with some of the exterior columns terminating at the buildings first significant set-back on the 29th floor. Most columns are square reinforced columns with rebar ranging from #7 to #10, but rectangular columns with the strong axis in the short building direction (east-west) are architecturally situated in central east and west apartments. Columns above the parking garage (Level 7) are designed with f'c = 5000 psi, and columns between Level 6 and the foundation are designed with f'c = 6500 psi. Banded tendons running through columns should be within 1.5 x T (thickness slab) of the column face and placed above other uniform tendons or rebar. Some drop panels are required on upper floors as column sizes decrease and slab edges become flush with exterior columns.

lateral system

The lateral load resisting system of Granby Tower consists of ordinary reinforced concrete shear walls that were designed in accordance to ACI 318-02. These two shear wall cores house the elevators, stairs, electrical and gas lines, and fire dampers. The first 6 levels consist of 24" thick reinforced shear walls with f'c = 8000 psi, while the remaining levels consist of 14" shear walls with 28-day compressive strengths of 6000 (Levels 7 through 23) and 5000 psi (Levels 24 through 34). Typical vertical reinforcement ranges in size and spacing from #10 @ 6" o.c. to #8 @ 12" o.c. while horizontal reinforcement ranges from #6 @ 6" o.c. to #5 @ 12" o.c. Typical end reinforcement consists of ten vertical rebar within a square section determined by the wall width and #4 ties @ 8" o.c vertical spacing from the foundation to Level 7 and #3 ties @ 8" o.c. vertical spacing from Level 7 to 34.

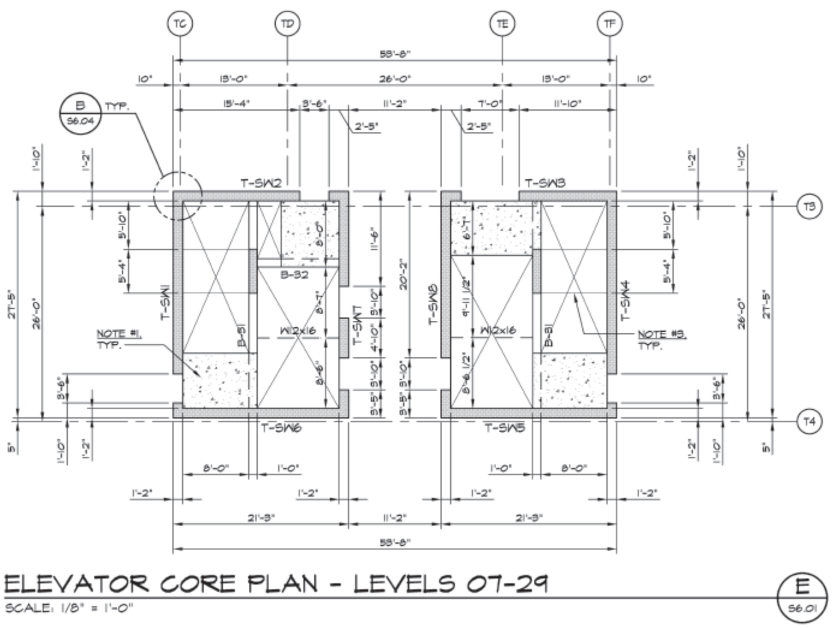


fig 4 – typical plan of shear wall core.

codes and material properties

codes and standards

At the time in which the Abiouness, Cross and Bradshaw began structural design of Granby Tower, the overarching permissible codes for design were the 2000 International Building Code (IBC), which references American Society of Civil Engineers (ASCE) 7-98, and Virginia Uniform Statewide Building Code 2000. Concrete was designed in accordance with American Concrete Institute (ACI) 318-99 and all masonry in accordance with ACI 530-99. Post-tensioning design references the Post-Tensioned Manual by the Post-Tensioned Institute, ACI 318-02, and IBC 2000. All steel design references the American Institute of Steel Construction (AISC) ASD 9th Edition, and cold-formed metal design references the 1996 American Iron and Steel Institute (AISI) Specification.

For my analysis of Granby Tower I utilized more recent building codes such as IBC 2006 and ASCE 7-05. All concrete design was based on ACI 318-05 and while I have not referenced any steel code thus far, I will utilize the Load and Resistance Factor Design information from AISC Thirteenth Edition Steel Manual.

materials

Concrete: Normal Weight Concrete

Foundations	f'c = 4000 psi / 5000 psi
Shear Walls	f'c = 8000 psi / 6000psi / 5000 psi
Slab on Grade	f'c = 4000 psi
Elevated Slabs	f'c = 5000 psi
Columns	f'c = 6500 psi / 5000 psi

Reinforcing Steel

Reinforcing Bar	ASTM A615, Grade 60
Welded Wire Fabric	ASTM A185

Structural Steel

Structural Tubing (HSS)	ASTM A500, Grade B, Fy = 46ksi
W-shapes	ASTM A992, Grade 50, Fy = 50 ksi
Other rolled plates and shapes	ASTM A36, Fy = 36 ksi

loads

dead loads

The dead loads used for design (as shown below) include all structural elements and permanent equipment at its full operating weight as required by ASCE 7-05 § 12.7.2 for effective seismic weight. Normal weight concrete was used for concrete calculations.

Level	Slab	Shear Walls	Columns	Curtain Wall	Beams	Drop Panels	Mech Eq	Total
Spire	0.0	0.0	0.0	0.0	0.0	0.0	0.0	83.0
34	250.8	32.0	3.8	11.0	282.8	0.0	2.3	582.7
33	613.6	280.8	16.5	22.0	155.5	0.0	0.0	1088.4
32	1027.6	303.3	76.1	29.0	361.2	0.0	84.8	1882.0
31	886.0	360.6	98.4	94.0	124.3	0.0	0.0	1563.3
30	1509.8	312.9	76.1	72.7	71.8	7.6	0.0	2050.9
29	1556.5	312.9	110.7	82.0	23.5	25.6	0.0	2111.2
28	1556.5	312.9	164.5	82.0	14.5	18.1	0.0	2148.5
27	1556.5	312.9	182.2	82.0	14.5	18.1	0.0	2166.2
26	1556.5	312.9	182.2	82.0	14.5	18.1	0.0	2166.2
25	1587.3	312.9	182.2	82.0	14.5	18.1	0.0	2197.0
24	1911.9	312.9	189.1	82.0	37.0	7.5	0.0	2540.4
23	1883.0	312.9	223.7	88.4	10.0	0.0	0.0	2518.0
22	1883.0	312.9	223.7	88.4	10.0	0.0	0.0	2518.0
21	1883.0	312.9	223.7	88.4	10.0	0.0	0.0	2518.0
20	1883.0	312.9	223.7	88.4	10.0	0.0	0.0	2518.0
19	1883.0	312.9	223.7	88.4	10.0	0.0	0.0	2518.0
18	1883.0	312.9	223.7	88.4	10.0	0.0	0.0	2518.0
17	1883.0	312.9	223.7	88.4	10.0	0.0	0.0	2518.0
16	1883.0	312.9	223.7	88.4	10.0	0.0	0.0	2518.0
15	1883.0	312.9	223.7	88.4	10.0	0.0	0.0	2518.0
14	1883.0	312.9	223.7	88.4	10.0	0.0	0.0	2518.0
13	1892.2	312.9	223.7	88.4	10.0	0.0	0.0	2527.2
12	1892.2	312.9	223.7	88.4	10.0	0.0	0.0	2527.2
11	1892.2	312.9	387.5	88.4	10.0	0.0	0.0	2691.0
10	1892.2	312.9	387.5	88.4	10.0	0.0	0.0	2691.0
9	1892.2	312.9	387.5	88.4	10.0	0.0	0.0	2691.0
8	1892.2	312.9	387.5	88.4	10.0	0.0	0.0	2691.0
7	1889.3	372.8	453.6	103.6	10.0	0.0	0.0	2829.3
6	2125.5	541.5	404.7	30.8	16.1	0.0	0.0	3118.6
5	2125.5	541.5	404.7	30.8	16.1	0.0	0.0	3118.6
4	2125.5	541.5	404.7	30.8	16.1	0.0	0.0	3118.6
3	2125.5	541.5	404.7	30.8	16.1	0.0	0.0	3118.6
2	2125.5	541.5	404.7	30.8	16.1	0.0	0.0	3118.6
1	1944.4	596.9	434.3	33.0	42.2	0.0	0.0	3050.8
SOG	0.0	841.2	612.0	23.3	0.0	0.0	0.0	1476.5
						Total Dead	**Load (k)**	**84528.0**

live loads

An extensive list of the live loads used in design of Granby Tower was provided with the structural general notes, but since my analysis was carried out with current codes, all assumed live loads were verified with ASCE 7-05.

Live Loads

Roofs	30 psf
Residential Floors	40 psf
Garage	50 psf
Balconies	100 psf
Public Rooms and Corridors	100 psf
Stairs	100 psf
Roof Garden	100 psf
Mechanical and Electrical Rooms	125 psf

snow loads

Norfolk, Virginia experiences mild winters with an expected ground snow load, $P_g = 10$ psf. There are very few flat or low sloped areas for snow to collect on the tower due to the slope of the spire. The exposed portion of the parking structure would be susceptible to some drift possibilities so the flat roof snow load (P_f) was calculated to be 6.3 psf. The calculations below were performed in accordance with ASCE 7-05 § 7.3.

Snow Load Calculations

Ground Snow Load, P_g	10 psf
Importance Factor, I	1.0
Snow Exposure Factor, C_e	0.9
Thermal Factor, C_t	1.0
Flat Roof Snow Load, $Pf = 0.7 * P_g * I * C_e * C_t =$	6.3 psf

wind loads

Wind analysis was completed using ASCE 7-05 § 6.5 Method 2 – Analytical Procedure. This Method was necessary over § 6.4 Method 1 – Simplified Procedure because my building height was greater than 60 feet and deemed partially enclosed by the designers. To maintain consistency with the proposed design, I elected to share many assumptions that the designer chose for their wind analysis.

General Information	Value	Source
Occupancy Category	II	General Structural Notes
Importance Factor	1.0	General Structural Notes
Basic Wind Speed, V	110 mph	General Structural Notes
Exposure Category	C	General Structural Notes
Enclosure Classification	Partially Enclosed	General Structural Notes
Internal Pressure, GC_{pi}	± 0.55	General Structural Notes

Detailed calculations implementing these assumptions are provided in appendix a. External Pressure Coefficients (C_p) and Gust Factors (G_f) were calculated using the Analytical Procedure § 6.5.11.2 which references Fig 6-6 and § 6.5.8 respectively. Pressures vary depending on the directionality of the wind based on the effective length and width of the building that the wind sees. A summary of the values needed to derive lateral wind pressures are listed below.

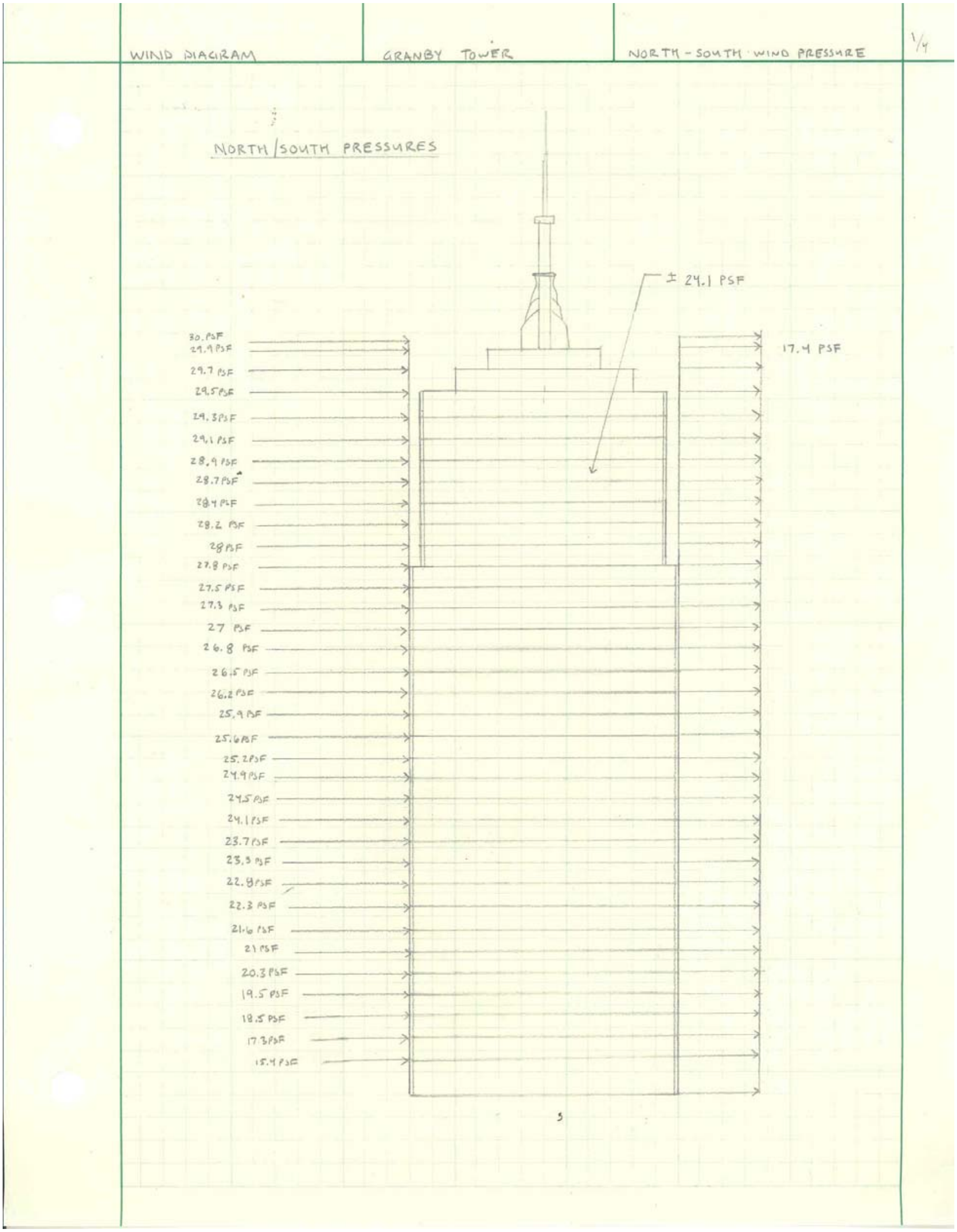
Factor	N-S	E-W	Source
C_p			
Windward	0.8	0.8	ASCE 7-05 § 6.5.11.2, Fig 6-6
Leeward	-0.465	-0.5	ASCE 7-05 § 6.5.11.2, Fig 6-6
Sidewall	-0.7	-0.7	ASCE 7-05 § 6.5.11.2, Fig 6-6
G_f	0.856	0.855	ASCE 7-05 § 6.5.8

As the next couple pages of calculations and diagrams display, wind pressures in the east-west direction are the controlling lateral load. East-West wind produces a Base Shear (V_b) of 2210 kips while base shear in the north-south direction is 1888 kips. This outcome is expected since the east-west faces have a larger surface area and could accrue more wind shear, which results in a higher base shear force.

