

Manhattan Center for the Performing Arts (MCPA)

Theo Dimitrasopoulos

Demi Fang

James Gales

A 60-story mixed-use high-rise in 8th Ave. and 45th St. in midtown Manhattan, serving as a university for the performing arts. With rehearsal spaces, classrooms, offices and retail space, as well as a 160'-span cantilevered auditorium, our proposal offers a vast range of functions aimed at establishing a strong educational and social presence in Broadway's theatre district.

PROGRAM

As indicated by Table 1 and Fig. 1, the program is dominated by the classroom space, rehearsal space, and performance space (auditorium) that make up the performing arts university. The university leases out the office space and will contract out the atrium+retail space to be completed by another engineer.

office	45,000 ft ²
classrooms	245,000 ft ²
rehearsal spaces	180,000 ft ²
cantilevered auditorium	10,000 ft ²
atrium + retail	64,000 ft ²
mechanical	21,250 ft ²

Table 1. Structure programming by square footage

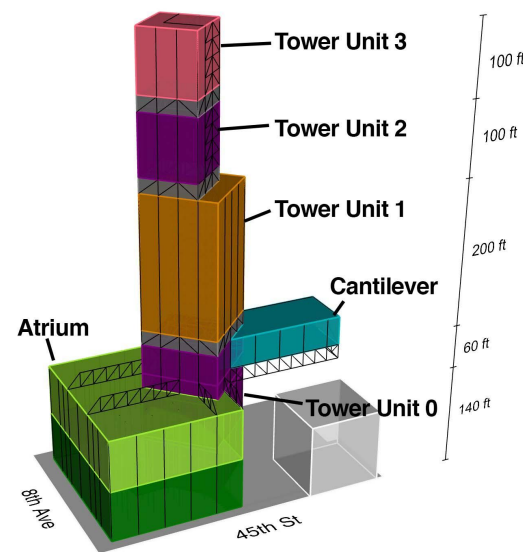


Fig. 1. Structure programming with colors as described in Table 1.

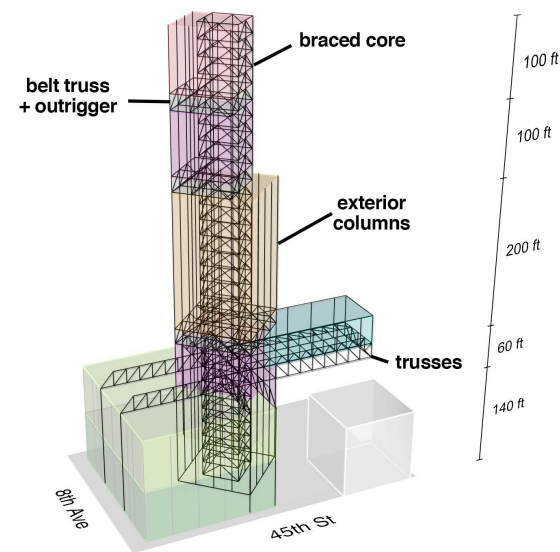


Fig. 2. Structural system overview.

STRUCTURAL SYSTEM OVERVIEW

Structural members are described in greater detail throughout the report, but Fig. 2 and the bullet list to the right illustrate the main structural features.

- Steel members
- Braced core (shear resistance)
- Belt truss + outrigger system (moment resistance)
- Trusses for load transfer of cantilever
- No column transfers within tower

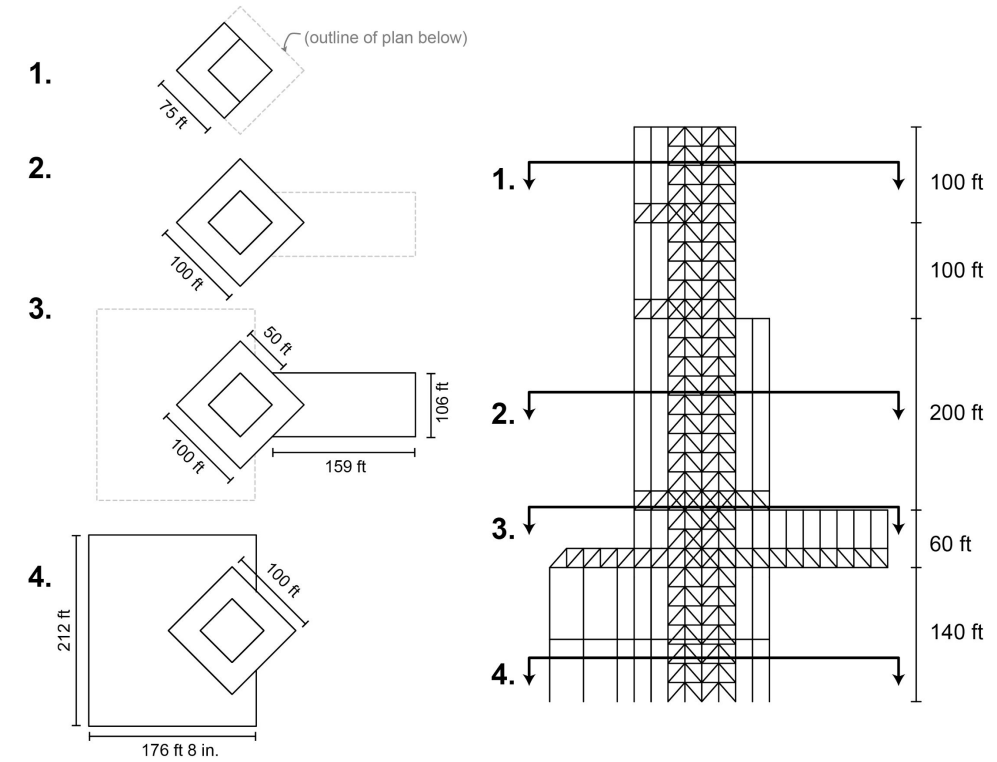


Fig. 3. North-facing elevation (right) with corresponding plans (left).

GRAVITY LOAD PATH: TOWER

The general gravity load path is typical, with loads transferring from slab to deck, deck to beam, beam to girder, and girder to column. No column transfers are needed in the tower.

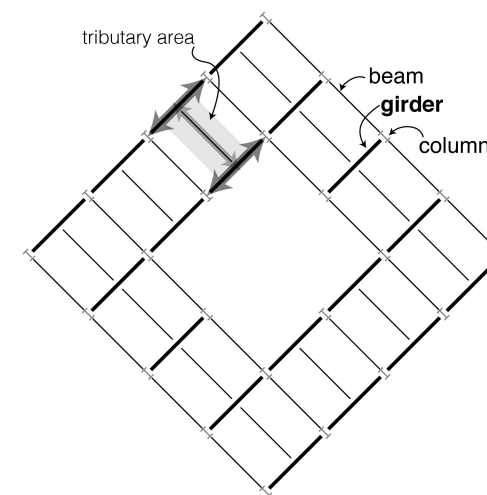


Fig. 4. Typical gravity load path at the plan level, illustrated here on typical Tower Unit 0 and 1 plan.

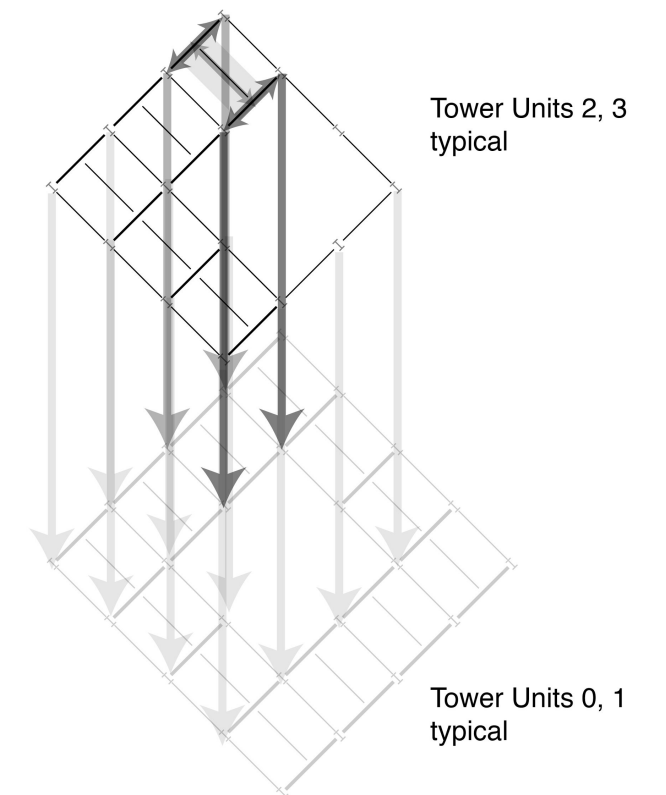


Fig. 5. Typical gravity load path at the elevation level, illustrated here on how Tower Units 2 and 3 transition into Tower Units 0 and 1. No column transfers needed.

GRAVITY LOAD PATH: CANTILEVER

Unlike the tower, the cantilever requires trusses to perform a load transfer of gravity into the main tower structure. Three trusses (dotted lines, Fig. 6 Plan) are laid upright underneath the cantilever and connect with the braced core of the tower.

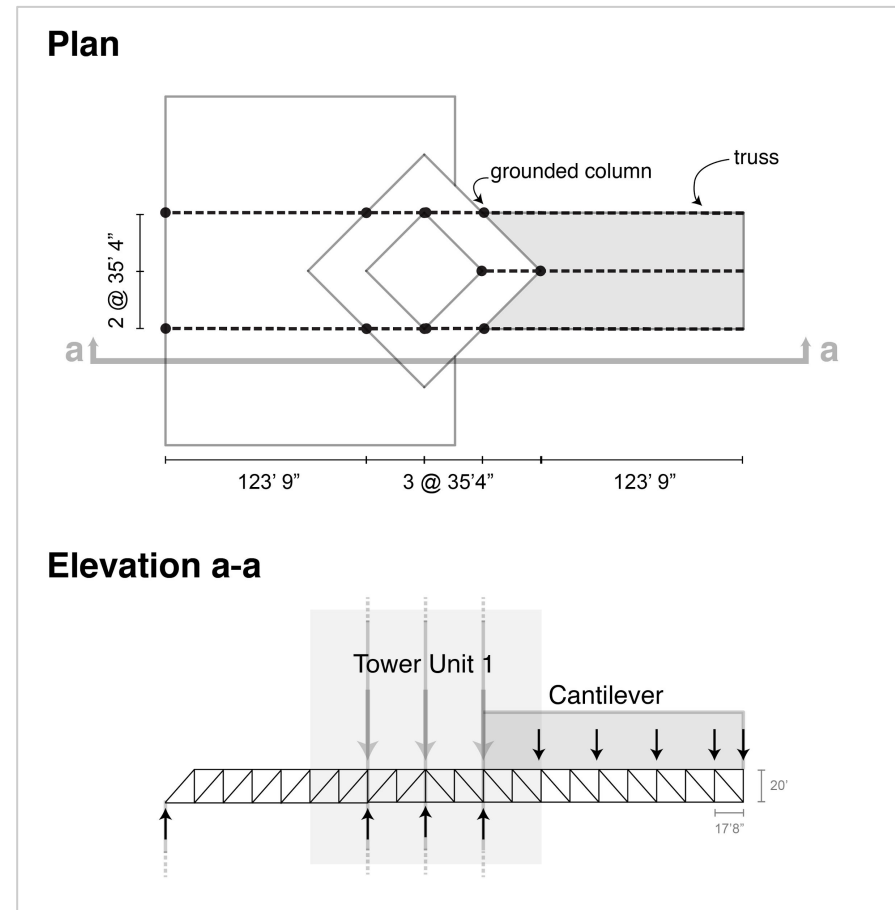


Fig. 6. Trusses make the load transfer from the cantilever (shaded) to the columns of the tower's braced core.

LATERAL LOAD PATH: TOWER SHEAR

Fig. 7. Lateral system overview. Non-vertical components of the braced core are engaged with shear resistance in the tower.