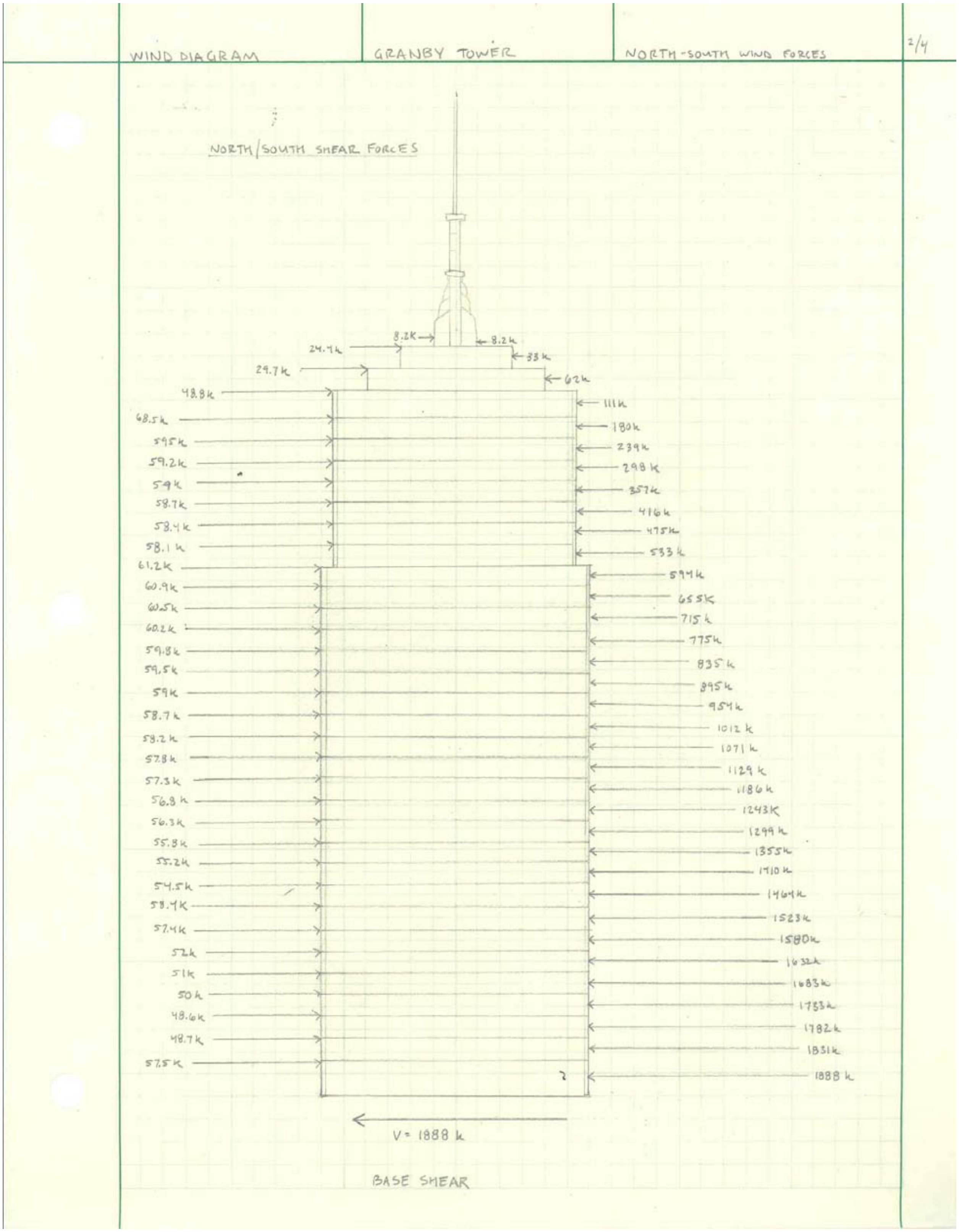
Wind Pressures (psf)

Story	h_x (ft)	K_z	q_z	N-S Windward	N-S Leeward	N-S Side Wall	E-W Windward	E-W Leeward	E-W Side Wall	GC_{pi} (±)
Spire btm.	367.41	1.66	43.83	30.012	-17.444	-26.260	29.977	-18.735	-26.230	24.10
34	361.25	1.66	43.67	29.905	-17.444	-26.260	29.870	-18.735	-26.230	24.10
33	349.00	1.65	43.35	29.689	-17.444	-26.260	29.654	-18.735	-26.230	24.10
32	338.75	1.64	43.08	29.503	-17.444	-26.260	29.468	-18.735	-26.230	24.10
31	325.50	1.62	42.72	29.256	-17.444	-26.260	29.222	-18.735	-26.230	24.10
30	315.25	1.61	42.44	29.060	-17.444	-26.260	29.026	-18.735	-26.230	24.10
29	305.00	1.60	42.14	28.858	-17.444	-26.260	28.824	-18.735	-26.230	24.10
28	294.75	1.59	41.84	28.651	-17.444	-26.260	28.618	-18.735	-26.230	24.10
27	284.50	1.58	41.53	28.439	-17.444	-26.260	28.405	-18.735	-26.230	24.10
26	274.25	1.57	41.21	28.220	-17.444	-26.260	28.187	-18.735	-26.230	24.10
25	264.00	1.55	40.88	27.994	-17.444	-26.260	27.962	-18.735	-26.230	24.10
24	253.75	1.54	40.54	27.762	-17.444	-26.260	27.729	-18.735	-26.230	24.10
23	243.50	1.53	40.19	27.522	-17.444	-26.260	27.490	-18.735	-26.230	24.10
22	233.25	1.51	39.83	27.274	-17.444	-26.260	27.242	-18.735	-26.230	24.10
21	223.00	1.50	39.45	27.017	-17.444	-26.260	26.986	-18.735	-26.230	24.10
20	212.75	1.48	39.06	26.751	-17.444	-26.260	26.720	-18.735	-26.230	24.10
19	202.50	1.47	38.66	26.474	-17.444	-26.260	26.443	-18.735	-26.230	24.10
18	192.25	1.45	38.24	26.186	-17.444	-26.260	26.156	-18.735	-26.230	24.10
17	182.00	1.44	37.80	25.886	-17.444	-26.260	25.856	-18.735	-26.230	24.10
16	171.75	1.42	37.34	25.572	-17.444	-26.260	25.542	-18.735	-26.230	24.10
15	161.50	1.40	36.86	25.243	-17.444	-26.260	25.213	-18.735	-26.230	24.10
14	151.25	1.38	36.36	24.897	-17.444	-26.260	24.868	-18.735	-26.230	24.10
13	141.00	1.36	35.82	24.532	-17.444	-26.260	24.503	-18.735	-26.230	24.10
12	130.75	1.34	35.26	24.145	-17.444	-26.260	24.117	-18.735	-26.230	24.10
11	120.50	1.32	34.66	23.733	-17.444	-26.260	23.706	-18.735	-26.230	24.10
10	110.25	1.29	34.01	23.293	-17.444	-26.260	23.266	-18.735	-26.230	24.10
9	100.00	1.27	33.32	22.820	-17.444	-26.260	22.793	-18.735	-26.230	24.10
8	89.75	1.24	32.57	22.306	-17.444	-26.260	22.280	-18.735	-26.230	24.10
7	77.75	1.20	31.60	21.642	-17.444	-26.260	21.617	-18.735	-26.230	24.10
6	67.50	1.17	30.68	21.008	-17.444	-26.260	20.983	-18.735	-26.230	24.10
5	57.25	1.13	29.63	20.292	-17.444	-26.260	20.268	-18.735	-26.230	24.10
4	47.00	1.08	28.43	19.466	-17.444	-26.260	19.443	-18.735	-26.230	24.10
3	36.75	1.03	26.99	18.484	-17.444	-26.260	18.462	-18.735	-26.230	24.10
2	26.50	0.96	25.20	17.254	-17.444	-26.260	17.234	-18.735	-26.230	24.10
1	15.50	0.85	22.51	15.412	-17.444	-26.260	15.394	-18.735	-26.230	24.10
SOG	0.00	0.00	0.00	0.000	0.000	0.000	0.000	0.000	0.000	0.000

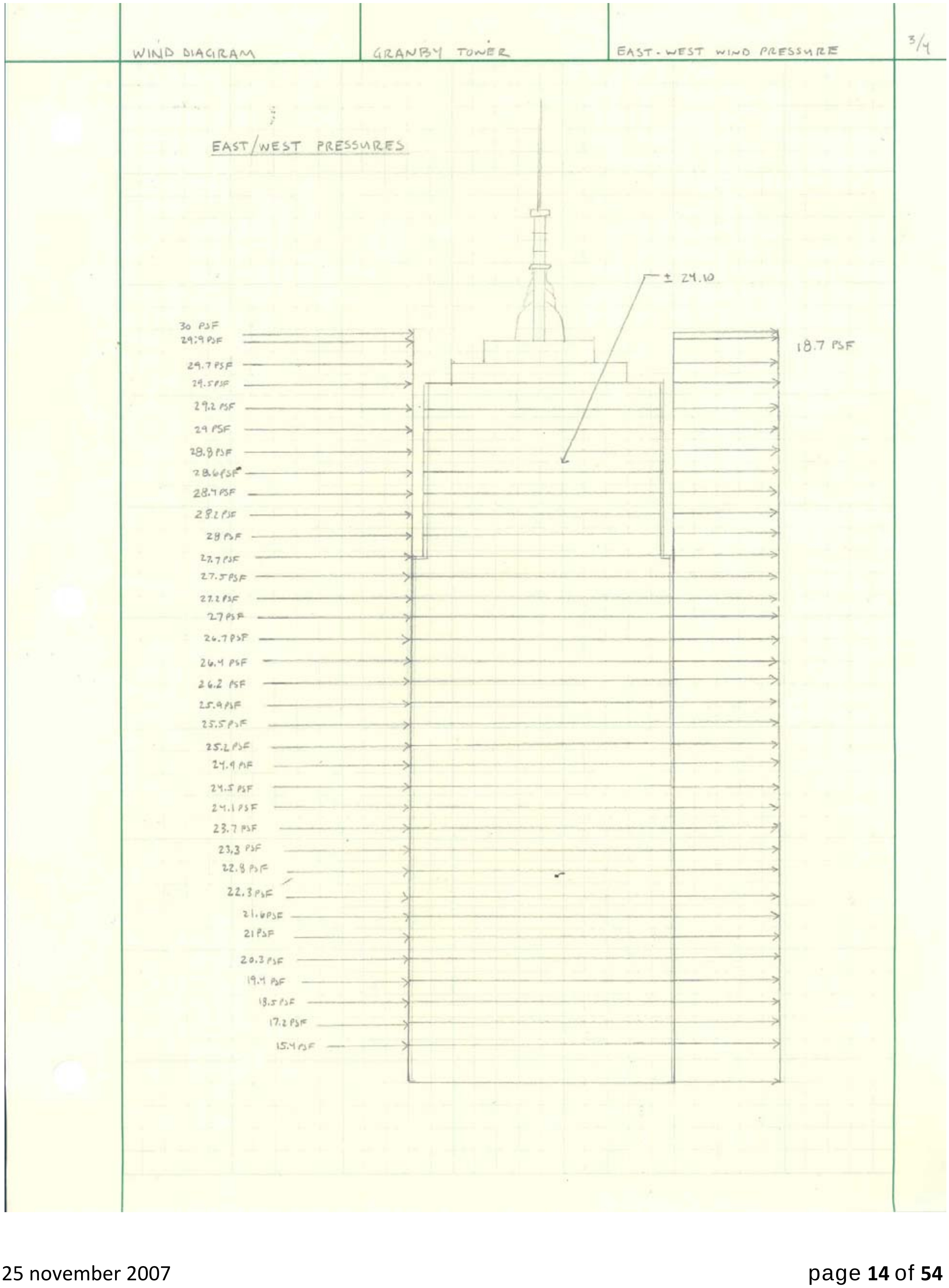
North – South Wind Diagram



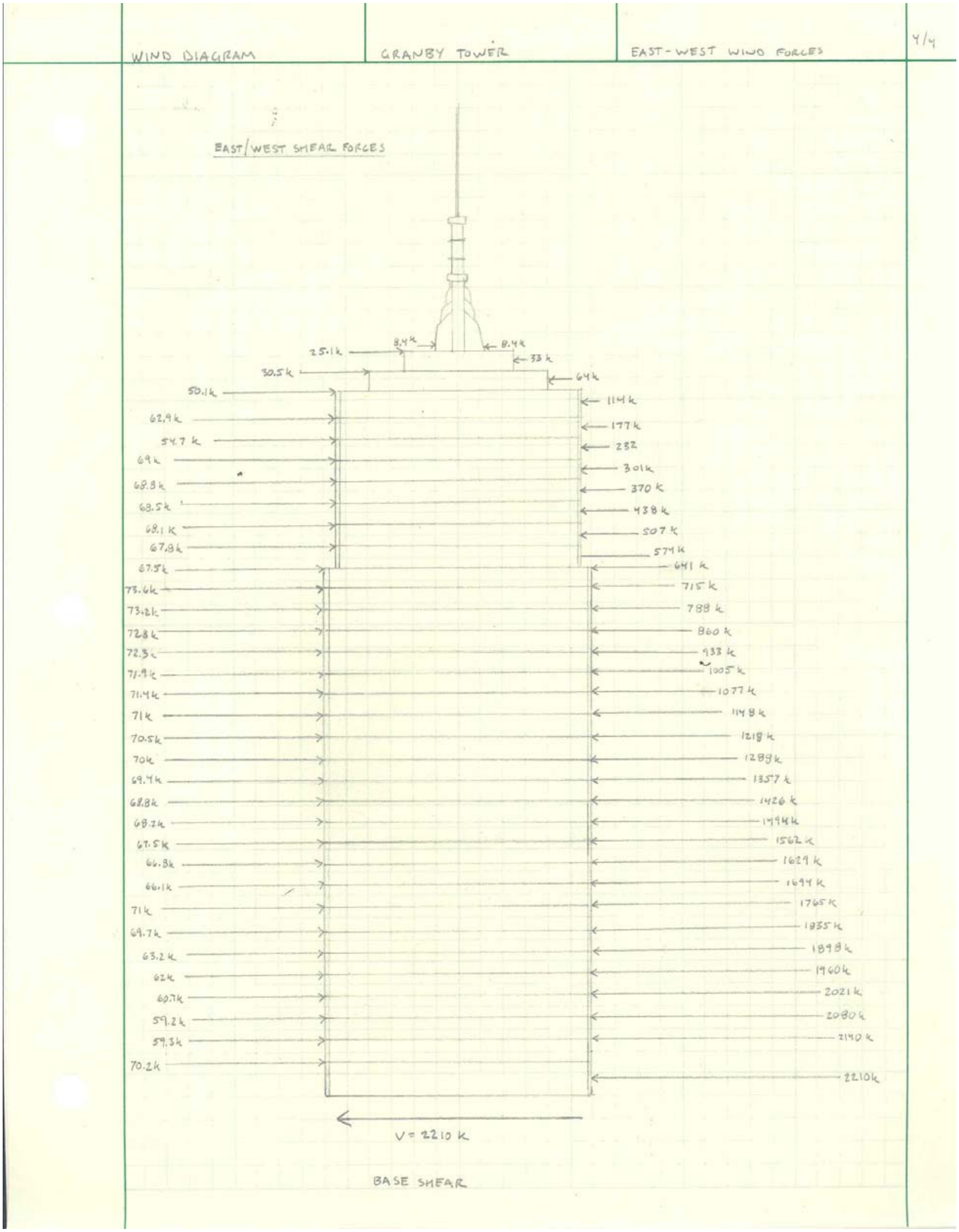
North – South Shear Forces



East – West Wind Diagram



East – West Shear Forces

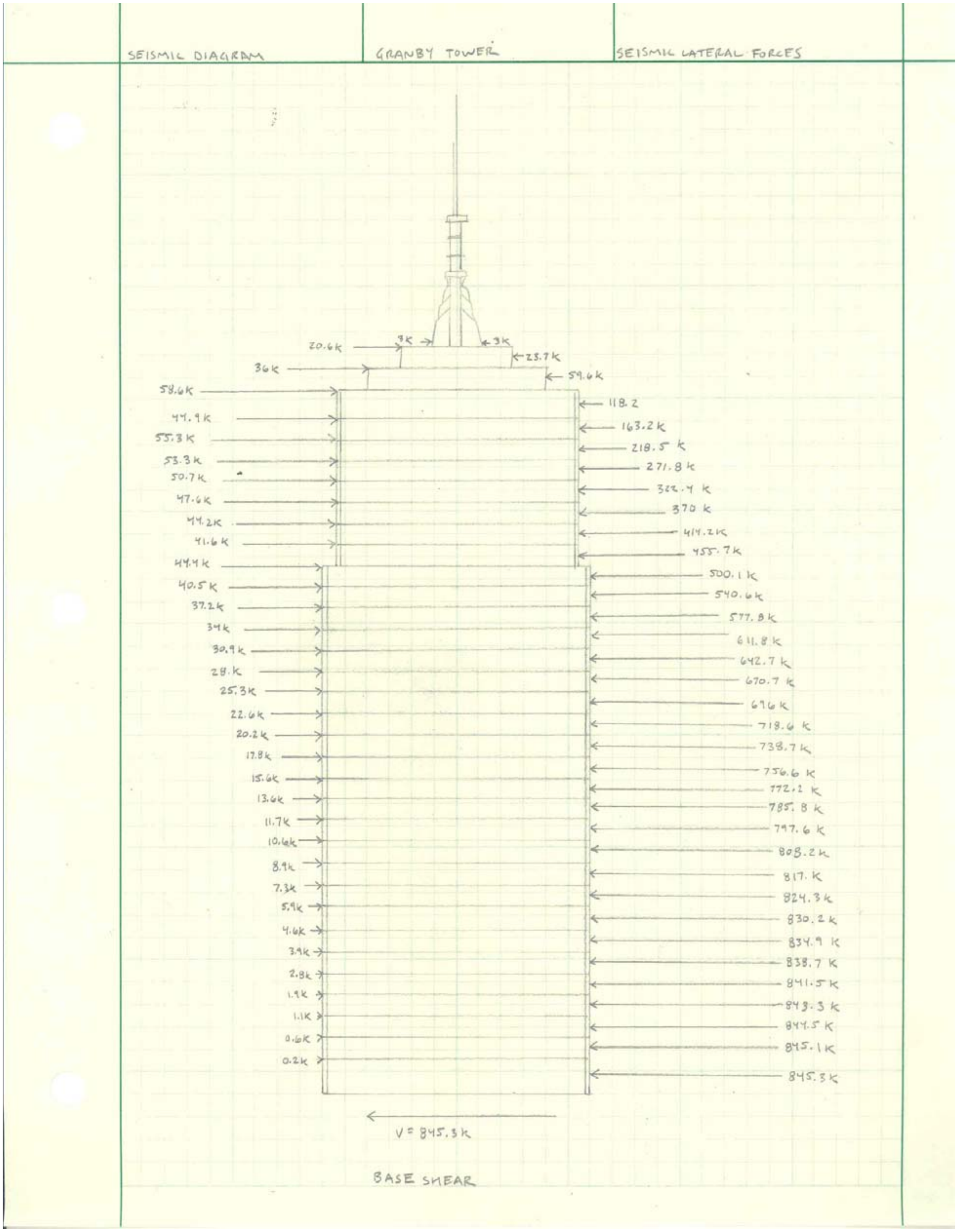


seismic loads

To calculate the seismic forces as seen by the Granby Tower I refrenced ASCE 7-05, §11 & §12 and IBC 2006. A very helpful tool for determing some seismic values was provided by the United States Government Seismic Design Value for Buildings (http://earthquake.usgs.gov/research/hazmaps/design). The USGS web site uses the latitude and longitude of the speific site to determine the mapped and adjusted spectral response accelerations depending on site class. The design engineer also provided some insight as to some of the values that were used in their seismic calcuation, so I made sure to check those values with some more current references. As the table shows below the siesmic base shear, V_b, was 845 kips. This was much lower than the base shear related to wind due to the favorable site class classification, the ordinary reinforced shear walls which have a response modification factor of 5, and building's location along the mid-Atlantic results in a higher wind speed.

Input	Value	Source
Occupancy Category	II	ASCE 7-05
Importance Factor	1.0	ASCE 7-05
Soil Site Class	D	Geotech Report
Seismic Design Category	B	ASCE 7-05
S_S	0.118	USGS.gov
S_1	0.048	USGS.gov
Fa	1.6	ASCE 7-05, Tbl 11.4-1
Fv	2.4	ASCE 7-05, Tbl 11.4-2
S_{DS}	0.126	ASCE 7-05
S_{D1}	0.077	ASCE 7-05
R	5	ASCE 7-05, Tbl 12.2-1
h_n	361.25	
C_t	0.02	ASCE 7-05, Tbl 12.8-2
x	0.75	ASCE 7-05, Tbl 12.8-2
T_a	1.66	
C_u	1.7	ASCE 7-05, Tbl 12.8-1
T	2.82	
T_L	8	ASCE 7-05, Fig 22-15
C_S	0.01	ASCE 7-05, Eq 12.8-5
k	2	ASCE 7-05, Sec 12.8.3
Effective Seismic Weight (W)	84528 k	
V_b	**845.3 k**	appendix c

Seismic Shear Forces



spot checks

post tension

To analyze the two-way flat-plate post-tension system in Granby Tower, I referenced the 6th Edition Post-Tensioned Manual by the Post-Tensioned Institute. This analysis considers the effect that stressing uniform tendons have on the frame through additional secondary moments which are caused by tendon eccentricity. Therefore the stiffness of the columns for a typical bay was taken into consideration through equivalent frame analysis.

My analysis of a typical three-bay grid on the 8th level calls for an 8" slab with 22 tendons uniformly distributed in the north-south direction with a force of 19.65 k/ft throughout the length. Banded tendons will run east-west over the column lines and reinforcing around the columns consists of (7) #5 in plane with the banded tendons, and spaced @ 6" o.c in both directions.

The only discrepancy between my analysis results and the structural engineers design occur in the force required to stress the uniform tendons. The plans call for 16.1 k/ft along the north wall with some minor increases at the east façade due to cantilevers. The difference in required stressing could be due to some of the assumptions I chose for analysis. I assumed all columns to be 30"x 30" square while in actuality the centrally located columns were 48"x 27". Originally I assumed this to be the result of any discrepancies, but I think the main difference between design results is because I analyzed a "typical" bay and not the entire floor plate. Other than this all other aspects of the design are adequate.

column

Granby Tower is supported by 32 gravity columns that range in size from 72"x 18" or 36"x 36" at the foundation to 18"x 18" at the 34th level. I chose to analyze a 36"x 36" column on Level 8 which was part of the grid I analyzed for post-tensioning design. While I assumed 30"x 30" columns in my design for a conservative approach, the moment occurring in that column due to gravity loads and secondary moments would be much the same. Because I analyzed a full height, interior column, I was able to reduce live loads to 40% of code requirements. The only inconsistency between my analysis and the actual design involves the size and number of reinforcing bars. The proposed design calls for slightly larger and a tighter spacing than I required but this could be due to consistency with the rest of the design or industry standard that I was not aware of. Therefore, column design is adequate.

shear wall

The lateral force resisting system in Granby Tower is comprised of two ordinary reinforced concrete shear wall cores. For my analysis I analyzed only the four shear walls in the east-west direction since those walls will take the majority of the shear in that direction. A conservative and simplified assumption that I made for my analysis involves considering the interior shear walls as smaller, full height segments instead of the designed shear walls with multiple openings at each floor. This assumption allowed me to hypothetically transfer more of the shear and moment forces to the two exterior 26' shear walls due to rigidity. Rigidity calculations resulted in 36% of the lateral forces transferred to each of the two outer most shear walls (north-most and shout-most walls) while roughly 13-14% of the load was seen by each interior wall. Under normal analysis, all four east-west shear walls would receive roughly a quarter of the lateral forces, but the two outer walls would still receive slightly more load due to effective area.

I chose to analyze the first story shear walls since this is the largest story height and the shear and moment forces would be the highest. The design specifies 24" thick shear walls with 2 curtains of #10 vertical and #6 horizontal reinforcement @ 6" o.c. My shear and moment analysis required only 1 curtain of #7 vertical and #6 horizontal reinforcement @ 6" o.c. Boundary elements were required since I analyzed the walls as straight segments and not part of a rectangular core, but the required corner reinforcing consisting of (10) #10 vertical reinforcement was adequate for the boundary element required in my design.

conclusion

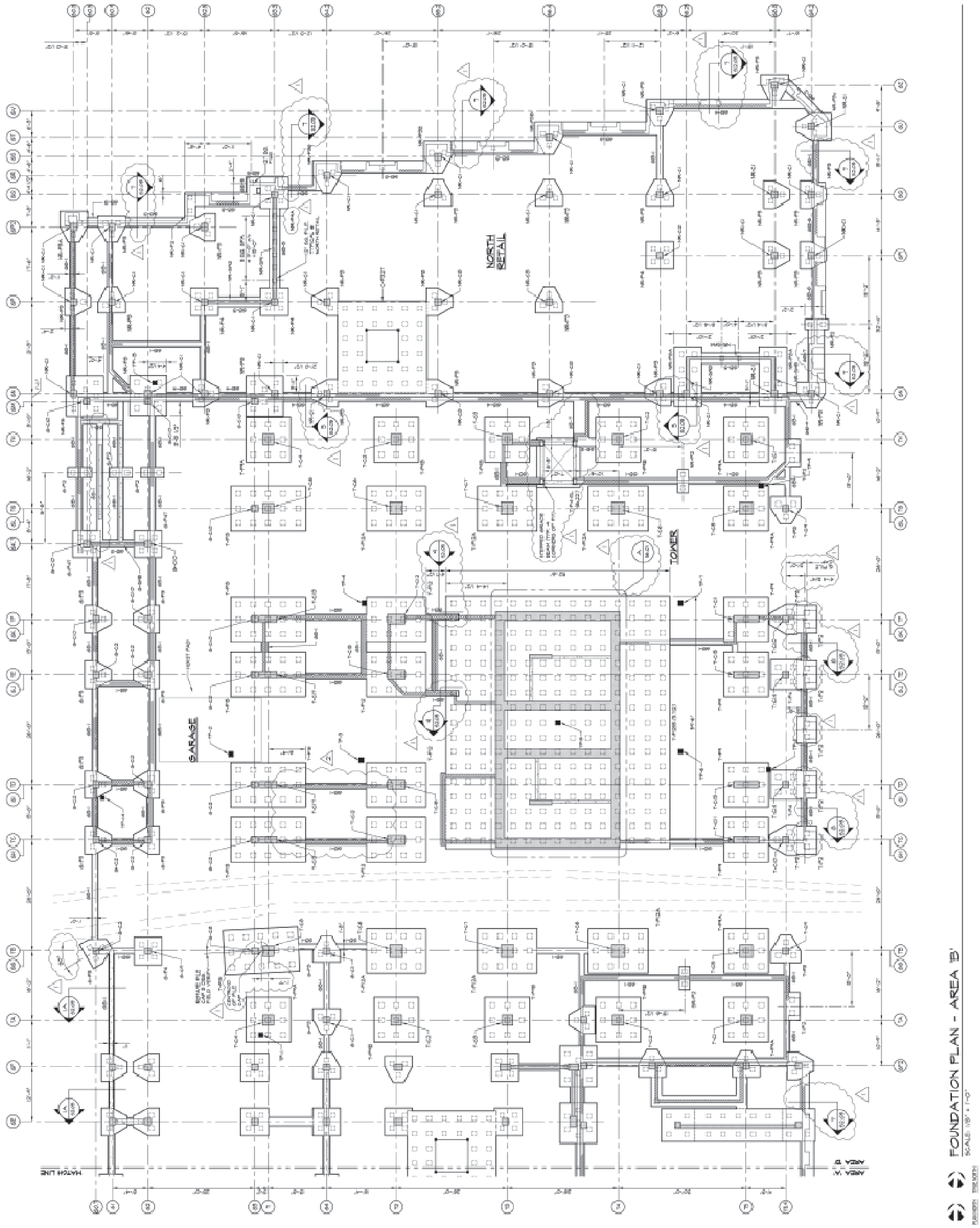
This first technical report proves that the existing structural conditions of Granby Tower, under current code and other design assumptions, are adequate in wind, seismic, and gravity loading. Since wind base shear controlled in the east-west direction, only the shear walls oriented in that direction were considered to resist lateral forces. Shear wall design was found to be adequate for shear and moment capacity. Some members seemed to be designed larger than necessary, and the overdesign of some members could be due to industry standard reinforcing or repetition of design. Discrepancies between my analysis and the structural engineers design could also be due to simplified assumptions made for ease of calculations, but all systems analyzed in this report were sufficiently designed.

appendix a

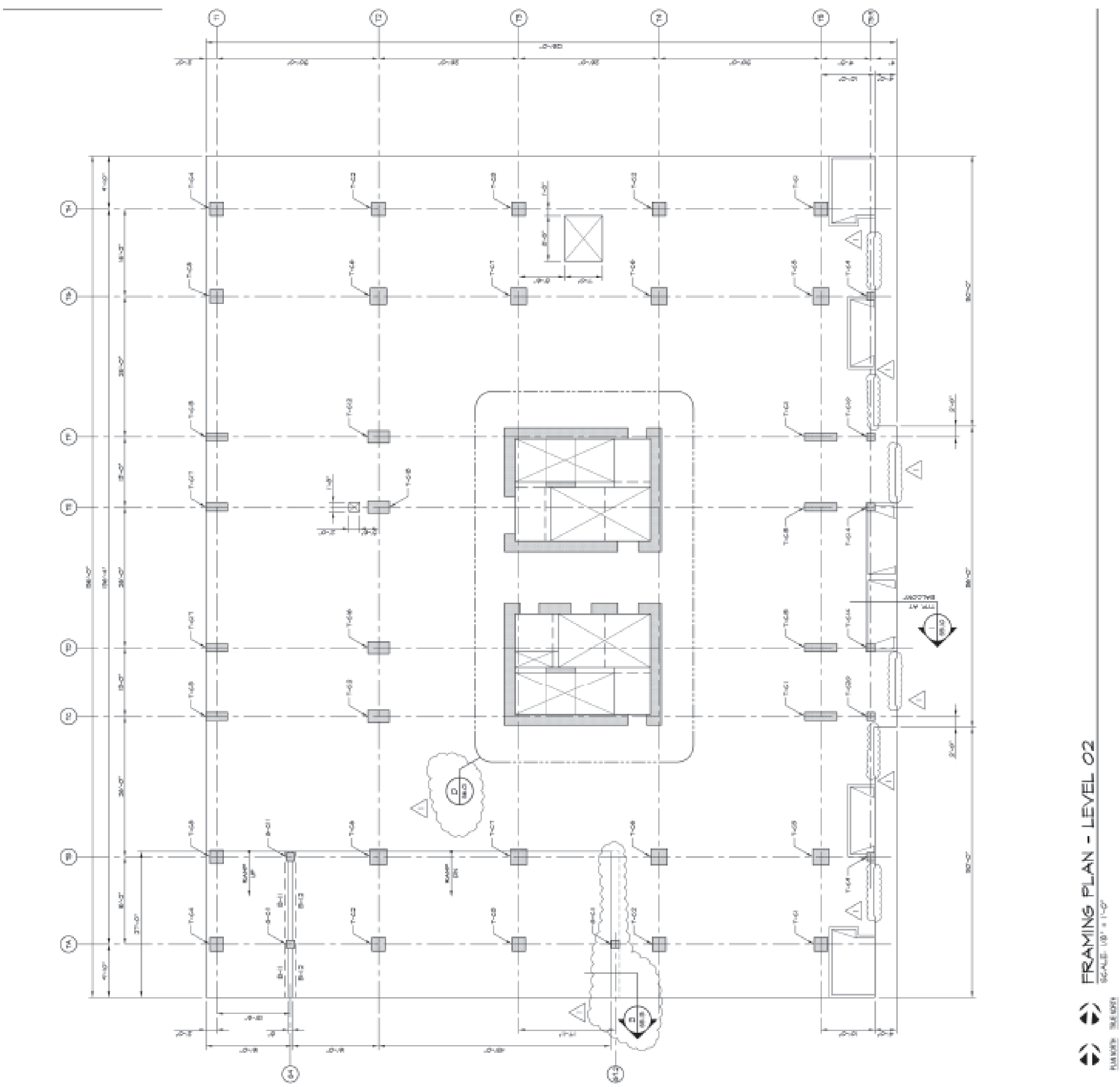
framing plans

The following images were provided by Turner Construction Company for use in Thesis Research. I've chosen to include several typical layouts of framing plans and shear wall layouts for reference. The plans that represent the largest number of floors were included as typical plans.