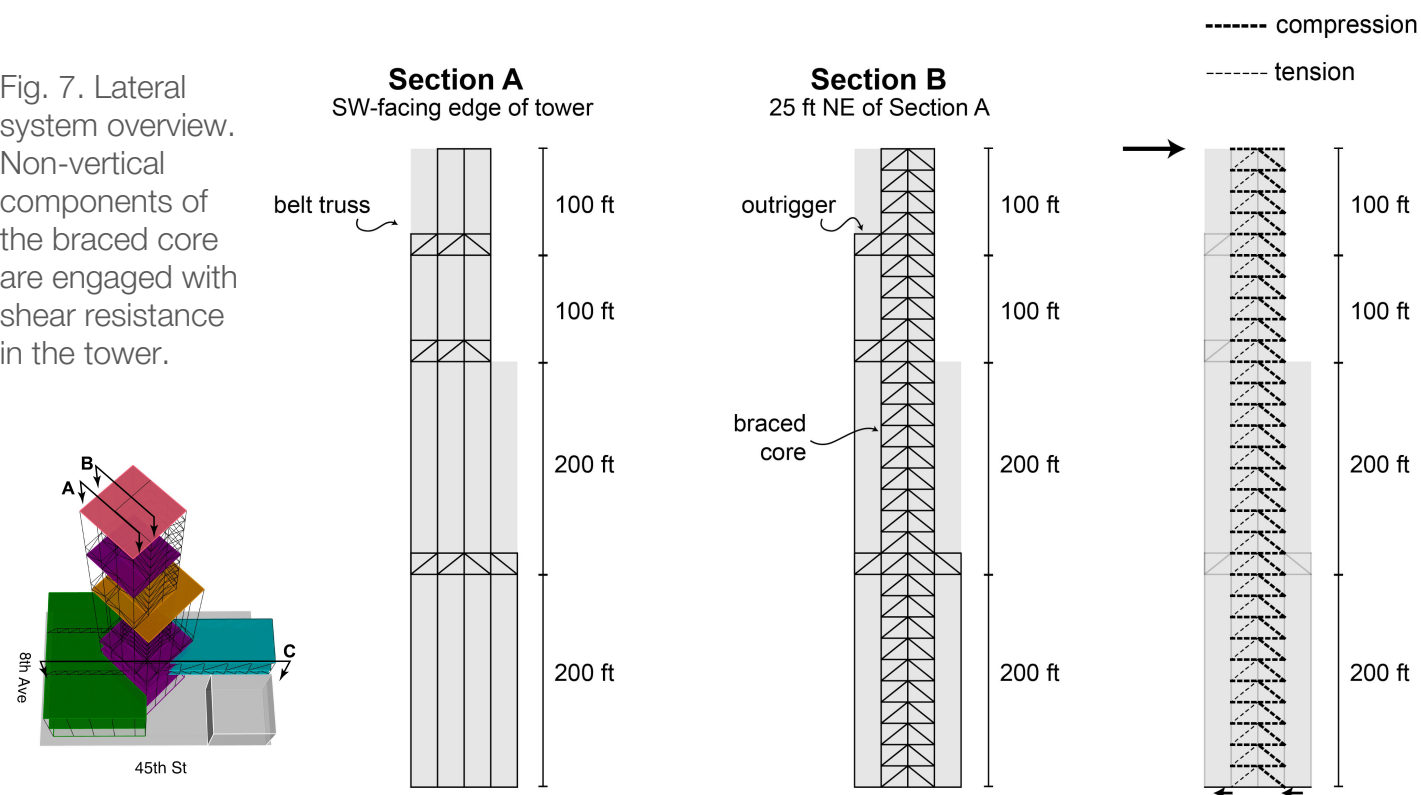


Fig. 7 provides an overview of the lateral system used. As the far right diagram in Fig. 7 illustrates, the shear in the tower is taken by the non-vertical members of the braced core.

East-west lateral loads pass through the cantilever into the braced core, as illustrated in Fig. 8. The more complex response to north-south lateral loads is discussed on the next page.

LATERAL LOAD PATH: CANTILEVER SHEAR

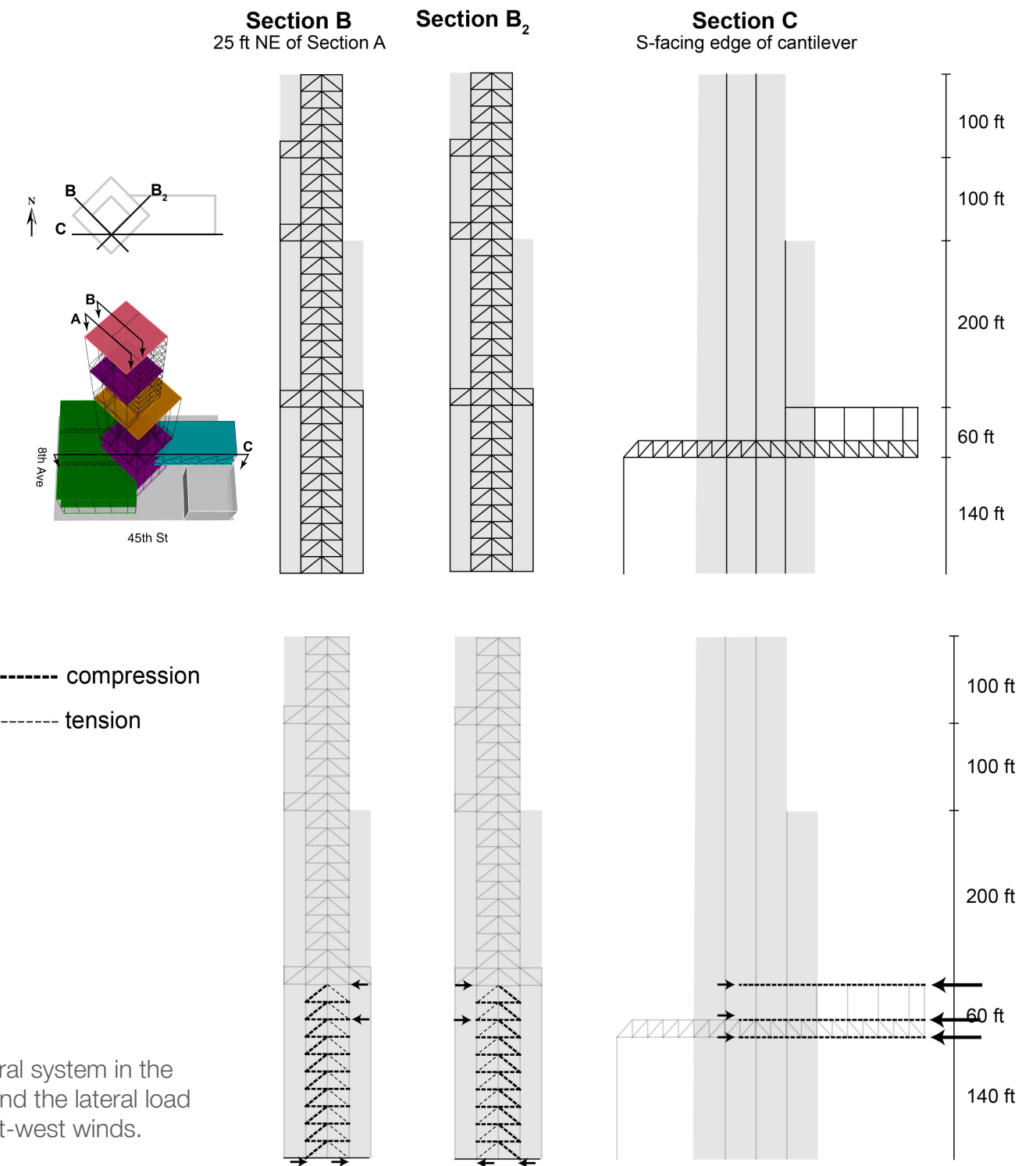


Fig. 8. Lateral system in the cantilever and the lateral load path of east-west winds.

LATERAL LOAD PATH: CANTILEVER SHEAR

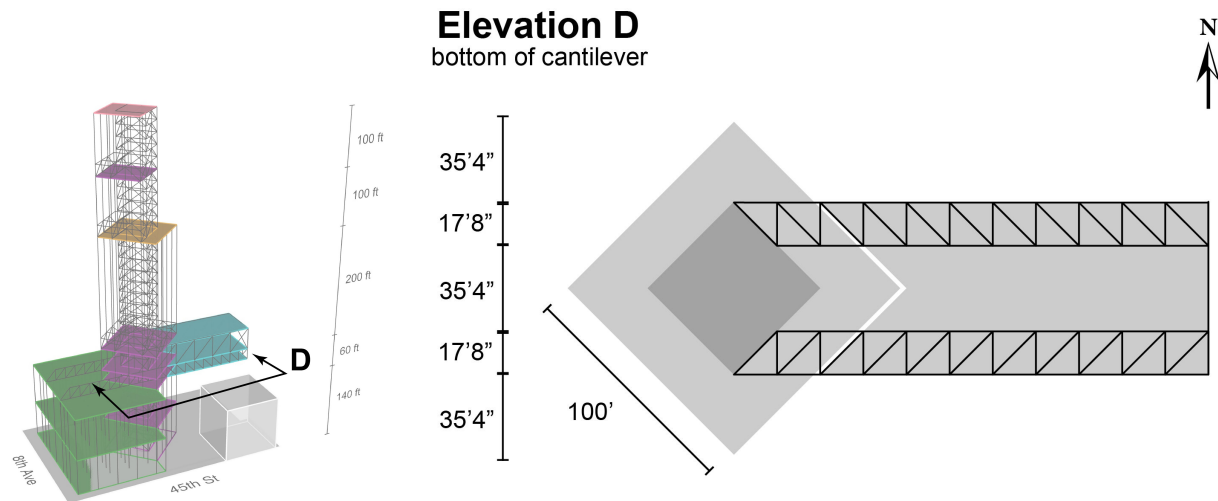


Fig. 9. Plan view of lateral system in cantilever.

Fig. 9 provides an overview of the two trusses laid flat underneath the cantilever to join with the braced core of the tower.

As illustrated in Fig. 10, lateral loads in the north-south direction transfer through the trusses and produce lateral forces acting along the braced core. The non-vertical members of the braced core carry these down to the base of the tower (Fig. 10 left).

The base reactions counter the “twisting” effect of wind on the cantilever.

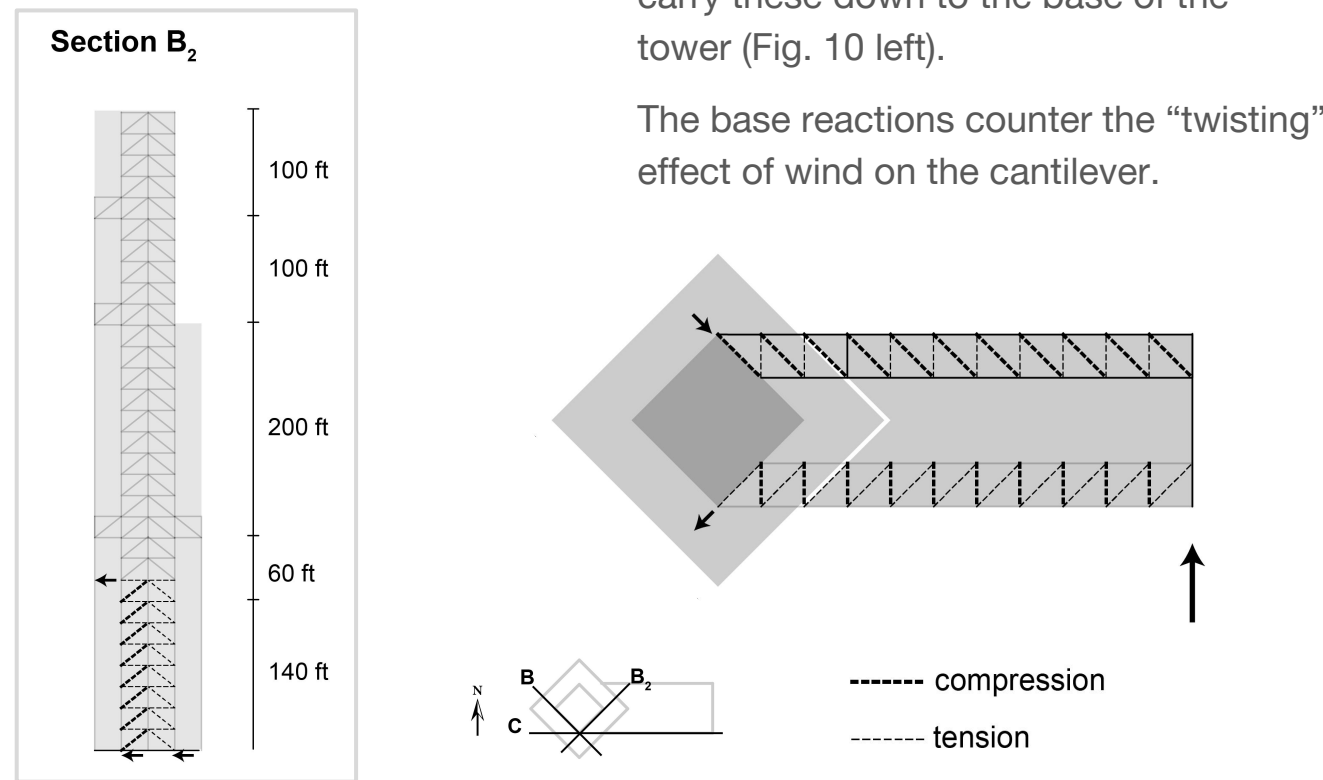


Fig. 10. Lateral load path of north-south lateral forces in the cantilever.

LATERAL LOAD PATH: MOMENT

Fig. 11 illustrates the lateral system components responsible for moment resistance in the tower. Belt trusses and outriggers extend the moment arm from the braced core, reducing the moment taken in the braced core (as will be seen in the overturning moment diagrams in a later section.)

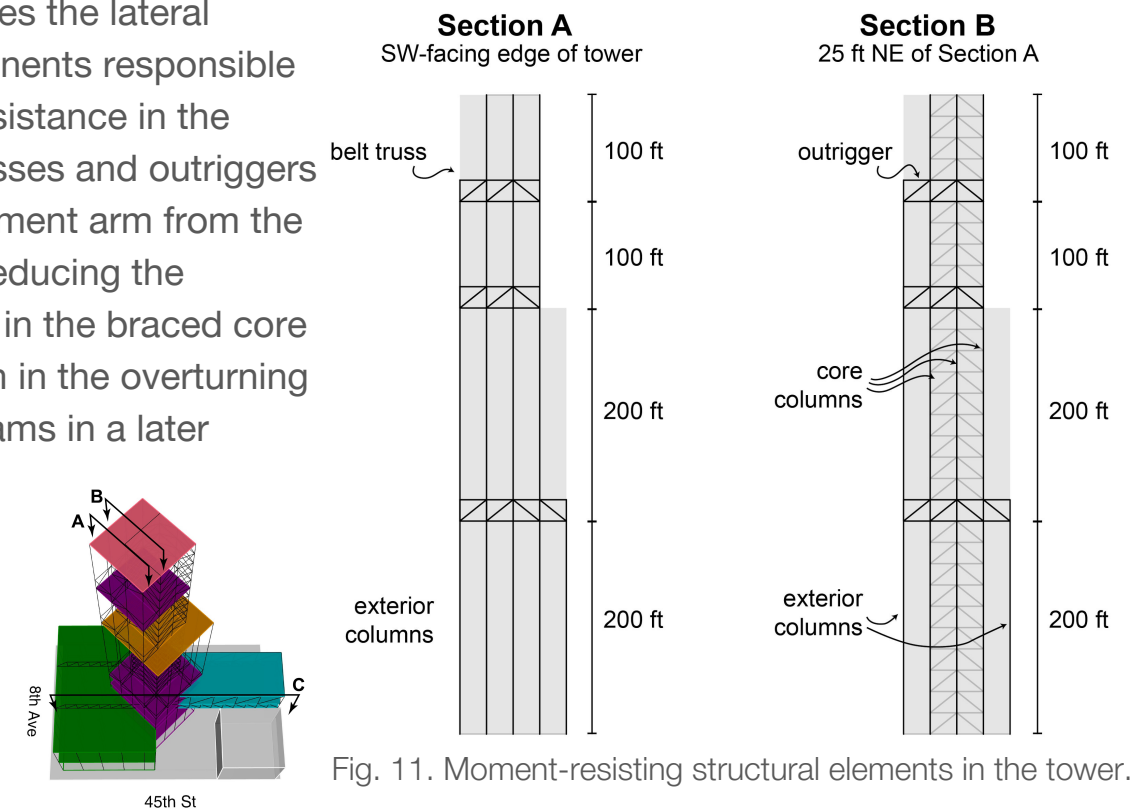


Fig. 11. Moment-resisting structural elements in the tower.

A final system of outriggers is introduced at the bottom of the cantilever. Two are extensions of the existing trusses involved in gravity load transfers. These outriggers extend the moment arm beyond the braced core to the atrium exterior columns to reduce moment taken in the braced core (Fig. 12).

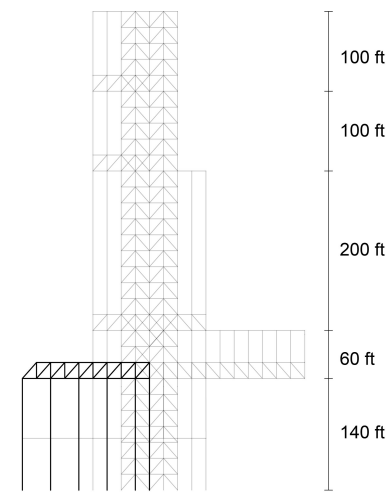
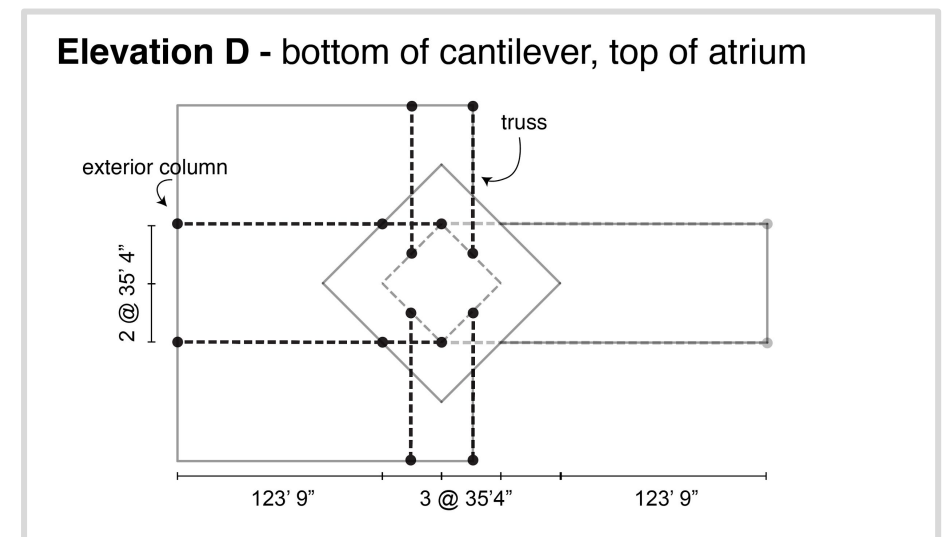


Fig. 12. Upright trusses connect the braced core to the atrium exterior columns as another moment reduction system.



LOAD COMBINATIONS

gravity only	$1.2D + 1.6L$
gravity and lateral loads (wind)	$1.2D + 1.0W + L$ $0.9D + 1.0W$
gravity and lateral loads (earthquake)	$1.2D + 1.0E + L$ $0.9D + 1.0E$

D = dead load
L = live load
W = wind load
E = earthquake load

Table 2. Load combinations examined for member design (ASCE 7-10 Section 2.3.2)

GRAVITY LOADS (D and L)

Construction Dead Loads	
3" concrete slab	56 psf
18-gauge steel deck	3 psf
self-weight of steel members	500 pcf

Superimposed Dead Loads	
flooring (linoleum, hardwood, quarry tile)	1, 4, or 16 psf
mechanical duct allowance	4 psf
ceilings with acoustical fiberboard	9 psf
1" glass cladding	180 pcf

Table 3. Dead loads applied (ASCE 7-10 Table C3-1)

Live Loads	
office	40 psf
classroom	40 psf
rehearsal	100 psf
auditorium bowl seating	150 psf
auditorium stage	120 psf
partition walls (if LL < 100 psf)	55 psf

Table 4. Live loads applied (ASCE 7-10 Table 4-1)

WIND LOADS (W)

Key parameters

- Risk Category II
- V = 120mph
- $K_d = 0.85$
- Surface Roughness = B
- Exposure Category = C
- $k_{zt} = 1$
- G = 0.85
- Enclosed Building
- Minimum Wind load design = 16 psf

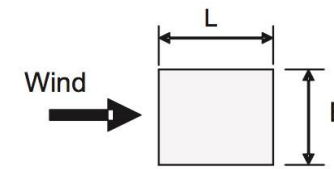


Fig. 13. L and B definitions (ASCE 7-10 Section 26)

Detailed wind calculations are provided in Appendix 1. Differing values of P_i are a result of the changing L/B ratio in the building footprint (Fig. 13.)

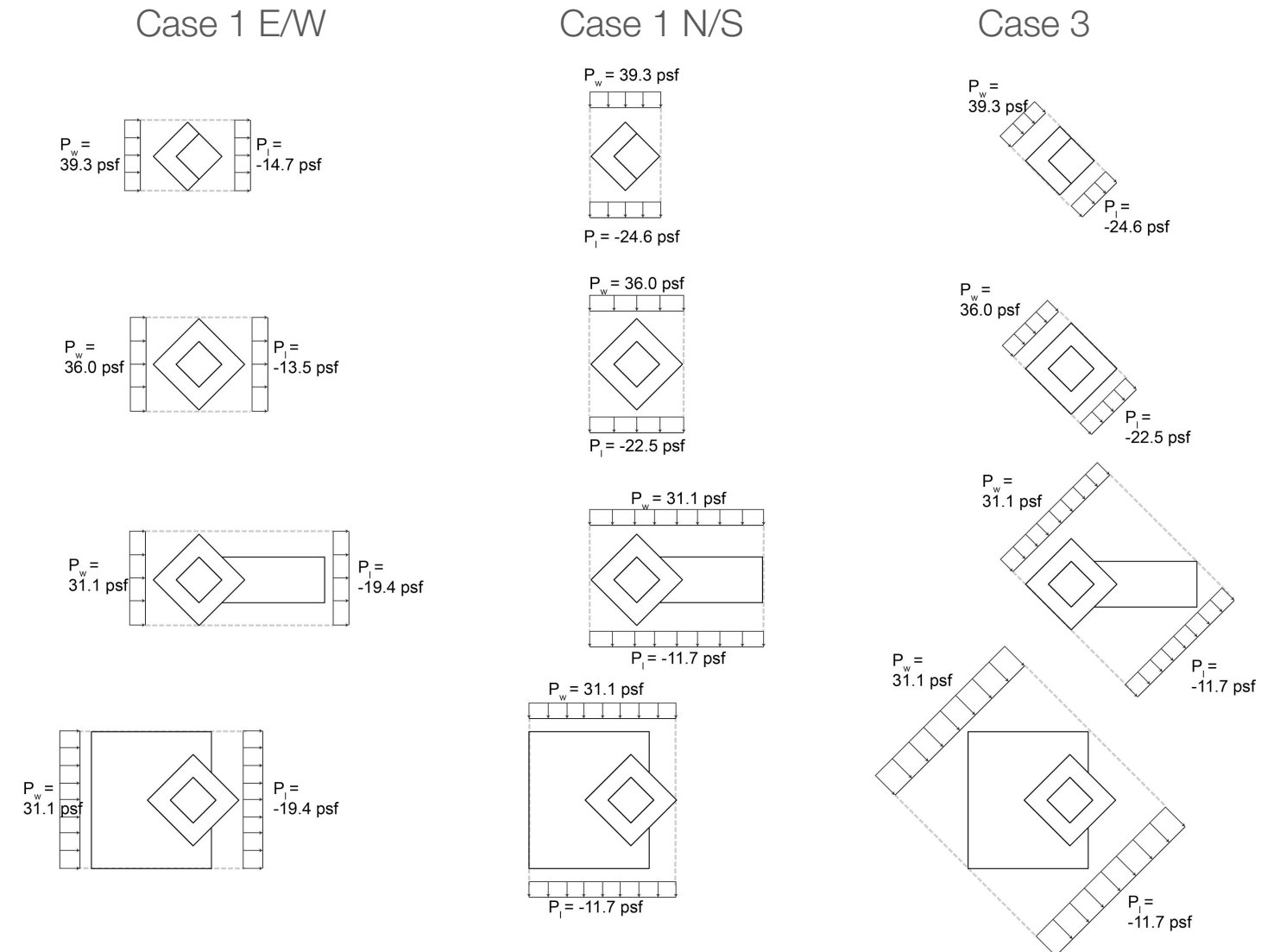


Fig. 14. Wind pressures at varying elevations of the structure. Cases 1 and 3 were assessed.