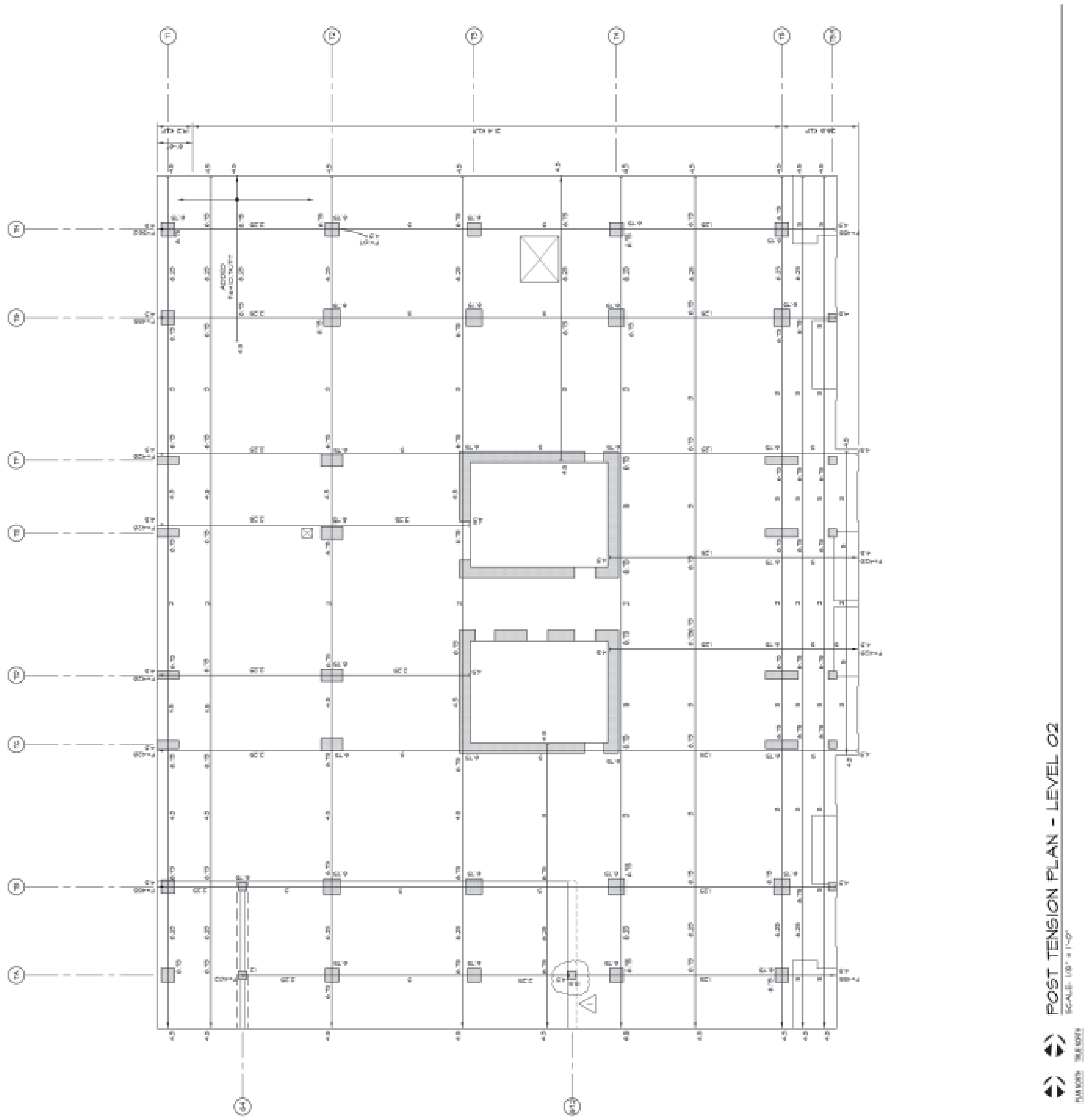
Tower Foundation Plan



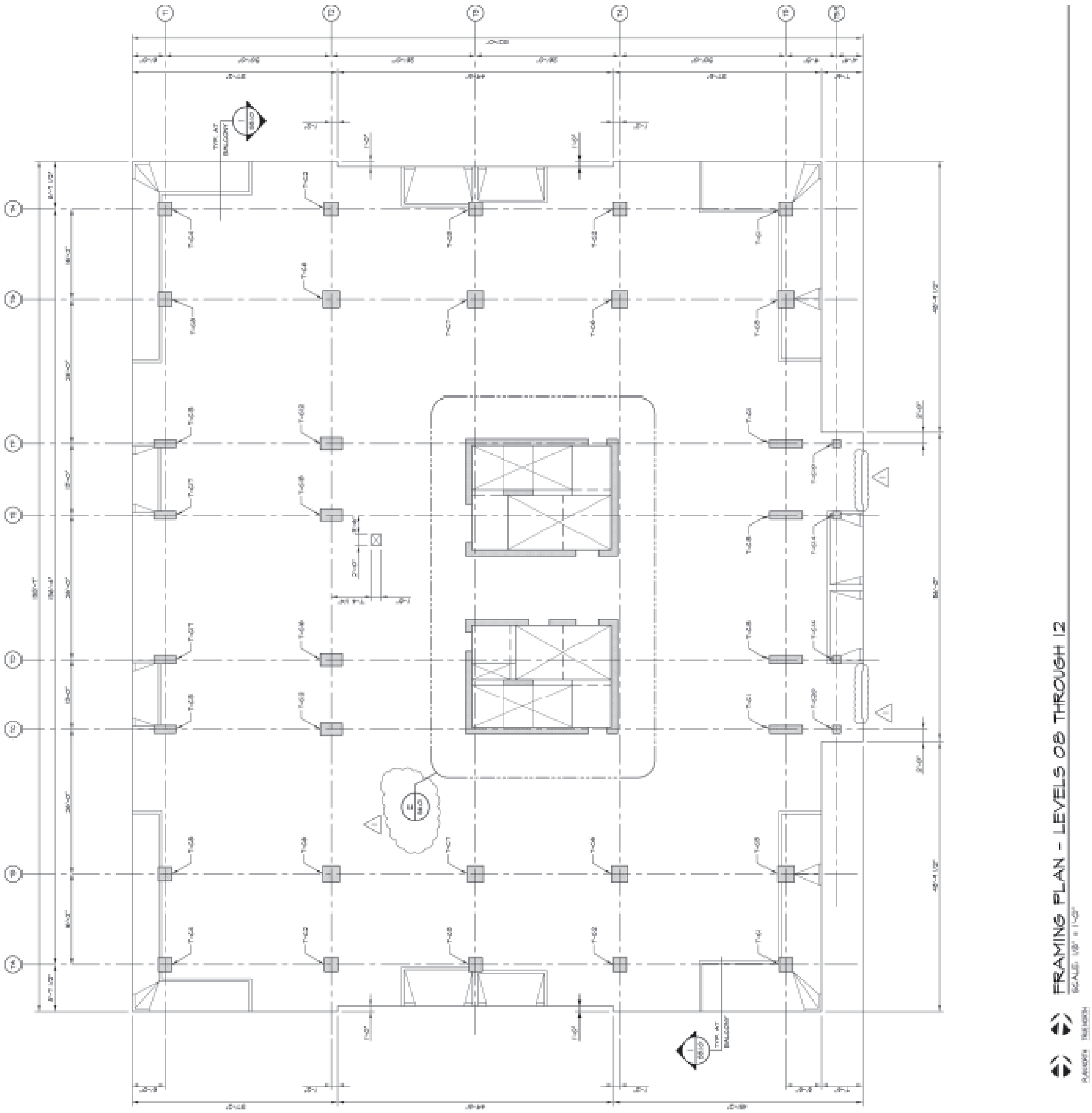
Typical Framing Plan – Level 2 - 7



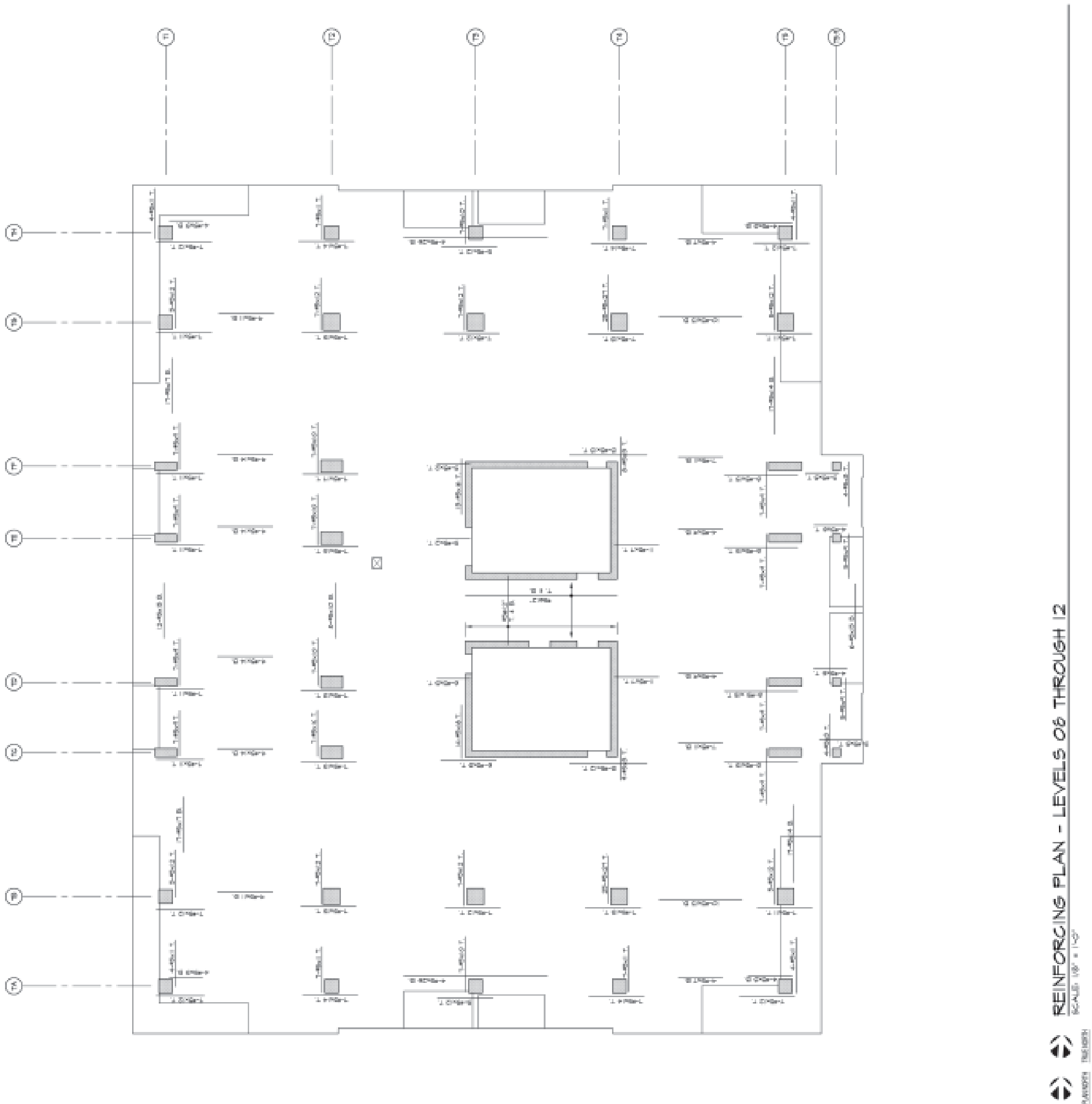
Typical Post Tensioning Plan – Level 2-7



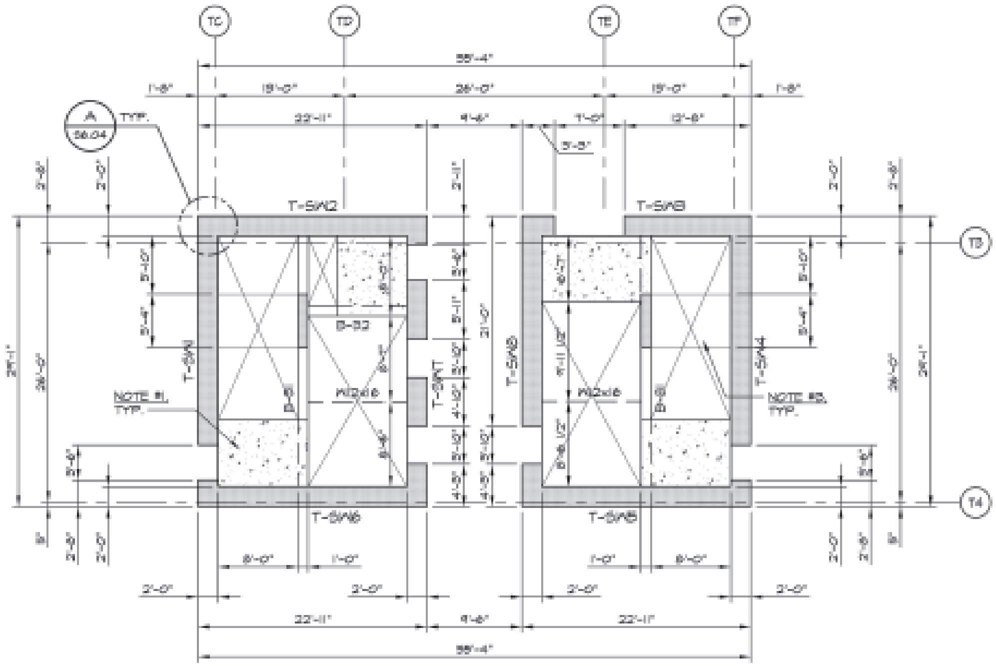
Framing Plan – Level 8 - 12



Typical Reinforcing Plan – Levels 8 - 12

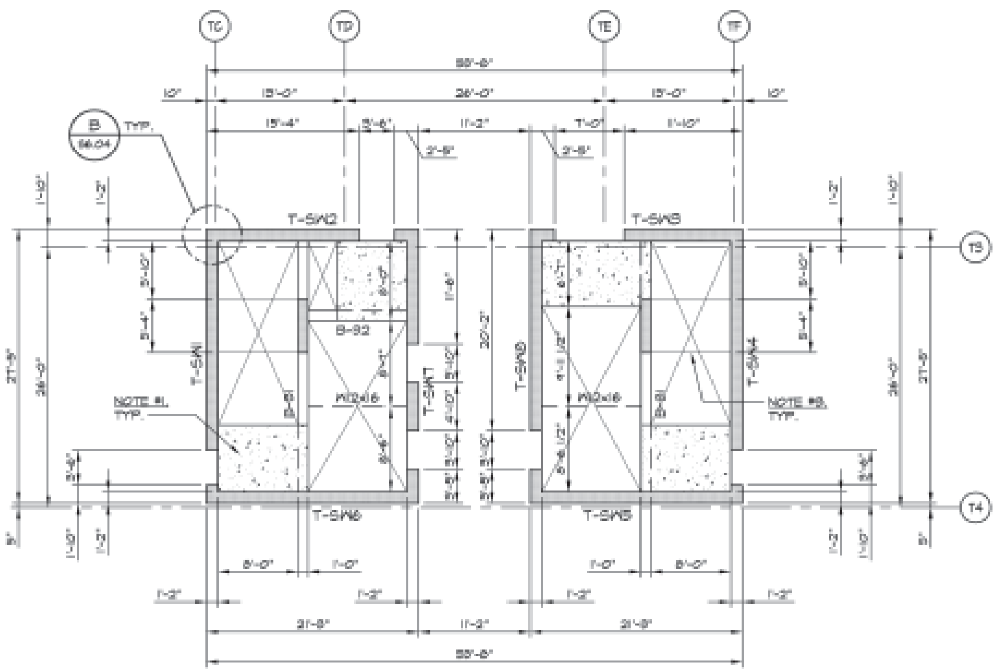


Shear Wall Plans



ELEVATOR CORE PLAN - LEVELS 02-06
SCALE: 1/8" = 1'-0"