EARTHQUAKE LOADS (E)

Key parameters

- Risk Category II
- Occupancy Category III
- R = 8.00 (torsional irregularities due to the cantilever)

Detailed earthquake parameters are presented in Appendix 2. Calculations are made based on ASCE 7-10 Section 21-22.

Base Shear:

$V = C_s W$, where W is weight of the structure

Type	Area (ft ²)	Unit (psf)	Weight (k)	
Roof	52,782.5	85	4,487	Total (k)
Floor	501,240	85	42,605	47,092

Horizontal/Vertical Distribution of Seismic Forces:

Vertical:

$F_x = C_{vx} V$, where $C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^n w_i h_i^k}$

Horizontal:

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

Finally,

$V = C_s W = 0.0455 * 34,629 = 2,143 \text{ k}$

LATERAL LOAD COMPARISON

It was observed that Case 3 winds achieved base shears and moments under those of Case 1. Therefore, between wind Cases 1 and 3, Case 1 governed.

Shear experienced at each floor could be calculated by calculating the cumulative shear of floors above the given floor. For simplicity, in wind cases these shears were only computed for the bottom floors of each unit.

Overtuning moment diagrams are created by taking the area under the curve of elevations above a given elevation. In the overturning moment diagrams of Fig.s 15-17, the dotted lines represent the total moment taken by the braced core. Recall the outrigger systems introduced in the lateral load path discussion; these systems reduce moment by 40% (tower outriggers) and 50% (atrium outriggers.)

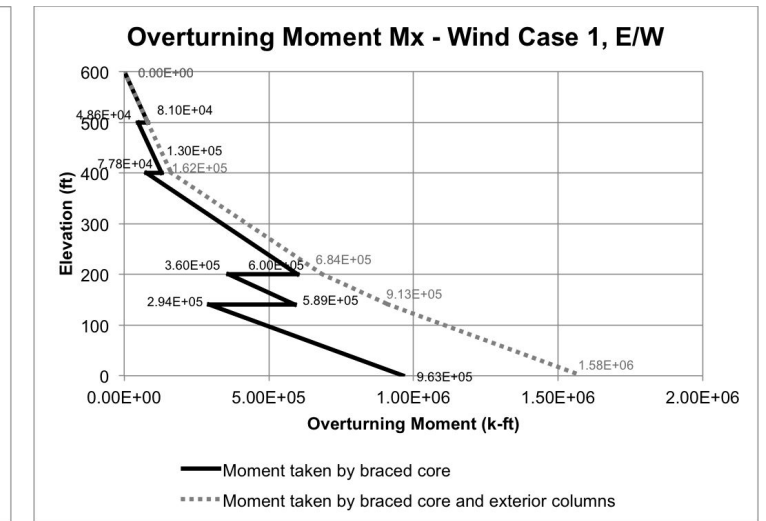
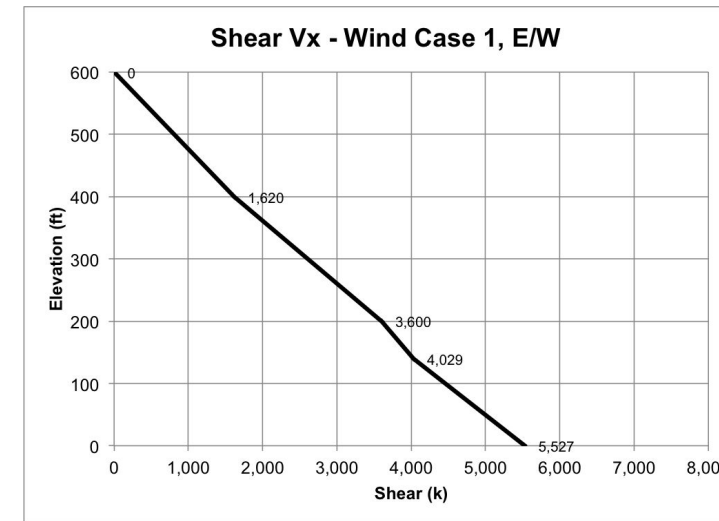


Fig. 15. Shear and overturning moment due to Wind Case 1 in the E/W direction.

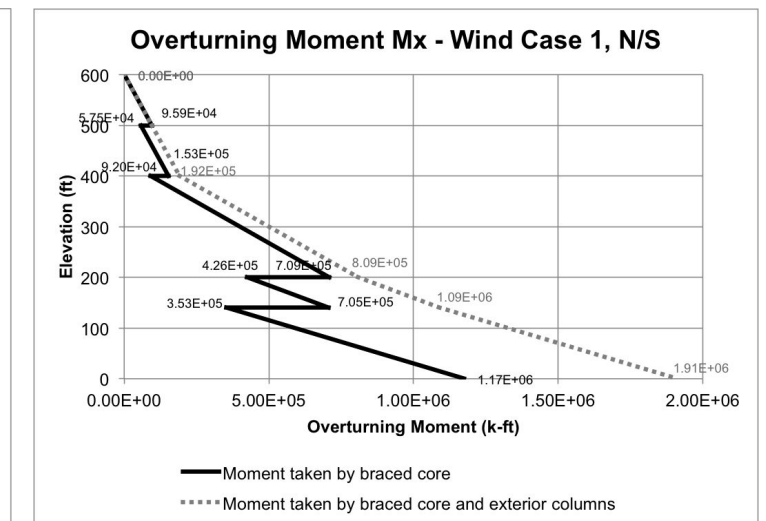
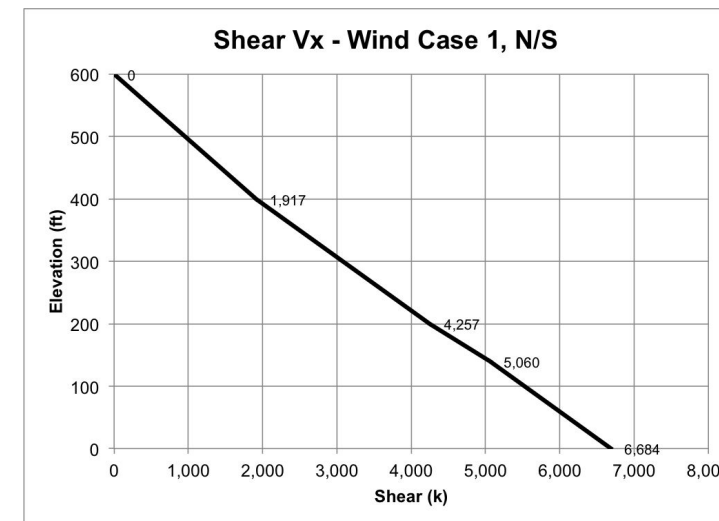


Fig. 16. Shear and overturning moment due to Wind Case 1 in the N/S direction.

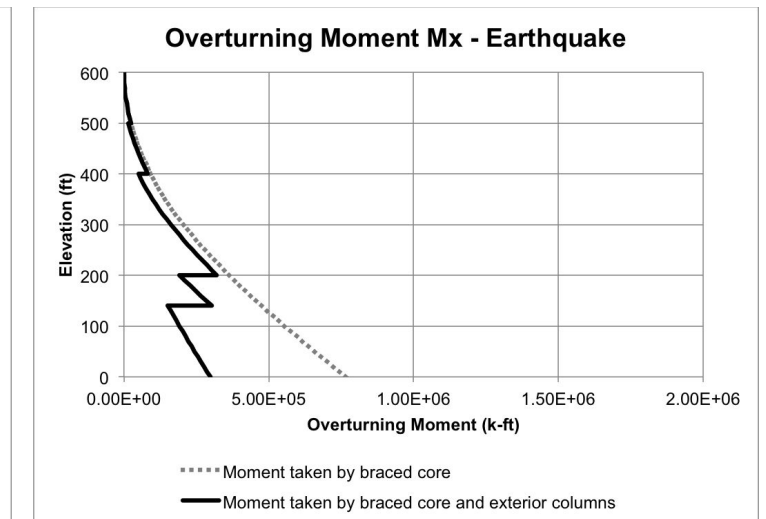
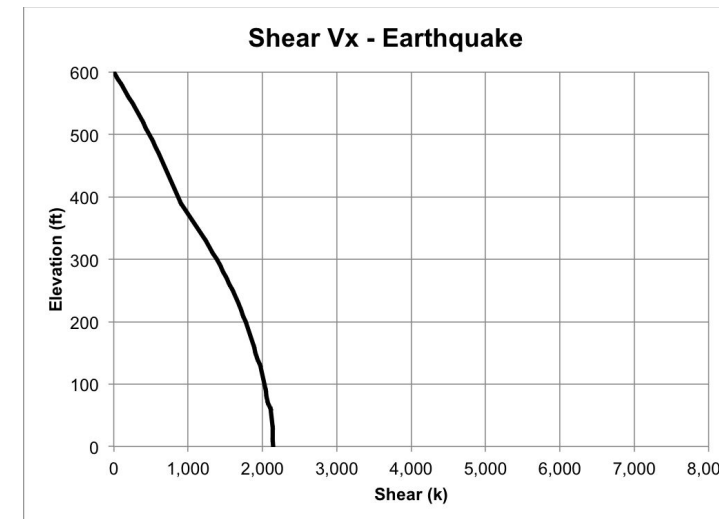


Fig. 17. Shear and overturning moment due to earthquake loads..

Note that the cardinal directions are taken loosely to align with the street grid, though this is not the case in reality.

LATERAL LOAD COMPARISON (continued)

Comparing shear and overturning moment at the base and at all elevations in Figs 15-17, it is clear that **wind loads dominate over earthquake loads**. The earthquake load combinations may be neglected.

Both axes of wind Case 1 should be considered in design because they engage different members.

MEMBER DESIGNS

Understanding the load paths and design loads on the structure informs the design of structural members.

Because the beams and girders are only involved in the gravity load path (as opposed to lateral), beams and girders throughout the structure were designed based on the gravity load combination $1.2D+1.6L$.

Columns were considered under all load combinations and designed for their worst case induced axial load (which sometimes required tension.)

Trusses (braced core, outriggers, cantilever trusses) were also considered under all load combinations and designed for their worst case loads.

	D	L	
Tower Unit 3	76 psf	95 psf	
Tower Unit 2	76 psf	95 psf	
Tower Unit 1	76 psf	100 psf	
Tower Unit 0	76 psf	95 psf	
Mechanical	73 psf	150 psf	
Canti-lever	corridors	77 psf	100 psf
	stage	77 psf	120 psf
	seating	77 psf	150 psf

Dead and live loads applied on tributary areas for beam and girder designs, determined by combining values from Tables 3 and 4 based on program. Does not include CDL such as glass cladding or self-weight, which is not applied to beams by tributary area

TOWER BEAM DESIGN

ID	TB1	TB2
size	W18x76	W18x97
length (ft)	25	25

Table 5. Floor beams in the tower.

ID	TB1	TB2
size	W14x43	W10x49
length (ft)	25	25

Table 6. Roof beams in the tower.