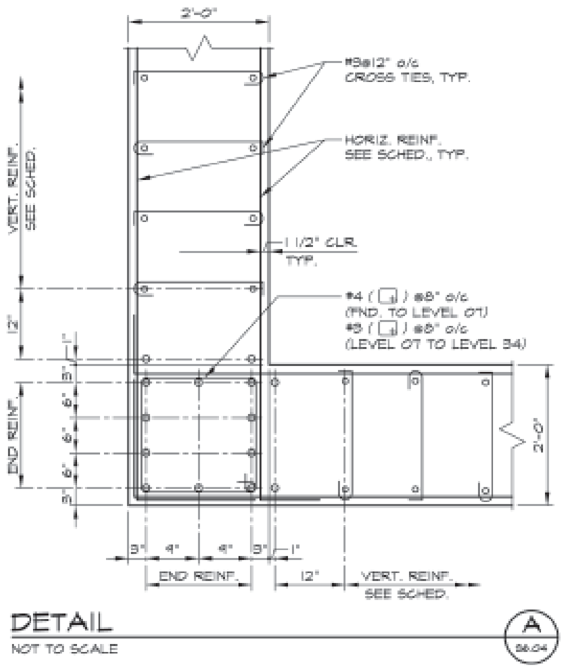
D
S6.01

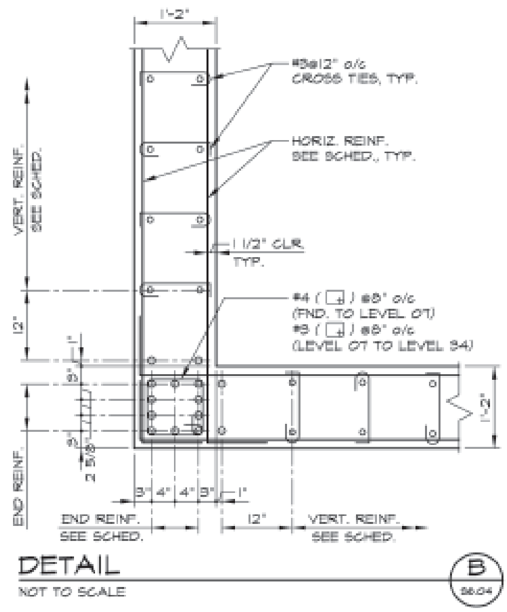


ELEVATOR CORE PLAN - LEVELS 07-29
SCALE: 1/8" = 1'-0"

E
S6.01

Typical Shear Wall Corner Detail





appendix b

wind

General Information	Value	Source
Occupancy Category	II	General Structural Notes
Importance Factor	1.0	General Structural Notes
Basic Wind Speed, V	110 mph	General Structural Notes
Exposure Category	C	General Structural Notes
Directionality Factor, k_d	0.85	ASCE 7-05 § 6.5.4.4
h	367.4 ft	Design
k_h	1.657	ASCE 7-05 § 6.5.6.6
k_z	$2.01(z/z_g)^{2/\alpha}$	ASCE 7-05 § 6.5.6.7
α	9.5	ASCE 7-05 Table 6-2
z_g	900 ft	ASCE 7-05 Table 6-2
k_{zt}	1.0	ASCE 7-05 § 6.5.7
T	2.82 sec	Seismic Calcs
n_1	0.355 Hz	Seismic Calcs
Building Rigidity	Flexible	Seismic Calcs

Factor	N-S	E-W	Source
C_p			
Windward	0.8	0.8	ASCE 7-05 § 6.5.11.2, Fig 6-6
Leeward	-0.465	-0.5	ASCE 7-05 § 6.5.11.2, Fig 6-6
Sidewall	-0.7	-0.7	ASCE 7-05 § 6.5.11.2, Fig 6-6
G_f	0.856	0.855	ASCE 7-05 § 6.5.8

Tower Gust Factor

Item	N-S	E-W
L	155.25	132.08
B	132.08	155.25
h	367.4	367.4
n_1	0.355	0.355
Rigidity	Flexible	Flexible
$\bar{z}$ (ft)	220.45	220.45
c	0.2	0.2
$I_{\bar{z}}$	0.146	0.146
ϵ	0.2	0.2
ℓ (ft)	500	500
$L_{\bar{z}}$	928.18	928.18
Q	0.837	0.834
g_Q	3.4	3.4
g_v	3.4	3.4
g_R	4.08	4.08
$\bar{\alpha}$	9.5	9.5
$\bar{b}$	0.65	0.65
$\bar{V}_{\bar{z}}$	7.181E+09	7.181E+09
N_1	4.588E-08	4.588E-08
R_h	1.315	1.315
R_B	2.164	0.756
R_L	1.081	0.808
R_n	3.428E-07	3.428E-07
β	0.5	0.5
R	0.00142	0.00079
G_f	0.856	0.855

WIND CALCULATIONS | GRANBY TOWER | 1

MAIN WIND FORCE RESISTING SYSTEM

METHOD 2 - ANALYTICAL PROCEDURE

OCCUPANCY CATEGORY = II
IMPORTANCE FACTOR = 1.0 (TABLE 6-1; V > 100 mph)
BASIC WIND SPEED, V = 110 mph
EXPOSURE CATEGORY = C
$K_d = 0.85$
$K_h = 1.657$

→ $h = 367$ ft

ft	Exp C
350	1.64
h	K_h
400	1.69

$K_h = 1.657$

$K_z =$

→ $K_z = 2.01\,(z/z_g)^{2/\alpha}$ ← SPREAD SHEET PER FLOOR

$\alpha = 9.5$
$z_g = 900$ ft

$K_{zt} = 1.0$
$T = 2.82$ sec
$n_1 = 0.355$ HZ
BUILDING = FLEXIBLE

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

$g_Q = 3.4$
$g_v = 3.4$
$g_R = 4.08$ $\qquad g_R = \sqrt{2\ln(3600\, n_1)} + \frac{0.577}{\sqrt{2\ln(3600\, n_1)}}$

N-S	E-W
L = 155'3"	L = 132'1"
B = 132'1"	B = 155'3"

$\bar{V}_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}} V\left(\frac{88}{60}\right) = \boxed{7.18003\text{E}9 \text{ ft/s}}$

$\bar{\alpha} = 9.5$
$\bar{b} = 0.65$
$\bar{z} = 0.6h = 220.45$
$\ell = 500$ ft
$\bar{\epsilon} = 1/5.0 = 0.2$
$c = 0.2$
$L_{\bar{z}} = \ell\left(\frac{\bar{z}}{10}\right)^{\bar{\epsilon}} = 500\left(\frac{220.45}{10}\right)^{0.2} = \boxed{928.18}$

$N_1 = n_1 L_{\bar{z}} / \bar{V}_{\bar{z}} = \boxed{4.589\text{E}-8}$

$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6} = 0.2\left(\frac{33}{220.45}\right)^{1/6} = \boxed{0.146}$

WIND CALCULATIONS | GRANBY TOWER | GUST FACTOR | 2

N-S

R_h: $\eta = 4.6 n_1 h / \bar{V}_{\bar{z}} = 4.6(0.355)(367.41)/(7.18 E9) = 8.356 E{-8}$

$R_h = 1.315$

R_B : $\eta = 4.6 n_1 B / \bar{V}_{\bar{z}} = 4.6(0.355)(132'1'')/\bar{V}_{\bar{z}} = 3.004 E{-8}$

$R_B = 2.164$

R_L: $\eta = 15.4 n_1 L / \bar{V}_{\bar{z}} = 15.4(0.355)(155.25)/\bar{V}_{\bar{z}} = 1.182 E{-7}$

$R_L = 1.081$

$R_n = 7.47 N_1 / (1 + 10.3 N_1)^{5/3}$

$R_n = 3.428 E{-7}$

$\beta = 0.5$

$R = \sqrt{\frac{1}{\beta} R_n R_h R_B (0.53 + 0.47 (R_L))} = \boxed{0.0014}$

$Q = \sqrt{\frac{1}{1 + 0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}} = \sqrt{\frac{1}{1 + 0.63\left(\frac{132.08 + 367.41}{928.18}\right)^{0.63}}} = 0.837$

$G_f = 0.925\left(\frac{1 + 1.7 I_{\bar{z}} \sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1 + 1.7 g_v I_{\bar{z}}}\right) = 0.925\left(\frac{1 + 1.7(0.146)\sqrt{(3.4)^2(0.837)^2 + (4.08)^2(0.0014)^2}}{1 + 1.7(3.4)(0.146)}\right)$

$= \boxed{0.856}$

WIND CALCULATIONS | GRANBY TOWER | GUST FACTOR | 3

E - W

$R_h = 1.315$

$R_B : \eta = 4.6 n_1 B / \bar{V}_{\bar{z}} = 4.6(0.355)(155.25)/(7.18E9) = 3.53\ E-8$

$R_B = 0.756$

$R_L : \eta = 15.4 n_1 L / \bar{V}_{\bar{z}} = 15.4(0.355)(132.04)/\bar{V}_{\bar{z}} = 1.0057\ E-7$

$R_L = 0.8075$

$R_n = 3.428\ E-7$

$\beta = 0.5$

$R = \sqrt{\frac{1}{\beta} R_n R_h R_B (0.53 + 0.47 R_L)} = 0.00079$

$Q = \sqrt{\frac{1}{1 + 0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}} = \sqrt{\frac{1}{1 + 0.63\left(\frac{155.25 + 367.41}{929.15}\right)^{0.63}}} =$

$G_F = \boxed{0.855}$

North – South Results

Story	h_x (ft)	Floor Height	Peri - meter	K_z	q_z	N-S Wind ward	N-S Lee ward	N-S Side Wall	Load (kip)	Shear (kip)	Moment (kip-ft)
Spire	367.41	0.00	56.00	1.66	43.83	30.012	-17.44	-26.26	8.19	8.19	3007.32
34	361.25	6.16	56.00	1.66	43.67	29.905	-17.44	-26.26	24.41	32.59	8817.25
33	349.00	12.25	56.00	1.65	43.35	29.689	-17.44	-26.26	29.69	62.29	10363.11
32	338.75	10.25	88.50	1.64	43.08	29.503	-17.44	-26.26	48.82	111.11	16537.50
31	325.50	13.25	124.83	1.62	42.72	29.256	-17.44	-26.26	68.50	179.60	22296.06
30	315.25	10.25	124.83	1.61	42.44	29.060	-17.44	-26.26	59.50	239.11	18758.06
29	305.00	10.25	124.83	1.60	42.14	28.858	-17.44	-26.26	59.24	298.35	18069.52
28	294.75	10.25	124.83	1.59	41.84	28.651	-17.44	-26.26	58.98	357.33	17384.22
27	284.50	10.25	124.83	1.58	41.53	28.439	-17.44	-26.26	58.71	416.04	16702.26
26	274.25	10.25	124.83	1.57	41.21	28.220	-17.44	-26.26	58.43	474.46	16023.71
25	264.00	10.25	124.83	1.55	40.88	27.994	-17.44	-26.26	58.14	532.60	15348.70
24	253.75	10.25	132.08	1.54	40.54	27.762	-17.44	-26.26	61.20	593.80	15529.76
23	243.50	10.25	132.08	1.53	40.19	27.522	-17.44	-26.26	60.88	654.68	14823.35
22	233.25	10.25	132.08	1.51	39.83	27.274	-17.44	-26.26	60.54	715.22	14121.04
21	223.00	10.25	132.08	1.50	39.45	27.017	-17.44	-26.26	60.19	775.41	13422.96
20	212.75	10.25	132.08	1.48	39.06	26.751	-17.44	-26.26	59.83	835.25	12729.28
19	202.50	10.25	132.08	1.47	38.66	26.474	-17.44	-26.26	59.46	894.70	12040.16
18	192.25	10.25	132.08	1.45	38.24	26.186	-17.44	-26.26	59.07	953.77	11355.78
17	182.00	10.25	132.08	1.44	37.80	25.886	-17.44	-26.26	58.66	1012.43	10676.34
16	171.75	10.25	132.08	1.42	37.34	25.572	-17.44	-26.26	58.24	1070.67	10002.06
15	161.50	10.25	132.08	1.40	36.86	25.243	-17.44	-26.26	57.79	1128.46	9333.17
14	151.25	10.25	132.08	1.38	36.36	24.897	-17.44	-26.26	57.32	1185.78	8669.96
13	141.00	10.25	132.08	1.36	35.82	24.532	-17.44	-26.26	56.83	1242.61	8012.71
12	130.75	10.25	132.08	1.34	35.26	24.145	-17.44	-26.26	56.30	1298.91	7361.78
11	120.50	10.25	132.08	1.32	34.66	23.733	-17.44	-26.26	55.75	1354.66	6717.54
10	110.25	10.25	132.08	1.29	34.01	23.293	-17.44	-26.26	55.15	1409.81	6080.45
9	100.00	10.25	132.08	1.27	33.32	22.820	-17.44	-26.26	54.51	1464.32	5451.03
8	89.75	10.25	132.08	1.24	32.57	22.306	-17.44	-26.26	58.41	1522.73	5242.19
7	77.75	12.00	132.08	1.20	31.60	21.642	-17.44	-26.26	57.43	1580.16	4465.43
6	67.50	10.25	132.08	1.17	30.68	21.008	-17.44	-26.26	52.06	1632.22	3513.84
5	57.25	10.25	132.08	1.13	29.63	20.292	-17.44	-26.26	51.09	1683.31	2924.76
4	47.00	10.25	132.08	1.08	28.43	19.466	-17.44	-26.26	49.97	1733.28	2348.59
3	36.75	10.25	132.08	1.03	26.99	18.484	-17.44	-26.26	48.64	1781.92	1787.51
2	26.50	10.25	132.08	0.96	25.20	17.254	-17.44	-26.26	48.69	1830.61	1290.39
1	15.50	11.00	132.08	0.85	22.51	15.412	-17.44	-26.26	57.50	1888.11	891.25
SOG	0.00	15.50	132.08	0.00	0.00	0.000	0.00	0.00	0.00	1888.11	0.00
TOTAL	**367.4**								**1888.1**		**352099.1**

East – West Results

Story	h_x (ft)	Floor Height	Perimeter (ft)	K_z	q_z	E-W Windward	E-W Leeward	E-W Side Wall	Load (kip)	Shear (kip)	Moment (kip-ft)
Spire	367.41	0.00	56.00	1.66	43.83	29.977	-18.735	-26.230	8.40	8.40	3086.92
34	361.25	6.16	56.00	1.66	43.67	29.870	-18.735	-26.230	25.06	33.46	9051.17
33	349.00	12.25	56.00	1.65	43.35	29.654	-18.735	-26.230	30.49	63.94	10639.36
32	338.75	10.25	88.50	1.64	43.08	29.468	-18.735	-26.230	50.13	114.07	16980.16
31	325.50	13.25	111.67	1.62	42.72	29.222	-18.735	-26.230	62.93	176.99	20482.36
30	315.25	10.25	111.67	1.61	42.44	29.026	-18.735	-26.230	54.67	231.66	17234.15
29	305.00	10.25	141.67	1.60	42.14	28.824	-18.735	-26.230	69.06	300.72	21064.05
28	294.75	10.25	141.67	1.59	41.84	28.618	-18.735	-26.230	68.76	369.49	20267.69
27	284.50	10.25	141.67	1.58	41.53	28.405	-18.735	-26.230	68.45	437.94	19475.11
26	274.25	10.25	141.67	1.57	41.21	28.187	-18.735	-26.230	68.14	506.08	18686.41
25	264.00	10.25	141.67	1.55	40.88	27.962	-18.735	-26.230	67.81	573.89	17901.71
24	253.75	10.25	141.67	1.54	40.54	27.729	-18.735	-26.230	67.47	641.36	17121.12
23	243.50	10.25	155.25	1.53	40.19	27.490	-18.735	-26.230	73.56	714.92	17911.54
22	233.25	10.25	155.25	1.51	39.83	27.242	-18.735	-26.230	73.16	788.08	17065.60
21	223.00	10.25	155.25	1.50	39.45	26.986	-18.735	-26.230	72.76	860.84	16224.64
20	212.75	10.25	155.25	1.48	39.06	26.720	-18.735	-26.230	72.33	933.17	15388.83
19	202.50	10.25	155.25	1.47	38.66	26.443	-18.735	-26.230	71.89	1005.06	14558.38
18	192.25	10.25	155.25	1.45	38.24	26.156	-18.735	-26.230	71.44	1076.50	13733.49
17	182.00	10.25	155.25	1.44	37.80	25.856	-18.735	-26.230	70.96	1147.46	12914.40
16	171.75	10.25	155.25	1.42	37.34	25.542	-18.735	-26.230	70.46	1217.92	12101.36
15	161.50	10.25	155.25	1.40	36.86	25.213	-18.735	-26.230	69.94	1287.85	11294.67
14	151.25	10.25	155.25	1.38	36.36	24.868	-18.735	-26.230	69.39	1357.24	10494.63
13	141.00	10.25	155.25	1.36	35.82	24.503	-18.735	-26.230	68.81	1426.05	9701.60
12	130.75	10.25	155.25	1.34	35.26	24.117	-18.735	-26.230	68.19	1494.24	8915.97
11	120.50	10.25	155.25	1.32	34.66	23.706	-18.735	-26.230	67.54	1561.77	8138.21
10	110.25	10.25	155.25	1.29	34.01	23.266	-18.735	-26.230	66.84	1628.61	7368.85
9	100.00	10.25	155.25	1.27	33.32	22.793	-18.735	-26.230	66.08	1694.70	6608.48
8	89.75	10.25	155.25	1.24	32.57	22.280	-18.735	-26.230	70.84	1765.54	6357.89
7	77.75	12.00	155.25	1.20	31.60	21.617	-18.735	-26.230	69.69	1835.23	5418.76
6	67.50	10.25	155.25	1.17	30.68	20.983	-18.735	-26.230	63.20	1898.43	4266.29
5	57.25	10.25	155.25	1.13	29.63	20.268	-18.735	-26.230	62.07	1960.50	3553.30
4	47.00	10.25	155.25	1.08	28.43	19.443	-18.735	-26.230	60.75	2021.26	2855.45
3	36.75	10.25	155.25	1.03	26.99	18.462	-18.735	-26.230	59.19	2080.45	2175.33
2	26.50	10.25	155.25	0.96	25.20	17.234	-18.735	-26.230	59.33	2139.78	1572.31
1	15.50	11.00	155.25	0.85	22.51	15.394	-18.735	-26.230	70.21	2209.99	1088.19
SOG	0.00	15.50	155.25	0.00	0.00	0.000	0.000	0.000	0.00	2209.99	0.00
TOTAL	**367.41**								**2209.99**		**401698.4**

appendix c

seismic

Input	Value	Source
Occupancy Category	II	ASCE 7-05
Importance Factor	1.0	ASCE 7-05
Soil Site Class	D	Geotech Report
Seismic Design Category	B	ASCE 7-05
S_S	0.118	USGS.gov
S_1	0.048	USGS.gov
Fa	1.6	ASCE 7-05, Tbl 11.4-1
Fv	2.4	ASCE 7-05, Tbl 11.4-2
S_{DS}	0.126	ASCE 7-05
S_{D1}	0.077	ASCE 7-05
R	5	ASCE 7-05, Tbl 12.2-1
h_n	361.25	
C_t	0.02	ASCE 7-05, Tbl 12.8-2
x	0.75	ASCE 7-05, Tbl 12.8-2
T_a	1.66	
C_u	1.7	ASCE 7-05, Tbl 12.8-1
T	2.82	
T_L	8	ASCE 7-05, Fig 22-15
C_S	0.01	ASCE 7-05, Eq 12.8-5
k	2	ASCE 7-05, Sec 12.8.3
V_b	**845.1 k**	

SEISMIC CALCULATIONS | GRANBY TOWER | 1

http://earthquake.usgs.gov/research/hazmaps/design/

LATITUDE = 36°51'12" → 36.8533

LONGITUDE = -76°17'21" → -76.28917

SITE CLASS B → $S_S = 0.118g$
$S_1 = 0.048g$

TECH REPORT CLASSIFIED $\boxed{\text{SITE CLASS D}}$

$F_a = 1.6$ [TABLE 11.4-1 : SITE CLASS D, $S_S \le 0.25$]
$F_v = 2.4$ [TABLE 11.4-2 : SITE CLASS D, $S_1 \le 0.1$]

$S_{MS} = F_a S_S = (1.6)(0.118g) = 0.188$
$S_{M1} = F_v S_1 = (2.4)(0.048g) = 0.115$

$S_{DS} = \frac{2}{3} S_{MS} = 0.126g$
$S_{D1} = \frac{2}{3} S_{M1} = 0.077g$

OCCUPANCY CATEGORY = II
IMPORTANCE FACTOR, I = 1.0

SEISMIC DESIGN CATEGORY :
TABLE 11.6-1 : $S_{DS} < 0.167$ → A
MORE SEVERE TABLE 11.6-2 : $0.067 \le S_{D1} \le 0.133$ → B ← CONTROLS

DESIGN VALUE

SEISMIC FORCE RESISTING SYSTEM (TABLE 12.2-1) : ORDINARY REINFORCED SHEAR WALLS

DESIGN VALUE

→ R = 5

→ NO LIMITATIONS WITH SDC = B

SEISMIC CALCULATIONS | GRANBY TOWER | 2

EQUIVALENT LATERAL FORCE METHOD

$V = C_s W$

W = EFFECTIVE SEISMIC WEIGHT [TOTAL DEAD + OPERATING WT OF EQPT

$$C_s = \min \begin{cases} S_{DS}/(R/I) \\ S_{D1}/[T(R/I)] & (T \le T_L) \\ S_{D1}T_L/T^2(R/I) & (T > T_L) \end{cases}$$

$S_{DS} = 0.126g$
$S_{D1} = 0.077g$
$R = 5$ [ASCE 7-05, p 120]
$I = 1.0$ [ASCE 7-05, p 116]
$T_L = 8$ SEC [ASCE 7-05, p 229]

$T_a = C_t h_n^x$

$h_n = (370.97' - 9.72') = \underline{361.25'}$ [BASE TO LEVEL 34]
$C_t = 0.02$, $x = 0.75$ } TABLE 12.8-2, p129 : ALL OTHER STRUCT. SYSTEMS

$T_a = (0.02)(361.25')^{0.75} = \boxed{1.66 \text{ SEC}}$

$T = C_u T_a$

$C_u = 1.7$ [ASCE 7-05, p129, TABLE 12.8-1]

$T = (1.7)(1.66 \text{ SEC}) = \boxed{2.82}$

$$C_s = \min \begin{cases} 0.126/(5/1) = 0.0252 \\ 0.077/[2.82(5/1)] = 0.0055 & (T \le T_L) \end{cases}$$

$C_s \ge 0.01$ (ASCE 7-05, 12.8-5)

$\boxed{C_s = 0.01}$

$W = (81972.1k)\{DL\} + (87.1k)\{\text{MECH LOAD}\} + (2448.9k)\{CW\} = \boxed{84508 k}$

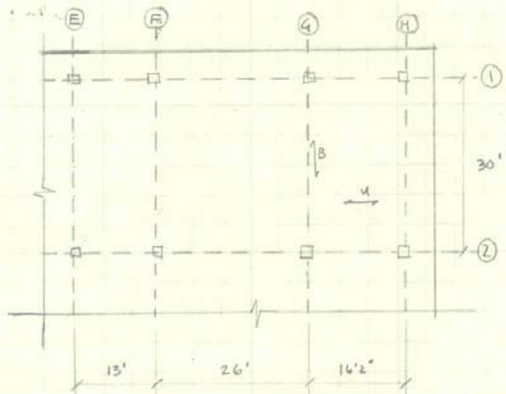
$V = (0.01)(84508k) = \boxed{845.1 k}$

Story	h_x (ft)	Floor Height	Floor Load W_x (kip)	$h_x^k W_x$	C_{vx}	$F_x = C_{vx}V$	V_x (kips)	Mx (ft.-k)
Spire (btm)	367.41	0.00	83.0	1.1204E+07	0.00360	3.04	3.04	1116.92
34	361.25	6.16	582.7	7.6045E+07	0.02441	20.63	23.67	7453.62
33	349.00	12.25	1088.4	1.3257E+08	0.04255	35.97	59.64	12553.25
32	338.75	10.25	1882.0	2.1596E+08	0.06932	58.59	118.24	19849.05
31	325.50	13.25	1563.3	1.6563E+08	0.05317	44.94	163.18	14628.06
30	315.25	10.25	2050.8	2.0381E+08	0.06542	55.30	218.48	17433.31
29	305.00	10.25	2111.2	1.9640E+08	0.06304	53.29	271.76	16252.73
28	294.75	10.25	2148.5	1.8666E+08	0.05992	50.65	322.41	14927.82
27	284.50	10.25	2166.2	1.7533E+08	0.05628	47.57	369.98	13534.33
26	274.25	10.25	2166.2	1.6293E+08	0.05230	44.21	414.19	12123.55
25	264.00	10.25	2197.0	1.5312E+08	0.04915	41.55	455.74	10968.15
24	253.75	10.25	2540.4	1.6357E+08	0.05251	44.38	500.12	11261.90
23	243.50	10.25	2518.0	1.4930E+08	0.04792	40.51	540.63	9863.79
22	233.25	10.25	2518.0	1.3699E+08	0.04397	37.17	577.80	8669.86
21	223.00	10.25	2518.0	1.2522E+08	0.04019	33.97	611.77	7576.38
20	212.75	10.25	2518.0	1.1397E+08	0.03658	30.92	642.69	6578.94
19	202.50	10.25	2518.0	1.0325E+08	0.03314	28.02	670.71	5673.12
18	192.25	10.25	2518.0	9.3065E+07	0.02987	25.25	695.96	4854.52
17	182.00	10.25	2518.0	8.3406E+07	0.02677	22.63	718.59	4118.71
16	171.75	10.25	2518.0	7.4276E+07	0.02384	20.15	738.74	3461.28
15	161.50	10.25	2518.0	6.5675E+07	0.02108	17.82	756.56	2877.83
14	151.25	10.25	2518.0	5.7603E+07	0.01849	15.63	772.19	2363.92
13	141.00	10.25	2527.2	5.0243E+07	0.01613	13.63	785.82	1922.15
12	130.75	10.25	2527.2	4.3204E+07	0.01387	11.72	797.55	1532.70
11	120.50	10.25	2691.0	3.9074E+07	0.01254	10.60	808.15	1277.52
10	110.25	10.25	2691.0	3.2709E+07	0.01050	8.87	817.02	978.46
9	100.00	10.25	2691.0	2.6910E+07	0.00864	7.30	824.32	730.14
8	89.75	10.25	2691.0	2.1676E+07	0.00696	5.88	830.21	527.85
7	77.75	12.00	2829.1	1.7102E+07	0.00549	4.64	834.85	360.78
6	67.50	10.25	3118.7	1.4210E+07	0.00456	3.86	838.70	260.24
5	57.25	10.25	3118.7	1.0222E+07	0.00328	2.77	841.47	158.78
4	47.00	10.25	3118.7	6.8892E+06	0.00221	1.87	843.34	87.85
3	36.75	10.25	3118.7	4.2120E+06	0.00135	1.14	844.49	42.00
2	26.50	10.25	3118.7	2.1901E+06	0.00070	0.59	845.08	15.75
1	15.50	11.00	3050.8	7.3295E+05	0.00024	0.20	845.28	3.08
SOG	0.00	15.50	1476.5	0.0000E+00	0.00000	0.00	845.28	0.00
TOTAL	**367.41**		**84528.0**	**3.1154E+09**	**1.00000**	**845.28**		**216038.3**

appendix d

spot checks

SPOT CHECK | GRANBY TOWER | POST TENSION / FRAME ANALYSIS | 1/15



BANDED TENDONS ALONG E,F,G,H

UNIFORM TENDONS IN 1,2... DIRECTION

ASSUME:

COLUMNS: 30 x 30 IN
STORY HEIGHT: 10'3"

f'_c = 5000 psi
w = 150 PCF
f_y = 60000 psi
f_{pu} = 270000 psi

LL = 40 PSF (RESIDENTAL FLOORS)
SIDL = 20 PSF

ANALYZE LEVEL 8, POST-TENSIONED FRAME (TYP.) [POST-TENSIONED MANUAL, 6th ED.]

→ SLAB

SLAB/DEPTH RATIO: $\frac{L}{45}$ [2-WAY SLAB, COL. ONLY, CONTINUOUS FLOOR SPAN]

LONGITUDINAL = $\frac{30(12)}{45}$ = 8 IN ← USE 8 IN SLAB [USED IN DESIGN]

TRANSVERSE = $\frac{26(12)}{45}$ = 6.9 IN

→ LOAD BALANCING

DEAD LOAD (SLAB): $\frac{8}{12}$(150 PCF) = 100 PSF

LIVE LOAD: 26x30 BAY → $L = 40\,PSF\left(0.25 + 15/\sqrt{(1)(780\,SF)}\right) = 40\,PSF(0.79) = 31\,PSF$

16x30 BAY → $L = 40\,PSF\left(0.25 + 15/\sqrt{(1)(484\,SF)}\right) = 40\,PSF(0.93) = 37\,PSF$

TOTAL FACTORED LOAD: 1.2(120 PSF) + 1.6(37 PSF) = $\boxed{203.2\ PSF}$

TENDONS TAKE 80% SLAB WEIGHT

w_{BAL} = 0.8(100 PSF) = 80 PSF

a = 8 IN − 1.25 − 1.25 = 5.5 IN (MAX TENDON SAG) [VALUES PER DESIGN]

$F_e = w_{BAL}L^2/8a = (80\,PSF)(30')^2/8\left(\frac{5.5}{12}\right) = 19.6\ k/FT$

SPOT CHECK | GRANBY TOWER | POST TENSION / FRAME ANALYSIS | 2/15

→ ASSUME 14 KSI LONG TERM LOSSES AND 1/2" ϕ, 7-WIRE STRAND

$0.153\left((0.7)(270\text{ ksi}) - 14\text{ ksi}\right) = 26.8\text{ k / TENDON}$

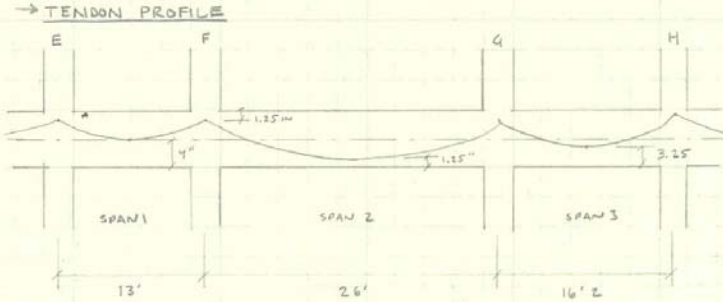
$\#\text{TENDONS} = (30')(19.6\text{k/Ft}) / (26.8\text{k/TENDON}) = 21.9\text{ TENDONS}$

USE 22 TENDONS

$F_e = 22(26.8\text{ k/TEN})/30' = 19.65\text{ k/FT}$

$F/A = 19.65/(8 \times 12) = 0.205\text{ ksi}$

→ TENDON PROFILE



ADJUST TENDON PROFILE

$W_{BAL} = \frac{8 F_e a}{L^2} = 8(19.65)(5.5/12)/(26')^2 = 0.107\text{ ksf}$

• $a_1 = \frac{W_{BAL} L^2}{8 F_e} = 0.107(13')^2/8(19.65) = 1.38\text{ in}$

MIDSPAN CGS $= \frac{1}{2}(4 + 6.75) - 1.38\text{ in} = \underline{4\text{ in}}$

ACTUAL SAG, SPAN 1 $= \frac{1}{2}(4 + 6.75) - 4\text{ in} = 1.375\text{ in}$

ACTUAL BALANCED LOAD: $W_{BAL} = 8(19.65)(1.375/12)/(13')^2 = \underline{0.1066\text{ ksf}}$

• $a_3 = \frac{W_{BAL} L^2}{8 F_e} = 0.107(16'2")^2/8(19.65) = 2.13\text{ in}$

MIDSPAN CGS $= \frac{1}{2}(4 + 6.75) - 2.13\text{ in} = \underline{3.25\text{ in}}$