Two tower beam (TB) types (based on tributary areas) were identified in the floor plans (Figs 18 and 19). For a quicker design process, the heaviest dead and live load combination in the tower (mechanical floors) was used to design the same TB1 and TB2 at all floors, listed in Table 5. This resulted in efficiencies ($M_u/\phi M_n$) varying from 53% to 99%. It is recognized that this design approach for convenience made for inefficient beam designs for some floors that did not require so much moment resistance.

Roof loads were significantly less (no live loads) and were designed with separate sizes, provided in Table 6. Roof framings are not provided here; the first roof is at the top of Tower Unit 3 and has a framing identical to that portrayed in Fig. 18. The second roof is portion of Tower Unit 1 not covered by Tower Unit 2.

Example beam calculations are provided in Appendix 3.

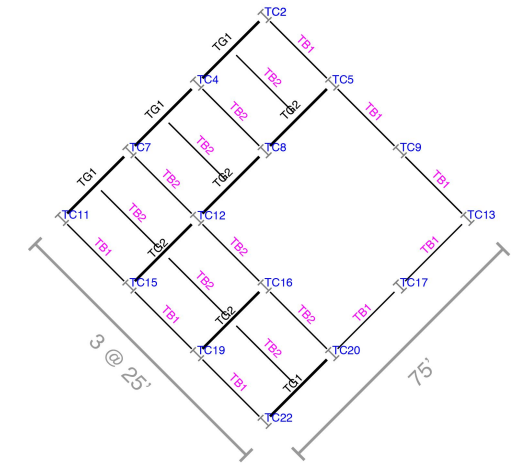


Fig. 18. Typical floor framing at Tower Units 2 and 3.

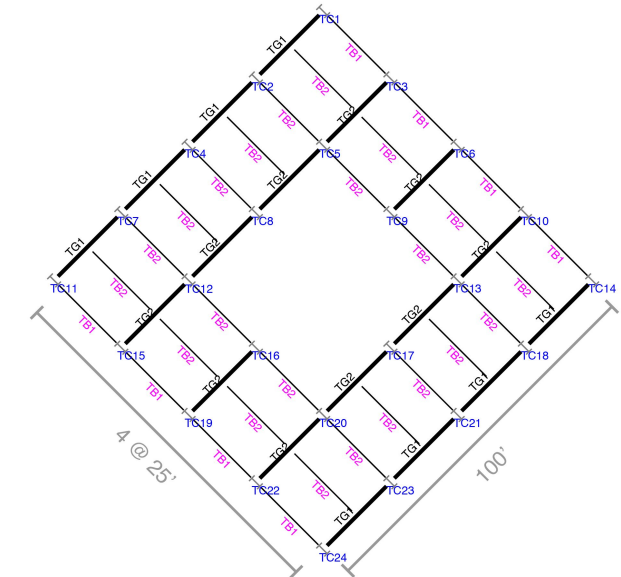


Fig. 19. Typical floor framing at Tower Units 0 and 1.

TOWER GIRDER DESIGN

ID	TG1	TG2
size	W18x55	W21x55
length (ft)	25	25

Table 7. Floor girders for non-mechanical floors.

ID	M-TG1	M-TG2
size	W21x62	W21x62
length (ft)	25	25

Table 8. Floor girders for mechanical floors.

ID	TG1	TG2
size	W14x30	W16x40
length (ft)	25	25

Table 9. Roof girders.

Two tower girder (TG) types were identified using floor framing (Fig. 18 and 19). The load combinations experienced at non-mechanical floors were approximated to be the same; girders on non-mechanical floors were designed based on the heaviest non-mechanical program load combination in the tower and are tabulated in Table 7.

Example girder calculations are provided in Appendix 4.

CANTILEVER BEAM DESIGN

ID	CB1	CB2	CB3	CB4	CB5	CB6
size	W14x90	W18x143	W18x143	W14x90	W8x18	W8x10
length (ft)	35.33	35.33	35.33	26.50	8.83	4.42

Table 10. Floor beams for cantilever

ID	CB1	CB2	CB3	CB4	CB5	CB6
size	W8x67	W12x72	W12x72	W10x45	W8x10	W8x10
length (ft)	35.33	35.33	35.33	26.50	8.83	4.42

Table 11. Roof beams for cantilever

Six cantilever beam (CB) types were identified from the floor framing of the cantilever (Fig. 20).

It is recognized that some beam lengths are longer than usual; deflection checks are key in these situations.

CANTILEVER GIRDER DESIGN

ID	CG1	CG2	CG3	CG4	CG5	CG6	CG7	CG8
size	W8x21	W21x48	W21x48	W14x34	W12x26	W36x160	W21x68	W14x34
length(ft)	17.67	35.33	35.33	35.33	17.67	70.67	35.33	35.33
efficiency	86.37%	93.58%	93.58%	92.77%	91.97%	98.13%	90.37%	92.77%

Table 12. Roof girders for cantilever

Eight cantilever girder (CG) types were identified from the floor framing of the cantilever (Fig. 20). Roof girder sizes are provided in Table 12.

The auditorium level beams are supported directly by the cantilever trusses (dotted, Fig. 20).

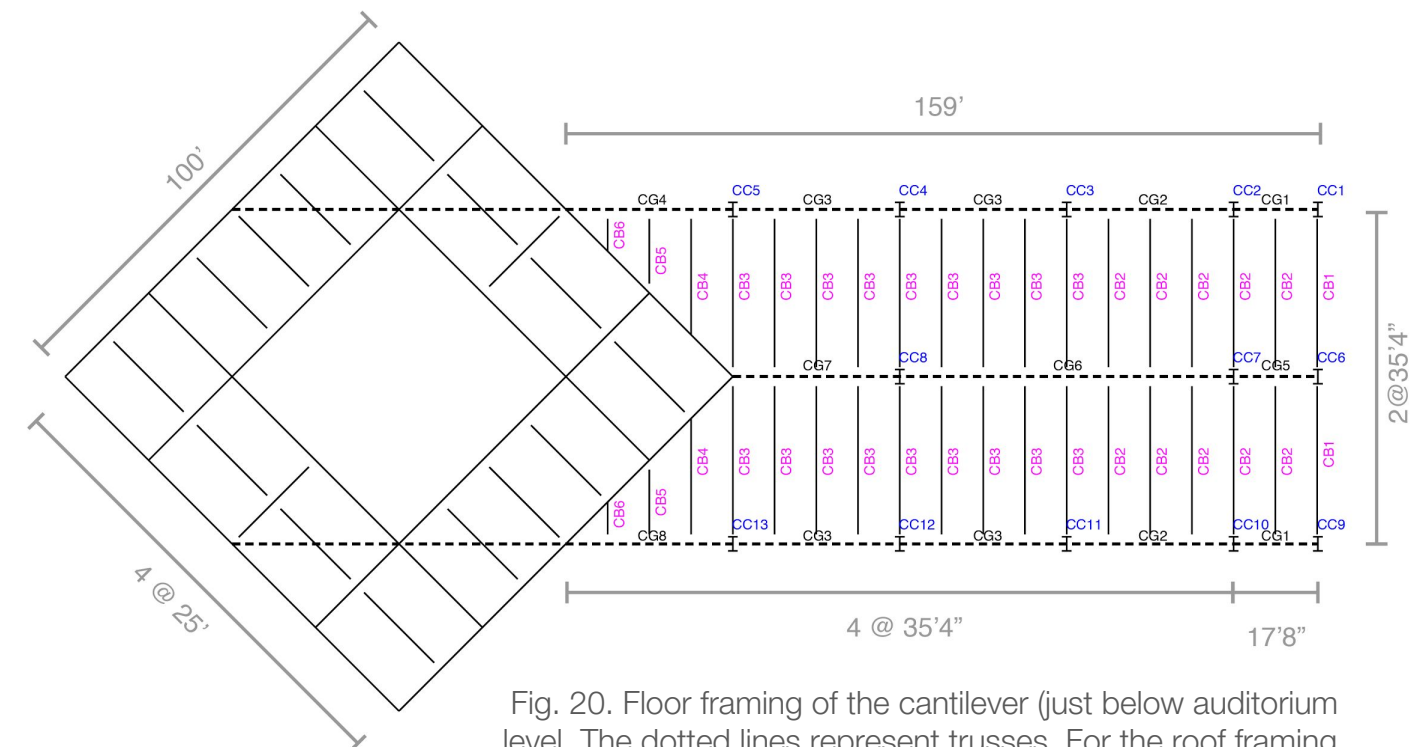


Fig. 20. Floor framing of the cantilever (just below auditorium level). The dotted lines represent trusses. For the roof framing, the dotted lines would represent girders.

TOWER COLUMN DESIGN

For a more efficient design process, columns are designed at the base of each unit. The axial effect of load combinations of gravity and wind, as outlined in Table 2, are tabulated, and the member is designed for the worst case axial load.

As an example, column TC8 of Tower Unit 3 is designed below. With $D = 76$ psf and $L = 95$ psf, column TC8 experiences the following axial loads:

1.2D+1.6L	1.2D+L	0.9D
1154 k	1069 k	370 k

Table 13. Axial loads experienced by column TC8 at the base of Tower Unit 3 (see also Fig.18.) Positive is compression in this table.

The moment experienced by the columns at the base of Tower Unit 3 (elevation = 500 ft) due to wind can be read off the overturning moment chart of the relevant wind case and direction.

From Fig. 16, the moment at elevation = 500 ft from N/S winds is 95,900 k-ft at the braced core (TC8 is a braced core column).

From Fig. 15, the moment at elevation = 500 ft from E/W winds is 81,000 k-ft at the braced core.

In order to determine the axial loads induced in the columns due to these moments, the quantity and length of moment arms for each wind case must be determined.

Fig. 21 illustrates these moment arms. In both wind direction cases, TC8 is one of 3 column couples in the braced core (thicker dotted lines, with squares representing columns in the braced core) that resist wind.

Each couple in the N/S direction resists $95,900 \text{ k-ft} / 3 = 31,967 \text{ k-ft}$ of moment. TC8 is in a couple distanced 35 ft apart; the axial load induced in TC8 from N/S wind is thus $31,967 \text{ k-ft} / 35 \text{ ft} = 913 \text{ k}$.

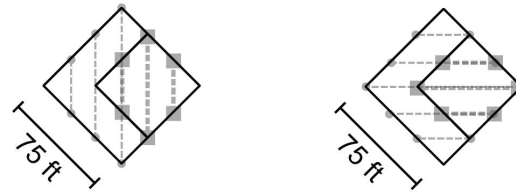


Fig. 21. Simplified typical floor plans of Tower Unit 3. Dotted lines visually represent the lengths of lever arms working to resist moment. Left: lever arms resisting N/S wind; right: lever arms resisting E/W wind.

Likewise, each couple in the E/W direction resists $81,000 \text{ k-ft} / 3 = 27,000 \text{ k-ft}$ of moment. TC8 is in a couple distanced 35 ft apart; the axial load induced in TC8 from N/S wind is thus $27,000 \text{ k-ft} / 35 \text{ ft} = 771 \text{ k}$.

TC8 could be receiving these axial loads as compressive or tensile ones based on the wind direction (e.g. north-blowing wind would induce compression, while south-blowing wind would induce tension.)

For this column, $913 \text{ k} > 771 \text{ k}$, so winds in the N/S direction govern for axial loads induced by winds on TC8.

The loads experienced by TC8 due to different load combinations can then be updated, as is tabulated in Table 14. A compressive requirement of 1982 k governs the design. Appendix 5 details column design for a given compressive requirement; W12x90 is selected.

Additionally, TC8 may experience a tension of up to 543 k. The designed section is checked for yielding given a tensile strength of 50 ksi steel. In the case of a W12x90 section selected for TC8, the tensile strength is $0.9 * 50 \text{ ksi} * 61.8 \text{ in}^2 = 2781 \text{ k}$, which is well over the required tensile strength.

This procedure of checking load combinations and designing columns is repeated throughout the base of each tower unit. (Effects of the cantilever are also considered; see the cantilever design for more.) The column designs selected are presented in Table 15.

1.2D+1.6L	1.2D+L+1.0W		0.9D+1.0W	
--	<i>moment resistance via compression</i>	<i>moment resistance via tension</i>	<i>moment resistance via compression</i>	<i>moment resistance via tension</i>
1154 k	1069+913 = 1982 k	1069-913 = 156 k	370+913 = 1283 k	370-913 = -543 k

Table 14. Axial loads induced in column TC8 of Tower Unit 3, considering the possibility of either compressive or tensile resistance of moment. Positive indicates compression and negative indicates tension.

BRACED CORE DESIGN

Fig. 16 left indicates that 1917 k of shear in the north/south direction is experienced at the base of Tower Unit 2 (i.e. elevation = 400 ft). (Both wind load combinations specify 1.0W, so the shear value from Fig. 2a. is used for design.)

As Fig. 22 illustrates, the 1917 k shear must be decomposed into vectors aligning with the windward faces of the braced core. Each planar truss of the braced core takes $1917 * 0.707 = 1356 \text{ k}$ (dotted vector in Fig. 22). The truss panel dimensions determine the axial load experienced in the diagonal members. The edges of the truss panel are 25' and 20', so the diagonal has length $\text{sqrt}(25^2 + 20^2) = 32'$. By symmetry, this horizontal force is taken equally by truss members 1 and 2. Using proportions, the compressive force experienced in truss member 2 is thus $1356 \text{ k} / 2 * 32/25 = 868 \text{ k}$. The equivalent tensile force is experienced in truss member 1. Finally, truss members 3 and 4 are in 1356 k of compression.

The vertical members in Fig. 5, section a-a are zero force members under lateral load and are designed as columns using the gravity load combination of 1.2D+1.6L.

It is observed from comparing Fig.s 15 (left) and 16 (left) that shear experienced due to Case 1 N/S winds consistently dominates over Case 1 E/W winds at every floor. Thus, only Case 1 N/S winds are used to design the braced core. Because shear is greatest at the bottom of each unit, the design of these non-vertical members of the braced core for the base of each structure unit is used throughout the above braced core trusses of the same unit.

From symmetry (wind loads can be applied in either direction), truss members 1 and 2 are designed as the same size with the requirement of needing to withstand the same magnitude of tensile and compressive force. Braced core truss loads and designs presented in Tables 16 and 17. Compressive members designed as with columns in Appendix 5, though elastic buckling controlled for these truss elements ($F_{cr} = 0.877 F_e$). See Appendix 7 for connection detail.

Column Type	Tower Unit 3		Tower Unit 2		Tower Unit 1		Tower Unit 0	
	P _u	Size	P _u	Size	P _u	Size	P _u	Size
Type 1 (TC1, TC14, TC24)	n/a	n/a	n/a	n/a	846 k	W14x90	1721 k	W14x176
Type 2 (TC2, TC22)	412 k	W12x58	760 k	W12x87	2383 k	W14x233	4110 k	W14x426
Type 3 (TC3, TC6, TC10, TC18, TC21, TC23)	n/a	n/a	n/a	n/a	1636 k	W14x176	3320 k (-900 k)	W14x311
Type 4 (TC4, TC7, TC15, TC19)	800 k	W12x96	1500 k	W14x145	3053 k	W14x311	4826 k	W14x455
Type 5 (TC5, TC20)	1078 k (-182 k)	W14x109	2000 k (-343 k)	W14x193	6921 k (-1862 k)	W14x665	11,661 k (-6070 k)	W14x730 + 3" pl on flanges
Type 6 (TC8, TC16)	1814 k (-550 k)	W14x176	3111 k (-800 k)	W14x311	10,203 k (-5400 k)	W14x730 + 1.5" pl on flanges	16,600 k (-9000 k)	W14x730 + 6.5" pl on flanges
Type 7 (TC9, TC17)	1239 k (-772 k)	W14x145	2051 k (-1200 k)	W14x233	9168 k (-5764 k)	W14x730 + 1" pl on flanges	15,500 k (-9394 k)	W14x730 + 5.5" pl on flanges
Type 8 (TC12, TC13)	1600 k (-184 k)	W14x176	2830 k (-300 k)	W14x283	7400 k (-1000 k)	W14x665	11,800 k (-7200 k)	W14x730 + 3" pl on flanges
Type 9 (TC11)	400 k	W14x61	760 k	W12x87	1584 k	W14x159	2510 k	W14x257

Table 15. Tower column designs (positive compression, negative tension).

	Length	Required axial strength	Section selected	Compressive strength	Tensile strength
Tower Unit 3, Members 1 & 2	32 ft	+/- 434 k	W14x211	-462 k	+2790 k
Tower Unit 3, Members 3 & 4	25 ft	-678 k	W14x193	-172 k	--
Tower Unit 2, Members 1 & 2	32 ft	+/- 868 k	W14x370	-894 k	+4905 k
Tower Unit 2, Members 3 & 4	25 ft	-1356 k	W14x370	-1466 k	--
Tower Unit 1, Members 1 & 2	32 ft	+/- 1927 k	W14x605 + 1" plates on flanges	-1965 k	+9495 k
Tower Unit 1, Members 3 & 4	25 ft	-3010 k	W14x605	-3086 k	--
Tower Unit 0, Members 1 & 2	32 ft	+/- 3026 k	W14x730 with 3" plates on flanges	-3190 k	+14508 k
Tower Unit 0, Members 3 & 4	25 ft	-4726 k	W14x730 with 2.5" plates on flanges	-4941 k	--

Table 17. Braced core truss designs (tension positive.) Tensile strengths presented only where needed.

Elevation	Shear from Case 1 N/S		Axial load in truss member (tension positive)	
	from Fig. 2a	decomposed	Members 1 & 2	Members 3 & 4
500 ft	958.5 k	678 k	+/- 434 k	-678 k
400 ft	1917 k	1356 k	+/- 868 k	-1356 k
200 ft	4257 k	3010 k	+/- 1927 k	-3010 k
0 ft	6684 k	4726 k	+/- 3026 k	-4726 k

Table 16. Loads experienced in braced core truss members due to Case 1 N/S winds.

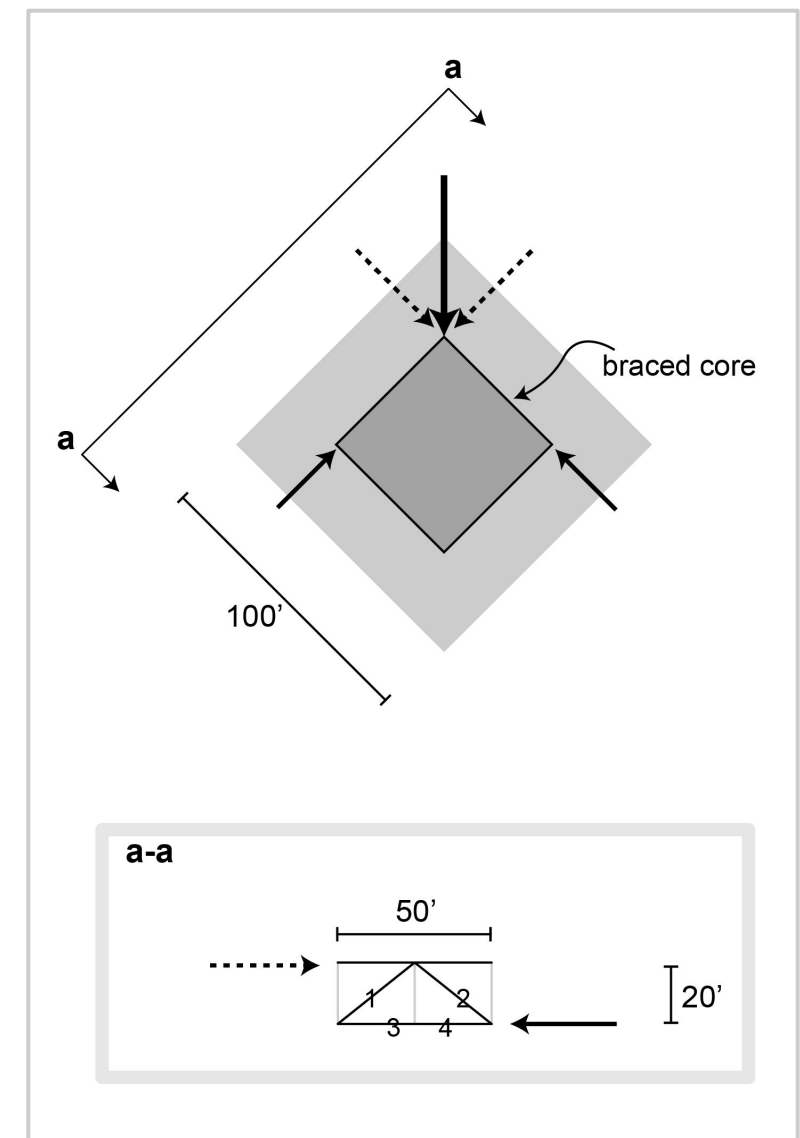


Fig. 22. Schematic for braced core truss design for wind blowing southward.

CANTILEVER DESIGN

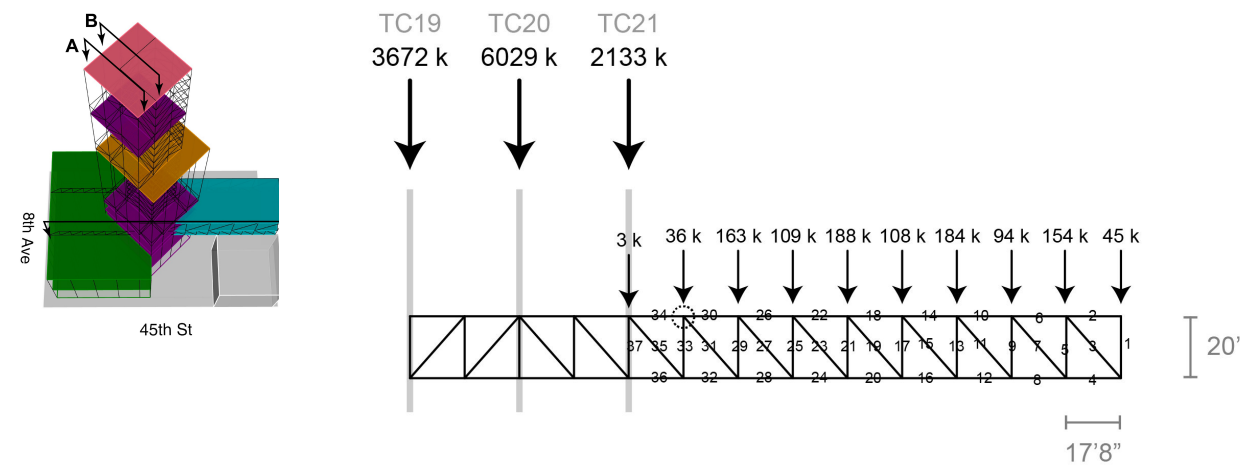


Fig. 23. Loads experienced by cantilever due to 1.2D+1.6L load combination.

The axial loads in the cantilever truss can now be updated to consider the wind load combinations, which changes the design requirement for the members. Because the planar truss runs east-west, only Case 1 E/W wind is considered. From Fig. 15 (left), the shear experienced at the bottom of the cantilever (elevation = 140 ft) is 4029 k.

This shear creates axial loads in the truss members that are larger than the axial loads induced by the 1.2D+1.6L combination alone. For example, see the following tabulation of axial loads in truss member 37 (from Fig. 23) based on different load combinations:

1.2D+1.6L	1.2D+L+1.0W	0.9D+1.0W
-5555 k	-9161 k	-6988 k

Table 18. Axial loads experienced in truss number 37 in the south-facing cantilever truss due to different load combinations (tension positive).

The member is thus designed for worst load combination. Truss member 37 has the highest compressive requirement in the truss examined; a W14x730 with 5" plates on flanges is selected for a compressive strength of 9242 k.

A connection detail of the circled joint in Fig. 23 is provided in Appendix 7.