ACTUAL SAG, SPAN 3 $= \frac{1}{2}(4 + 6.75) - 3.25\text{ in} = 2.125\text{ in}$

ACTUAL BALANCED LOAD: $W_{BAL} = 8(19.65)(2.125/12)/(16'2")^2 = \underline{0.1065\text{ ksf}}$

SPOT CHECK | GRANBY TOWER | POST TENSION / FRAME ANALYSIS | 3/15

NET LOAD CAUSING BENDING :

SPAN 1 : $W_{NET} = 0.160 - 0.1066 = 0.0534$ KSF

SPAN 2 : $W_{NET} = 0.151 - 0.107 = 0.044$ KSF

SPAN 3 : $W_{NET} = 0.157 - 0.1065 = 0.0505$ KSF

→ EQUIVALENT FRAME

* ASSUME ALL COLUMNS 30 x 30 IN FOR SIMPLICITY , H = 10.25'

COLUMN STIFFNESS :

$I = \frac{1}{12}(30)(30^3) = 67500 \text{ in}^4$

$E = E_{col} / E_{SLAB} = 1.0$

$k_c = \frac{4EI}{L-2h} = 4(67500)(1.0)/(10.25(12) - 2(8\text{in})) = 2523.4 \text{ in}^3$

TORSIONAL STIFFNESS :

$C = \left[1 - 0.63\frac{x}{y}\right]\frac{x^3y}{3} = \left[1 - 0.63\left(\frac{8}{30}\right)\right]\left(\frac{(8)^3(30)}{3}\right) = 4260 \text{ in}^4$

$K_t = \frac{9CE}{L_2(1 - c_2/L_2)^3} = (9(4260)(1.0))/((30 \times 12)(1 - 1.33/30')^3) = 122 \text{ in}^3$

$K_{EC} = \left[\frac{1}{\Sigma k_c} + \frac{1}{\Sigma K_t}\right]^{-1} = \left[\frac{1}{2(2523.4)} + \frac{1}{2(122)}\right]^{-1} = 232.7 \text{ in}^3$

SLAB STIFFNESS :

$K_{S1} = [4(1.0)(30')(8")^3] / [12(13') - (30/2)] = 435.8 \text{ in}^3$

$K_{S2} = [4(1.0)(30')(8")^3] / [12(26') - (30/2)] = 206.9 \text{ in}^3$

$K_{S3} = [4(1.0)(30')(8)^3] / [12(16'2") - (30/2)] = 343.2 \text{ in}^3$

FIXED END MOMENTS

SPAN 1 : $0.0534(13)^2/12 = 0.752$ ft k

SPAN 2 : $0.044(26)^2/12 = 2.479$ ft k

SPAN 3 : $0.0505(16'2")^2/12 = 1.10$ ft k

SPOT CHECK | GRANBY TOWER | POST TENSION / FRAME ANALYSIS | 4/15

DISTRIBUTION FACTORS

SPAN 1, EXT : 435.8 / (435.8 + 232.7) = 0.65

SPAN 3, EXT : 343.2 / (343.2 + 232.7) = 0.60

SPAN 1, INT : 435.8 / (435.8 + 232.7 + 206.9) = 0.50

SPAN 2, COL F : 206.9 / (435.8 + 232.7 + 206.9) = 0.24

SPAN 2, COL G : 206.9 / (206.9 + 232.7 + 343.2) = 0.26

SPAN 3, INT : 343.2 / (206.9 + 232.7 + 343.2) = 0.44

	E		F		G		H
DF	0.65	0.50	0.24	0.26	0.44	0.60	
FEM	-0.752	0.752	-2.479	2.479	-1.1	1.1	
DIST	0.489	0.864	0.414	-0.359	-0.607	-0.66	
CO	0.432	0.245	-0.180	0.207	-0.330	-0.304	
DIST	-0.281	-0.033	-0.016	0.032	0.054	0.182	
	-0.112	1.828	-2.261	2.359	-1.983	0.318	

→ NET TENSILE STRESSES

$-M_{MAX}$ @ FACE OF COL F

$$-M_{MAX} = -2.261 + \frac{1}{3}\left(\frac{0.044\,(26)}{2}\right)\left(\frac{30}{12}\right) = -1.784 \text{ ft k}$$

$$S = bh^2/6 = 12\,(8)^2/6 = 128 \text{ in}^3$$

$$f_{t,b} = -f_{pL} \pm \frac{M_{NET}}{S_{t,b}} = -0.205 \pm \frac{12\,(1.784)}{128}$$

$$= -0.038\ ,\ -0.372 \text{ ksi} \quad (\text{NO TENSION})$$

ALLOWABLE COMPRESSION

$0.6\,f'_c$ (AT TRANSFER, $f'_{ci} = 0.75 f'_c$) $= 0.6\,(0.75)(5 \text{ ksi}) = 2.25 \text{ ksi} > f_{t,b}$ OK ✓

$0.45\,f'_c$ (AT SERVICE LOAD) $= 0.45\,(5 \text{ ksi}) = 2.25 \text{ ksi} > f_{t,b}$ OK ✓

SPOT CHECK | GRANBY TOWER | POST TENSION / FRAME ANALYSIS | 5/15

+M MAX @ MIDSPAN OF SPAN 2

$$+M_{max} = \left(\frac{(0.044)(26)^2}{8}\right) - 2.479 = 1.239 \text{ ft k}$$

$$f_{t,b} = -f_{pc} \pm \frac{M_{net}}{S} = -0.205 \pm \frac{12(1.239)}{128}$$

$$= -0.089, -0.32 \text{ ksi} \quad (\text{NO TENSION})$$

ALLOWABLE COMPRESSION : $2.25 \text{ ksi} > f_{t,b}$ OK ✓

→ FLEXURAL CAPACITY

$$FEM_1 = (0.1066)(13)^2/12 = 1.5 \text{ ft k}$$

$$FEM_2 = (0.107)(26)^2/12 = 6.03 \text{ ft k}$$

$$FEM_3 = (0.1065)(16'2'')^2/12 = 2.32 \text{ ft k}$$

	E		F		G		H
DF	0.65	0.50	0.24	0.26	0.44	0.60	
FEM	-1.5	1.5	-6.03	6.03	-2.32	2.32	
DIST	0.975	2.265	1.087	-0.965	-1.632	-1.392	
CO	1.133	0.488	-0.482	0.544	-0.696	-0.816	
DIST	-0.736	-0.003	-0.001	0.040	0.067	0.49	
	-0.128	4.25	-5.426	5.649	-4.581	0.602	

→ SECONDARY MOMENTS

$$M_{1,EXT} = 0.128 - 19.6(4-1.25)/12 = -4.36 \text{ ft k}$$

$$M_{3,EXT} = 0.602 - 19.6(2.75)/12 = -3.8 \text{ ft k}$$

$$M_{1,INT} = 4.25 - 19.6(2.75)/12 = -0.24 \text{ ft k}$$

$$M_{2,F} = 5.426 - 4.492 = 0.93 \text{ ft k}$$

$$M_{2,G} = 5.649 - 4.492 = 1.16 \text{ ft k}$$

$$M_{3,INT} = 4.581 - 4.492 = 0.09 \text{ ft k}$$

SPOT CHECK | GRANBY TOWER | POST TENSION / FRAME ANALYSIS | 6/15

→ FACTORED LOAD MOMENTS

$FEM_1 = 0.208\,(13)^2/12 = 2.93$ ft k

$FEM_2 = 0.194\,(26)^2/12 = 10.93$ ft k

$FEM_3 = 0.203\,(16'2'')^2/12 = 4.42$ ft k

	E	F		G		H
DF	0.65	0.50	0.24	0.26	0.44	0.60
FEM	-2.93	2.93	-10.93	10.93	-4.42	4.42
DIST	1.905	4.00	1.92	-1.693	-2.86	-2.652
CO	-2.00	0.953	-0.846	0.96	-1.326	-1.432
DIST	-1.3	-0.054	-0.026	0.095	0.161	0.859
SUM	-0.325	7.829	-9.882	10.292	-8.439	1.195
2ND MOM	-4.36	-0.24	0.93	-0.93	0.09	-3.8
@ COL	-4.685	7.589	-8.952	9.362	-8.349	-2.605

→ DESIGN MOMENTS @ MIDSPAN

SPAN 1:

$$V_{EXT} = \left[\frac{0.208(13)}{2} - \frac{7.589-4.685}{13}\right] = 1.13 \text{ k/ft}$$

$V_{INT} = 1.43$ k/ft

POINT OF ZERO SHEAR AND MAX MOMENT:

$x = 1.13/(0.208) = 5.43$ FT FROM COL E ℄

POSITIVE MOMENT

$M_{MAX} = 0.5(1.13)(5.43) - 4.685 = -1.62$ ft k/ft

SPAN 2:

$$V = \left[\frac{0.208(26)}{2} - \frac{9.362-8.952}{26}\right] = 2.69 \text{ k/ft}$$

$x = 2.69/(0.208) = 12.92$ FT

$M_{MAX} = 0.5(2.69)(12.92 \text{ ft}) - 9.362 = 8.02$ ft k/ft

SPAN 3: $V = \left[\frac{0.208(16.16)}{2} - \left(\frac{8.349-2.605}{16.16}\right)\right] = 1.33$ k/ft ← V_{EXT}

$x = 1.33/(0.208) = 6.39$ ft

$M_{MAX} = 0.5(1.33)(6.39 \text{ ft}) - 2.605 = 1.64$ ft k/ft

SPOT CHECK | GRANBY TOWER | POST TENSION / FRAME ANALYSIS | 7/15

→ FLEXURAL STRENGTH

CAPACITY CHECK AT INT. SUPPORT

$A_S = 0.00075\, A_{cf} = 0.00075\,(30')(12)(8") = 2.16\ in^2$

→ TRY (7) #5 @ 6IN O.C.

$\text{BAR LENGTH} = 2(26 - 30/12)/6 + 30/12 = 10'4"$

$A_S = \frac{7(0.31)}{30'} = 0.072\ in^2/ft$

→ CALC DESIGN STRESS IN TENDON

$f_{ps} = f_{pe} + 10000 + \frac{f'c}{300\rho_p}$

$\rho_p = \frac{A_{ps}}{bd} = 0.153(22\ \text{TENDONS}) / (30)(12)(8 - 1.25in) = 0.00139$

$f_{pe} = (0.7(270) - 14) = 175\ ksi$

$f_{ps} = 175 + 10 + \frac{5}{300(0.00139)} = 197\ ksi$

$f_{ps} < 0.85 f_{pu} = 0.85(270) = 230$ ok
$f_{ps} < f_{se} + 30 = 175 + 30 = 205$ ok

$F_{su} = 197(0.153)(22)/30' = 22.1\ k/ft$ (TENDONS)

$F_u = 60\ ksi\,(0.072\ in^2/ft) = 4.32\ k/ft$ (REBAR)

$F_{TOT} = 26.42\ k/FT$

→ DEPTH OF COMPRESSION BLOCK

$a = \frac{F}{0.85\, b\, f'c} = 26.42/((0.85)(12)(5ksi)) = 0.52\ in$

$\varepsilon_t = (6.75 - 0.48)(0.003)/(0.52/0.85) = 0.0307$

$d - a/2 = (6.75 - 0.52/2)/12 = 0.54\ ft$ * ASSUME REBAR + TENDONS IN SAME PLANE

SPOT CHECK | GRANBY TOWER | POST TENSION / FRAME ANALYSIS | 8/15

→ MOMENT CAPACITY @ COLUMN ℄

$\phi M_n = 0.9(0.54)(26.42) = 12.84$ ft k/ft > 9.32 ft k/ft ok ✓

PERMISSIBLE CHANGE IN NEGATIVE MOMENT

$1000\, \varepsilon_t = 1000(0.0307) = 30.7\% > 20\%$ MAX

AVAILABLE INCREASE: $0.2(9.32) = 1.86$ ft k/ft

ACTUAL INCREASE: $12.84 - 9.32 = 3.52 \nleq 1.86$ ft k/ft NO GOOD

→ MOMENT CAPACITY @ MIDSPAN OF SPAN 2

$8.02 - 1.86$ ft k/ft $= 6.16$ ft k/ft

$A_{ps} f_{ps} = 22.1$ k/ft

$a = 22.1/(0.85)(12)(5) = 0.433$ in

$\phi M_n = 0.9(22.1)(\frac{1}{12})(5.5 - 0.433/2) = 8.76$ ft k/ft > 6.16 ft k/ft ok ✓

→ MOMENT CAPACITY @ MIDSPAN OF SPAN 1

$\phi M_n = 0.9(22.1)(\frac{1}{12})(2.75 - 0.433/2) = 4.2$ ft k/ft > 1.62 ft k/ft ok ✓

→ MOMENT CAPACITY @ MIDSPAN OF SPAN 3

$\phi M_n = 0.9(22.1)(\frac{1}{12})(3.25 - 0.433/2) = 5.03$ ft k/ft > 1.64 ft k/ft ok ✓

→ EXTERIOR COLUMNS [SIMILAR TO INTERIOR SINCE TENDONS ANCHORED AT SLAB EDGE]

$A_{s\,min} = 0.00075(30')(12)(8\text{ in}) = 2.16\text{ in}^2$ → TRY (7) #5 @ 6" o.c., $A_s = 2.17\text{ in}^2$

$A_s = 7(0.31)/(30') = 0.072\text{ in}^2/\text{ft}$

$\rho_p = A_{ps}/bd = 22(0.153)/(30)(12)(6.75) = 0.00139$

$f_{ps} = 175 + 10 + \frac{5}{300(0.00139)} = 197$ ksi [CHECK SAME AS INT. COLUMN ok ✓]

$A_{ps} f_{ps} = 22(0.153)(197)/30 = 22.1$ k/ft

$a = (22.1 + 60(0.072))/(0.85)(12)(5) = 0.52$ in

SPOT CHECK | GRANBY TOWER | POST TENSION / FRAME ANALYSIS | 9/15

TENDONS: $d - a/2 = 6.75 - 0.52/2 = 6.49\ in/12 = 0.54\ ft$

REBAR: SAME PLANE AS TENDONS SINCE TENDONS ANCHORED AT ℄ OF SLAB DEPTH AT EDGE OF CANTILEVER SLAB

$\phi M_n = 0.9(22.1 + 4.32)(0.54) = 12.84\ ft\,k/ft > 3.8\ ft\,k/ft$ ok ✓

→ SHEAR CAPACITY @ EXTERIOR COLUMN

$V_{uH} = 1.33\ k/ft\,(30\ ft) = 40\ k$

$M_{TRANS} = 2.605\ ft\,k/ft\,(30ft) = 78.15\ ft\,k$

COMBINED SHEAR STRESS @ INSIDE FACE:

$d = 0.8(8in) = 6.4\ in$

$c_1 = 30in$ $\qquad b_1 = 30in + 3.2 = 33.2\ in$

$c_2 = 30in$ $\qquad b_2 = 30in + 6.4 = 36.4\ in$

$A_c = ((2)(33.2) + 36.4))(6.4) = 657.9\ in^2$

$J_c/c = \left[2(33.2)(6.4)(33.2 + 2(36.4)) + (6.4)^3\left(2(33.2) + 36.4\right)/33.2\right] / (6.4in) = 7165\ in^3$

$\gamma_v = 1 - \left(\frac{1}{1 + 2/3\sqrt{33.2/36.4}}\right) = 0.39$

$V_u = 40000/657.9in^2 + 0.39(12.84)(1000)(12)/7165 = 69.18\ psi$

$V_c = 4\sqrt{5000} = 283\ psi \rightarrow \phi V_n = 0.75(283) = 212.25\ psi$

$\phi V_n = 212.25\ psi > V_u = 69.18\ psi$ ok ✓

→ SHEAR CAPACITY @ INTERIOR COLUMN

$V_u = (1.33 + 2.69)(30') = 120.6\ K$

$M_{TRANS} = (30')(9.362 - 8.349\ ft\,k/ft) = 30.4\ ft\,k$

$V_u = 120600/657.9 + 0.39(30.4)(1000)(12)/7165 = 203.2\ psi$

$\phi V_n = 212.25\ psi > V_u = 203.2\ psi$ ok ✓

→ $\boxed{\text{SHEAR AND FLEXURE CAPACITY ADEQUATE}}$

USE 8" FLAT PLATE SLAB W/ 22 TENDONS UNIFORMLY DISTRIBUTED IN NORTH-SOUTH DIRECTION AND BANDED OVER THE COLUMN LINES IN THE EAST-WEST DIRECTION. USE (7) #5 @ 6" o.c. IN BOTH DIRECTIONS AROUND COLUMN IN SAME PLANE AS TENDONS.