FOUNDATION DESIGN

Designing for 4 columns ranging from 1259-16000 k. Three column categories are being considered based on the service load range:

- Category I: TC1,3,6,10,11,18,21,23,24 | **2400k design load**
- Category II: TC2,4,7,15,19,22 | **3500k design load**
- Category III: TC5,8,9,12,13,14,16,17,20 | **16050k design load**

	Col. size	Foot. size	q_u (k/ft ²)	d (in)	V_{u1} (k)	V_{u2} (k)	ϕV_c (k)	M_u (k-ft)
I	17"x17"	5'x5'	96	29"	989	560	1012	1.5e6
II	20"x20"	6'x6'	97	35"	1454	825	1461	2.6e6
III	30"x30"	14'x14'	82	89"	8008	4688	8038	2.4e7

Table 19: characteristic values per column category. Refer to Appendix 8 for detailed calculations.

600k rock anchors will be placed throughout footings corresponding to columns TC5, TC8, TC9, TC12, TC13, TC16, TC17 and TC20 experience tension (negative values provided in Table 15) to counteract the tensile forces that can develop in those columns.

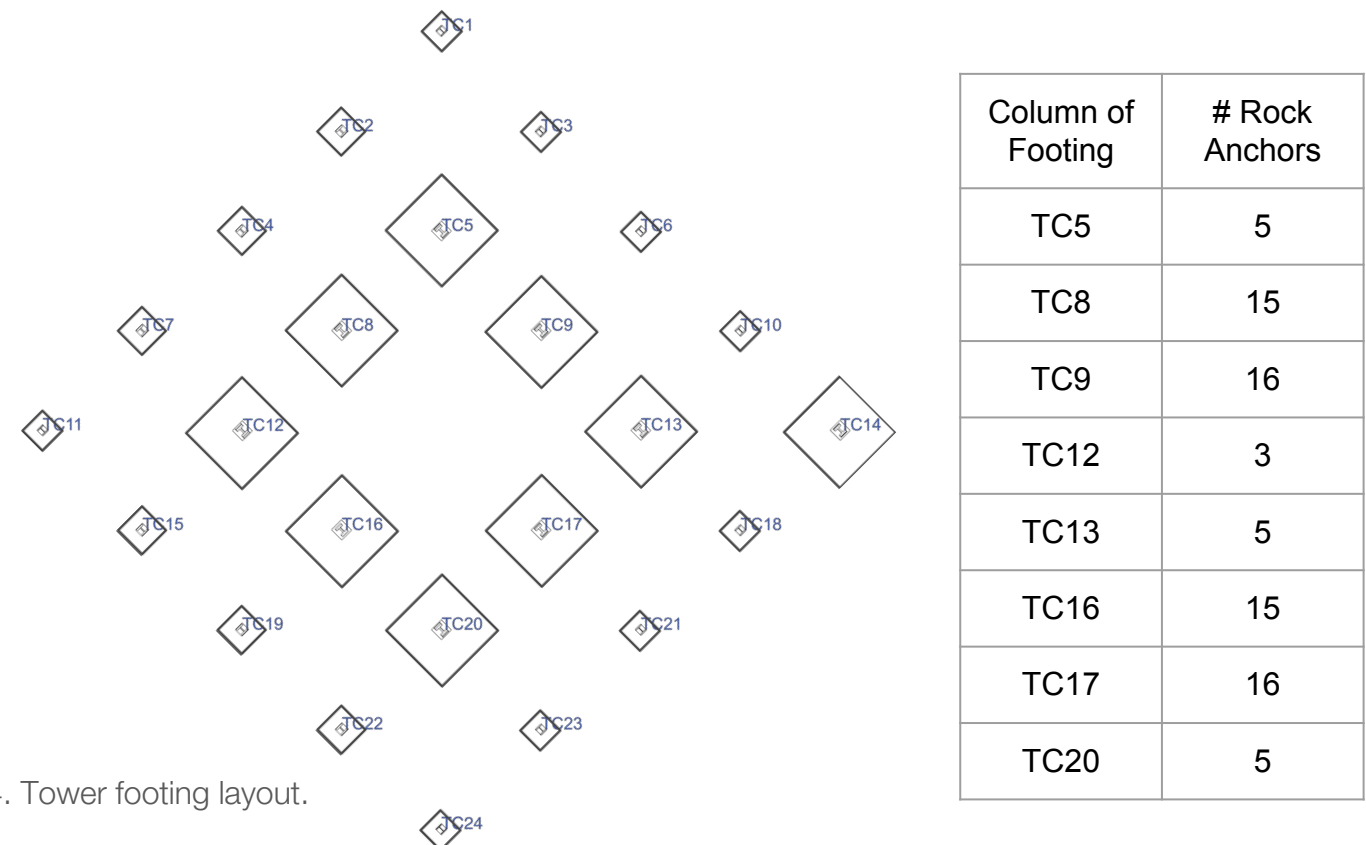


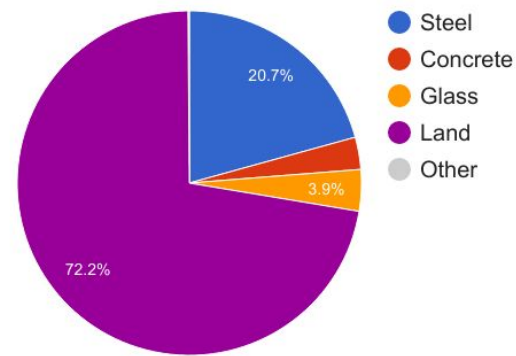
Fig. 24. Tower footing layout.

COST

Gross area: 437,240 ft²

Steel (\$5000/ton)	10,000 tons	\$50.0M
Concrete slab and foundations (\$56/ft ³)	130,650 ft ³	\$7.3M
Glass (\$40/ft ²) Curtain wall	235,560 ft ²	\$9.4M
Mechanical/Plumbing (\$100,000/floor)	3 floors	\$0.3M
Land (\$2000/ft ²)	87,200 ft ²	\$174M

TOTAL: \$241M
= \$551/ft²



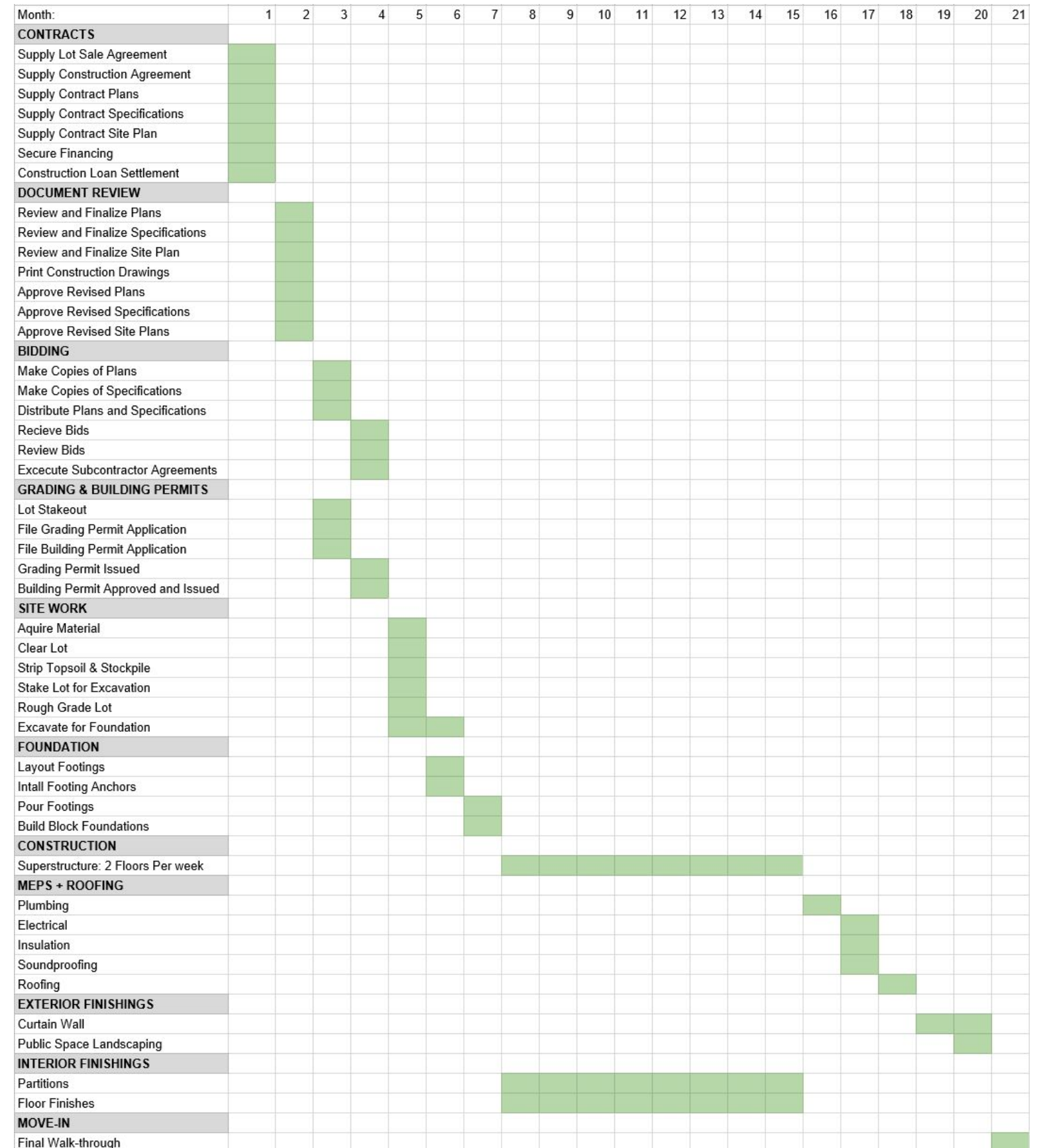
ANNUAL REVENUE

Tuition (25% of \$40,000 / student / year)	6000 students	\$60M
Endowment	Donor	\$1M
Theater Performances (\$10 / ticket / show)	800 seats	\$1.2M
Office Space (\$100 / ft ² / floor / year)	5625 ft ²	\$0.56M

TOTAL ANNUAL REVENUE: \$62.76M
= \$143.5/ft²

RETURN TIME
= 4 YEARS

CONSTRUCTION SCHEDULE



APPENDIX 1: WIND CALCULATIONS

Wind Calculations

Main Wind-Force Resisting Structure

References from ASCE 7-10

General Requirements:

Basic Wind Speed (V) [figure 26.5-1A-C]

Wind Directionality Factor (k_d) [section 26.6]

Exposure Category [section 26.8]

Topographic Factor (k_{zt}) [section 26.9]

Enclosure Classification [section 26.10]

Risk Category II

V=120mph

$K_d=0.85$ [table 26.6-1]

Surface Roughness=B [section 26.7.2]

Exposure Category=C [section 26.7.3]

$k_{zt}=1$ [section 26.8.2]

G=0.85 [section 26.9.1]

Enclosed Building [section 26.10.1]

Minimum Wind load design = 16 psf

Steps to determine MWFRS

- 1) Risk Category = II
- 2) Wind Speed (V=120mph)
- 3) Load Parameters
 - a) $K_d = 0.85$
 - b) Exposure Category = C
 - c) $k_{zt} = 1$
 - d) G = 0.85
 - e) Enclosure Classification = Enclosed
- 4) Velocity Pressure Coefficient ($k_z = 2.01 \cdot (z/z_g)^{2/\alpha}$, $z_g = 900$ ft, $\alpha = 9.5$)
- 5) Velocity pressure ($q_z = 0.00256 k_z k_{zt} k_d V^2$)
- 6) External pressure coefficient (C_p)
 - a) Windward = 0.8
 - b) Leeward = -0.5 (L/B ratio 0-1), -0.3 (L/B ratio 2)
- 7) Wind pressure $p = qGC_p$

Example Calculation at z = 600ft

+ = towards building face, - = away from building face

$$q_z = 0.00256(2.01(600/900)^{2/9.5})(1)(0.85)(120)^2 = 57.83 \text{ psf}$$

$$P_{\text{windward}} = (57.83)(0.85)(0.8) = 39.32 \text{ psf}$$

$$P_{\text{leeward1}} = (57.83)(0.85)(-0.5) = -24.58 \text{ psf}$$

$$P_{\text{leeward2}} = (57.83)(0.85)(-0.03) = -14.75 \text{ psf}$$

APPENDIX 2: EARTHQUAKE CALCULATIONS

Constants:

- Occupancy Category III
- R = 8.00
- For NYC:
 - $S_s = 0.364g$
 - $S_1 = 0.070g$
 - $F_a = 1.2$
 - $F_v = 1.7$

Calculated Parameters:

- $S_{MS} = F_a S_s = 0.4368$
- $S_{M1} = F_v S_1 = 0.119$
- $S_{DS} = \frac{2}{3} S_{MS} = 0.2912$
- $S_{D1} = \frac{2}{3} S_{M1} = 0.0793$
- $C_s = S_{DS} / (R/I_e) = 0.0455$
- $C_u = 1.7$, $S_{DS} \leq 0.1$
- $T_a = C_u h_n^x = 0.03 * 0.0731^a * 0.75 = 4.22 * 10^{-3} \text{ sec. (a = R/2)}$

APPENDIX 3: BEAM DESIGN

Example calculation:

Beam design

Given: $M_u = 1832 \text{ k-ft}$
length = 35 ft

We know $c_b = 1.14$ for this beam:

$$M_u/c_b = \frac{1832 \text{ k-ft}}{1.14} = 1607.3 \text{ k-ft}$$

$$L_b = \text{length, in this beam case} = 35 \text{ ft}$$

In AISC Steel Construction Manual Table 3-10, look up an appropriate section for $M_u/c_b = 1607 \text{ k-ft}$

$$L_b = 35 \text{ ft}$$

→ get section W30 x 191.