→ ALL CRITERIA CHECKS WITH DESIGN AND COLUMN ASSUMPTION MORE CONSERVATIVE.

SPOT CHECK | GRANBY TOWER | COLUMN G2, LEVEL 8 | 10/15

COLUMN DESIGN (COLUMN G2, LEVEL 8)

LOADS:

DEAD: SLAB = 100 PSF

SIDL = 20 PSF

$$\text{COLUMNS} = (150\ \text{PCF})(10.25)\left[(36\times36)(4) + (30\times30)(12) + (24\times24)(5) + (18\times18)(3)\right]\left(\tfrac{1}{144}\right) = 211.8\ \text{k}$$

LIVE: RESIDENTIAL = 40 PSF

$$LL = 40\ \text{PSF}\left(0.25 + 15/\sqrt{(4)(590)(24\ \text{FLOORS})}\right) = 40\ \text{PSF}\,(0.313)$$

≥ 0.4

LL = 16 PSF

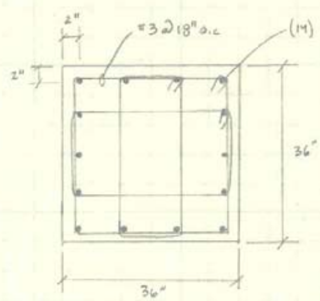
$$P_u = 1.2\left(211.8\text{k} + 120\ \text{PSF}(590)(24)\right) + 1.6\left((16\ \text{PSF})(590)(24)\right) = 2655.7\ \text{k}$$

$M_u = 30.4$ ft k (FROM PAGE 9)

ASSUME:

$f'_c = 5000$ psi, $f_y = 60000$ PSI, NWC

COL 36x36 w/ (14) #9 AND #3 TIES @ 18 IN o.c.



(14) #9, $A_s = 14\,(1.0\ \text{in}^2) = 14\ \text{in}^2$

PURE AXIAL: $P_o = 0.85 f'_c A_c + A_s F_y$

$$P_o = 0.85\,(5\ \text{ksi})(36^2 - 14\ \text{in}^2) + 14\ \text{in}^2\,(60\ \text{ksi}) = \underline{6288.5\ \text{k}} \rightarrow \phi = 0.65,\ \phi P_o = 4087.5\text{k}$$

BALANCED STRAIN CONDITION

$$c = \frac{0.003}{0.003 + \varepsilon_y}(d_2) = 0.003(34\ \text{in})/(0.003 + 60/29000) = 20.1\ \text{in}$$

$$a = \beta_1 c = 0.85\,(20.1\ \text{in}) = 17.1\ \text{in}$$

$$\varepsilon_{s_1} = \frac{0.003}{c}(c - d_1) = 0.003\,(20.1 - 2\ \text{in})/20.1\ \text{in} = 0.0027 > \varepsilon_y = 0.00207$$

$f_{s_1} = 60$ ksi

SPOT CHECK | GRANBY TOWER | COLUMN G2, LEVEL 8 | 11/15

$\varepsilon_{s2} = 0.003(20.1-10)/(20.1) = 0.0015 < \varepsilon_y$

$f_{s2} = 0.0015(29000) = 43.72$ ksi

$\varepsilon_{s3} = 0.003(20.1-18)/(20.1) = 0.00031 < \varepsilon_y$

$f_{s3} = 0.00031(29000) = 9.1$ ksi

$\varepsilon_{s4} = 0.003(20.1-26)/(20.1) = -0.00088 < \varepsilon_y$

$f_{s4} = -0.00088(29000) = -25.54$ ksi

$\varepsilon_{s5} = 0.003(20.1-34)/(20.1) = -0.002075 > \varepsilon_y$

$f_{s5} = -60$ ksi

$P_b = 0.85(5)(36)(20.1) + 4(60\text{ ksi}) + 2(43.72) + 2(9.1) + 2(-25.54) + 4(-60)$

$= \underline{3129.86\text{ k}} \rightarrow \phi P_b = 0.65(3129.86) = 2034.4$ k

$M_b = 0.85(5)(36)(0.85)(20.1)\left(18 - \frac{1}{2}(0.85)(20.1)\right) + 4(60)(18-2) + 2(43.72)(18-10)$
$+ 2(-25.54)(18-26) + 4(-60)(18-34)$

$= \underline{2796.5\text{ ft k}} \rightarrow \phi M_b = 0.65(2796.5) = 1817.7$ ft k

PURE BENDING

$\Sigma F = 0 = 0.85(5)(36)(0.85)c + 4\left(\frac{0.003}{c}\right)(c-2)(29000) + 2\left(\frac{0.003}{c}\right)(c-10)29000 - 8(60)$

$c = 3.48$ in

$f_{s1} = (0.003/3.48)(3.48-2)(29000) = 37$ ksi (c)

$\varepsilon_{s2} = (0.003/3.48)(3.48-10) = -0.0056 < \varepsilon_y = -0.00207$ OK

$M_o = 0.85(5)(36)(0.85)(3.48)\left(18 - \frac{1}{2}(0.85)(3.48)\right) + 4(37)(18-2) + 2(-60)(18-10)$
$+ 2(-60)(18-26) + 4(-60)(18-34)$

$= \underline{1140.4\text{ ft k}} \rightarrow \phi M_o = 0.9(1140.4) = 1026.4$ ft k

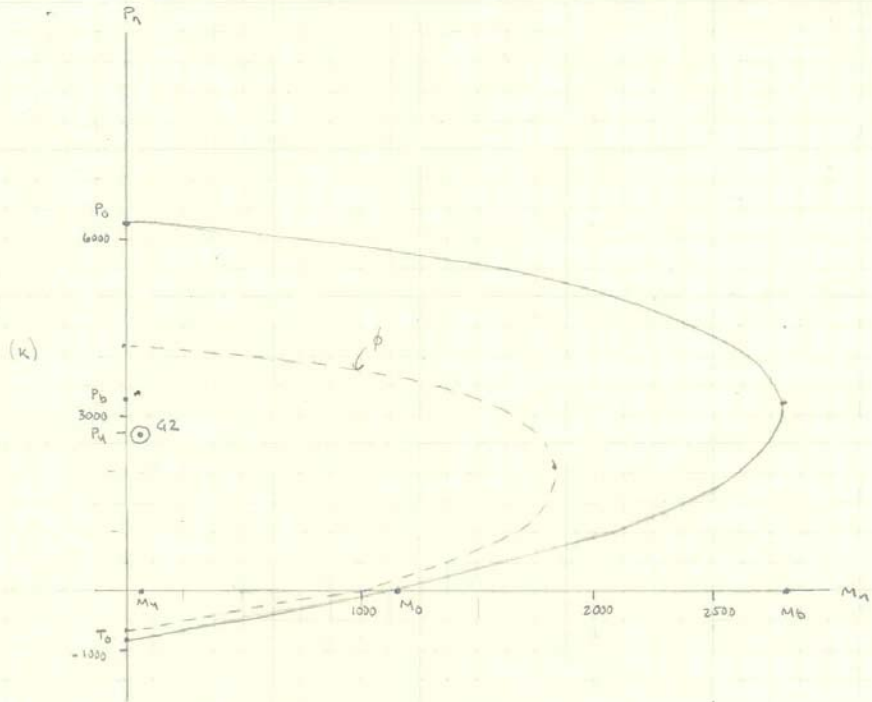
PURE TENSION

$T_o = \Sigma A_{si} f_{si} = 14\text{ in}^2(-60) = \underline{-840\text{ k}}$

$\phi T_o = 0.9(-840\text{ k}) = -756$ k

SPOT CHECK | GRANBY TOWER | COLUMN G2, LEVEL 8 | 12/15

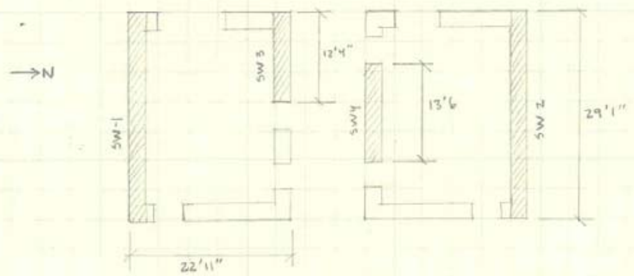
COLUMN INTERACTION DIAGRAM



COLUMN G2 IS ADEQUATE BASED ON INTERACTION DIAGRAM

SPOT CHECK | GRANBY TOWER | SHEAR WALL | 13/15

ANALYZE MAIN WALLS IN E-W DIRECTION (DIRECTION OF SHEAR + MOMENT)



$f'_c = 8000$ PSI
$f_y = 60000$ PSI

$w = 24$ IN
$h = 15.5$ ft (LEVEL 1)
VERT REINF = #10 @ 6"
HOR REINF = #6 @ 6"

VALUES FROM E-W WIND PRESSURES

$V_u = 2232.8$ k
$M_u = 401742.7$ ft k

$P_{u\,SW1,SW2} = (0.15\text{ kcf})\left[(2)(29)(67.5) + \frac{14}{12}(29)(300')\right] = 2110$ k

$P_{u\,SW3} = (0.15\text{ kcf})\left[2(12.3)(67.5) + \frac{14}{12}(12.3)(270')\right] = 830$ k

$P_{u\,SW4} = (0.15\text{ kcf})\left[2(13.5)(67.5) + \frac{14}{12}(13.5)(280')\right] = 935$ k

RIGIDITY $= 1/\Delta$

$\Delta = \Delta_F + \Delta_S = PH^3/3E_m I + 1.2PH/E_v A$

$E_m = 5700\sqrt{f'_c} = 5700\sqrt{8000} = 509823$

$E_v = 0.4\,E_m = 0.4(509823) = 203929$

$R_{SW1,SW2} = \left[2110k(367')^3/3(509823)(\frac{1}{12})(24)(29\times 12)^3\right] + \left[1.2(2110k)(367')/(203929)(24)(29)(12)\right]$

$R_{1,2} = 1/0.00136 = 738.2$

$R_{SW3} = \left[830k(337')^3/3(509823)(\frac{1}{12})(24)(12.3\times 12)^3\right] + \left[1.2(830k)(337')/(203929)(24)(12.3)(12)\right]$

$R_3 = 1/0.00369 = 270.7$

$R_{SW4} = \left[935k(347)^3/3(509823)(\frac{1}{12})(24)(13.5\times 12)^3\right] + \left[1.2(935)(347)/(203929)(24)(13.5)(12)\right]$

$R_4 = 1/0.0035 = 286.1$

SPOT CHECK | GRANBY TOWER | SHEAR WALL | 14/15

DIRECT SHEAR

$SW1 = [738.2/(738.2+738.2+270.7+286.1)] = 0.363$

$V_{u,sw_1} = 0.363(2232.8k) = 810.5k$
$M_{u,sw_1} = 0.363(401742.7\ ft\,k) = 145833\ ft\,k$] SW2 VALUES

$SW3 = 270.7/(738.2+738.2+270.7+286.1) = 0.133$

$V_{u\,sw_3} = 0.133(2232.8k) = 297.3k$
$M_{u\,sw_3} = 0.133(401742.7\ ft\,k) = 53431.8\ ft\,k$

$SW4 = 286.1/(738.2+738.2+270.7+286.1) = 0.141$

$V_{u\,sw4} = 0.141(2232.8k) = 314.8k$
$M_{u\,sw4} = 0.141(4017427\ ft\,k) = 56645.7$

SW1 + SW2

→ CHECK NEED FOR BOUNDARY ELEMENT

$C = P_u/2 + M_u/(h_w - b/2) = (2110/2) + (145833/(26-2)) = 7131.8\ k$

$A_g = 2ft(26ft) = 52\ ft^2$

$I_g = \frac{1}{12}(2ft)(26ft)^3 = 2929\ ft^4$

$f_c = P_u/A_g + M_u(h_w/2)/I_g = (7131.8/52ft^2) + ((145833\ ft\,k)(13ft)/2929\ ft^4) = 647.3\ ksf$

$f_c = 647.3\ ksf\left(\frac{1}{144}\right) = 4.5\ ksi$

$0.2(f'_c) = 0.2(8ksi) = 1.6\ ksi \rightarrow f_c > 0.2f'_c \therefore$ $\boxed{\text{BOUNDARY ELEMENT NEEDED}}$

→ LONGITUDINAL AND TRANSVERSE REINFORCEMENT

$V_n \geq 2A_{cv}\sqrt{f'_c}$

$V_n = 810.5k \geq 2(26)(2)(144)\sqrt{8000} = 1339.5\ k \therefore$ $\boxed{\text{1 CURTAIN ADEQUATE}}$

REQUIRED $\rho_l = 0.0015$, $\rho_h = 0.0025$

$A_{s_v} = 0.0015(24)(12) = 0.43\ in^2/FT \rightarrow$ TRY #7 @ 12" o.c. $(A_s = 0.6\ in^2/FT)$

$A_{s_H} = 0.0025(24)(12) = 0.72\ in^2/FT \rightarrow$ TRY #6 @ 6" o.c. $(A_s = 0.88\ in^2/FT)$

$\boxed{\text{TRY 1 CURTAIN OF \#7 VERTICAL @ 12" o.c. AND \#6 HORIZONTAL @ 6" o.c.}}$

SPOT CHECK	GRANBY TOWER	SHEAR WALL	15/15

→ NOMINAL SHEAR CAPACITY

$V_n = A_{cv}(\alpha_c \sqrt{f'c} + \rho f_y)$

$\alpha_{cv} = h_w/\ell_w = 367'/26' = 14.12 > 2$; $\alpha_c = 2$

$A_{cv} = (24)(26)(12) = 7488 \text{ in}^2$

$V_n = 7488(2\sqrt{8000} + (0.6/(24)(12))(60000)) = 2275.5 \text{ k}$

$\phi V_n = 0.6(2275.5\text{k}) = 1365.3 \text{ k} > V_u = 810.5 \text{ k}$ ok ✓

→ BOUNDARY ELEMENT CAPACITY

ASSUME (10) #6 WITHIN 2' x 2' SECTION

$A_s = 10(0.44) = 4.4 \text{ in}^2$

$\rho_s = 4.4 \text{ in}^2/(24)(24) = 0.0076 < \rho_{min} = 0.01 \rightarrow$ TRY #10

PROVIDED IN DESIGN

$A_s = 10(1.27) = 12.7 \text{ in}^2$

$\rho_s = 12.7 \text{ in}^2/(24)(24) = 0.022$

$\rho_{min} = 0.01 \leq \rho_s = 0.022 \leq \rho_{max} = 0.06$ ok ✓

∴ BOUNDARY ELEMENT CAPACITY ADEQUATE

SHEAR WALL 1 + SHEAR WALL 2 REQUIRE ATLEAST 1 CURTAIN OF #7 VERTICAL REINFORCEMENT @ 12" o.c AND #6 HORIZONTAL REINFORCEMENT @ 6" o.c. BOUNDARY ELEMENTS SHOULD CONSIST OF (10) #10 REBAR EVENLY DISTRIBUTED THROUGH THE 24"x24" SECTION.

DESIGN PROVIDES 2 CURTAINS OF #10 VERTICAL REINFORCEMENT @ 6" o.c WITH #6 HORIZONTAL REINFORCEMENT @ 6" o.c FOR FIRST 7 STORIES. BOUNDARY ELEMENTS SAME AS REQUIRED FOR CAPACITY.

ASSUME SHEAR WALL 3 AND 4 ADEQUATE SINCE IN ACTUALITY WALLS SW3 + SW4 ARE PART OF A LARGER WALL WITH OPENINGS THAT CREATE HORIZONTAL WALL SEGMENTS. EACH WALL ONLY NEEDS TO TAKE 13% OR 14% OF TOTAL LATERAL FORCES. SO ALL SHEAR WALLS ADEQUATE FOR DESIGN.