In Table 3-2 of AISC SCM, section W30 x 191 gives $L_r = 36.8 \text{ ft}$ and $L_p = 12.2 \text{ ft}$.

Since $L_p < L_b < L_r$, we can use the following equation to calculate ϕM_n for the beam:

$$\phi M_n = c_b \left(\phi M_p - (\phi M_p - \phi M_r) \left(\frac{L_b - L_p}{L_r - L_p} \right) \right) \leq \phi M_p$$

Values ϕM_p and ϕM_r can also be determined for a section using Table 3-2. For W30 x 191,

$$\phi M_r = 1580 \text{ k-ft and } \phi M_p = 2530 \text{ k-ft.}$$

This gives $\phi M_n = 1880 \text{ k-ft}$, which is sufficient for $M_u = 1832 \text{ k-ft}$ (efficiency of 97.44%).

This section is used for all other beams in the cantilever except where efficiencies were especially low (< 40%); in that case smaller beam sections were designed with the same process.

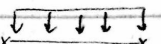
Finding M_u for each beam:

$$(1.2D + 1.6L) \times \frac{k}{1000 \text{ lb}} \times \text{tributary width (ft)} = q \text{ (k/ft)}$$

in psf
linear distributed load applied to beam,
max moment = $\frac{qL^2}{8}$

(cantilever worst beam moment)

$$\text{length} = 35 \text{ ft}$$



This table is for $c_b = 1.0$; need to normalize M_u .

W30 x 191 beam check, cont'd

flange local buckling check:
check AISC 16.1 - Table B4.1b

$$\lambda = \frac{b_f}{t_f} = \frac{b_f/2}{t_f} = 6.35 \text{ (from Table 1-1 of SCM)}$$

$$\lambda_p = 0.38 \sqrt{\frac{E}{F_y}} = 0.38 \sqrt{\frac{29,000 \text{ ksi}}{50 \text{ ksi}}} = 9.15 \text{ (for } F_y = 50 \text{ ksi)}$$

$\lambda < \lambda_p \Rightarrow$ compact

web local buckling check:

$$\lambda = \frac{h}{t_w} = 37.7 \text{ (from Table 1-1 of SCM)}$$

$$\lambda_p = 3.76 \sqrt{\frac{E}{F_y}} = 3.76 \sqrt{\frac{29,000 \text{ ksi}}{50 \text{ ksi}}} = 90.6$$

$\lambda < \lambda_p \Rightarrow$ compact

deflections check:

Simply supported beam:

$$\Delta_e = \frac{5wL^4}{384EI} \text{, where } w \text{ is unfactored live load.}$$

$$= \frac{5(2.7 \text{ k/ft})(35 \text{ ft})^4}{384(29,000 \text{ ksi})(9200 \text{ in}^4)} \times \frac{12^3 \text{ in}^3}{\text{ft}^3}$$

$$= 0.342 \text{ in.}$$

$$\frac{L}{360} = \frac{35 \text{ ft}}{360} \times \frac{12 \text{ in}}{\text{ft}} = 1.17 \text{ in.}$$

$$\Delta_e < \frac{L}{360} \Rightarrow \text{deflections OK.}$$

APPENDIX 4: GIRDER DESIGN

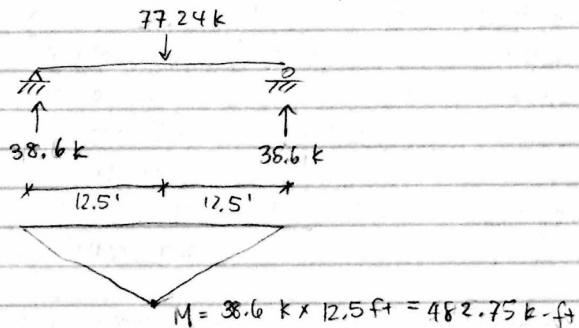
Example calculation:

Girder design

Each girder takes beam reactions as point loads along its length.

Here we give example calculations of a girder in the tower. This girder takes one beam reaction of 77.24 k (reaction + self wt of beam) at its midpoint, with girder length 25 ft.

M_u is calculated to be 482.75 k-ft:



We know that $C_b = 1.67$ for a beam with a point load acting at its midpoint and laterally braced at its midpoint (from steel design class handout)

$$\text{We use } M_u/C_b = \frac{482.75}{1.67} = 289 \text{ k-ft}$$

and $L_b = 12.5 \text{ ft}$ (unbraced length).

Repeat beam design procedure.

Table 3-10 → pick W18x55

Table 3-2 → $L_p < L_b < L_r$, so use same equation for ϕM_n as presented in beam design example calc.

$$\Rightarrow \phi M_n = 491.7 \text{ k-ft}, \text{ sufficient for } M_u = 482.75 \text{ k-ft.}$$

local buckling + deflections calculations done as in beam design example.

APPENDIX 5: COLUMN DESIGN

Example calculation:

Column design

Columns are tabulated for the number of girders and beams they are taking. These are summed with the loads taken by the columns with upper "units" of the building.

This example calculation examines Tower Plan 2's worst column, which takes $P_u = 2815 \text{ k}$ (from the upper unit of Tower Plan 3 and from the 10 floors of Tower Plan 2, all including self wt of beams, girders, and columns designed for Tower Plan 3.)

The column height is 10 ft.

A k of 0.7 is assumed (the frame is slightly stiffer than pinned-pinned conditions.)

Table 4-1 of AISC-SCM was used to look up a W14 section adequate for $kL = 7 \text{ ft}$ with $P_u = 2815 \text{ k}$.

A W14x283 was selected. Values of r_y and A_g were obtained from the table (assumed y-axis buckling controls).

Euler buckling stress F_e was calculated:

$$F_e = \frac{\pi^2 E}{(kL/r)^2} = 101.6 \text{ ksi}$$

Since $F_e > (0.4 F_y = 22 \text{ ksi})$, inelastic buckling was confirmed and the following equation used:

$$F_{cr} = \left[0.658^{F_y/F_e} \right] F_y = 40.7 \text{ ksi} \quad (\text{for } F_y = 50 \text{ ksi})$$

$$\phi P_n = 0.9 F_{cr} A_g = 3051 \text{ k}$$

This is sufficient for $P_u = 2815 \text{ k}$.

This column is used everywhere in Plan 2.

APPENDIX 6: BOLT CALCULATIONS

Connection Calculations: Cantilever Truss Diagonal Member

Required Strength of member = 7900 kips

Determine Single Shear Strength

$$d_b = 1.5 \text{ in (A490 bolts)}$$

$$\text{Minimum Spacing (S)} = (2.6667)d_b = 4.0005 \text{ in}$$

$$\text{Minimum Edge Distance (} l_e \text{)} = (1.125)d_b = 1.6875 \text{ in}$$

$$\mu = 0.5 \text{ (Class B)}$$

$$d_{hg} = 1.5 + 1/8 + 1/16 = 1.6875 \text{ in}$$

$$T_b = 148 \text{ ksi}$$

$$D_u = 1.13$$

$$h_{sc} = 0.85 \text{ (oversized)}$$

$$R_n = \mu D_u h_{sc} T_b N_s \rightarrow (N_s = 1)$$

$$R_n = 71.077 \text{ kips/bolt}$$

$$\phi R_n = 0.85 \times 71.077 = 60.42 \text{ kips/bolt}$$

$$\text{Required strength}/\phi R_n = \# \text{ Bolts Required} \rightarrow 7900/60.42 = 130.76 \text{ bolts}$$

Choose 4 columns of bolts (2 sides)

$$130.76/8 = 16.35 \text{ bolts} \rightarrow \text{Use 17 bolts}$$

Whitmore Section of Gusset Plate

$$WS = \text{gage of bolts} + \tan(30) \times [\text{length of bolt group}] \times 2$$

$$WS = 4.0005 + \tan(30) \times [17 \times 4.005] \times 2 = 77.91 \text{ in}$$

Try 2 in Gusset Plate (A36 steel)

$$A_g = t_g \times WS = 2 \times 64.05 = 155.82 \text{ in}^2$$

Check Tension Yielding

$$\phi = 0.9$$

$$\phi R_n = \phi F_y A_g = 0.9 \times 36 \times 155.82 = 10097.21 \text{ kips} >$$

$$7900 \text{ kips}$$

APPENDIX 6: BOLT CALCULATIONS (continued)

Check Block Shear Rupture in Plate

$$\phi = 0.75$$

Uniform tensile stress $\rightarrow U_{bs} = 1$

$$t_g = 2 \text{ in}$$

$$A_{gv} = t_g * 4 * [(\# \text{bolts} - 1)S + \ell_e] = 525.56 \text{ in}^2$$

$$A_{nv} = A_{gv} - t_g * 4 * [(\# \text{bolts} - 0.5) * d_{hg}] = 302.81 \text{ in}^2$$

$$A_{nt} = t_g * (S - d_{hg}) = 4.63 \text{ in}^2$$

$$\phi R_n = \phi(0.6F_u A_{nv} + U_{bs} F_u A_{nt}) \leq \phi(0.6F_y A_{gv} +$$

$$U_{bs} F_u A_{nt})$$

$$U_{bs} F_u A_{nt} = 268.31 \text{ kips}$$

$$0.6F_y A_{gv} = 11352.18 \text{ kips}$$

$$0.6F_u A_{nv} = 10537.93 \text{ kips}$$

$$\phi R_n = 0.75(10537.93 + 268.3) \leq 0.75(11352.18 +$$

$$268.3)$$

$$= 8715.37 \text{ kips} \leq 8104.68 \text{ kips} \quad \text{O.K.}$$

Bolt bearing on plate

Based on spacing (S)

$$\phi r_n = 157 * 2 = 314 \text{ kips} > 60.42 \text{ kips/bolt} \quad \text{O.K.}$$

Based on edge distance (ℓ_e)

$$\phi r_n = 57.1 * 2 = 114.2 \text{ kips} > 60.42 \text{ kips/bolt} \quad \text{O.K.}$$

Bolt bearing on Flange

Based on spacing (S)

$$\phi r_n = 157 * 2 = 314 \text{ kips} > 60.42 \text{ kips/bolt} \quad \text{O.K.}$$

Based on edge distance (ℓ_e)

$$\phi r_n = 64 * 2 = 128 \text{ kips} > 60.42 \text{ kips/bolt} \quad \text{O.K.}$$

After further calculations, we also decided to weld the connections for the braced core (Fig. A-2). This is due to the high number of bolts required as well as the required spacing between bolts. If a bolt connection were used, the gusset plates between 2 nodes on the same side would overlap as well as the bolts, which would have caused structural problems.

APPENDIX 7: CONNECTION DETAILS

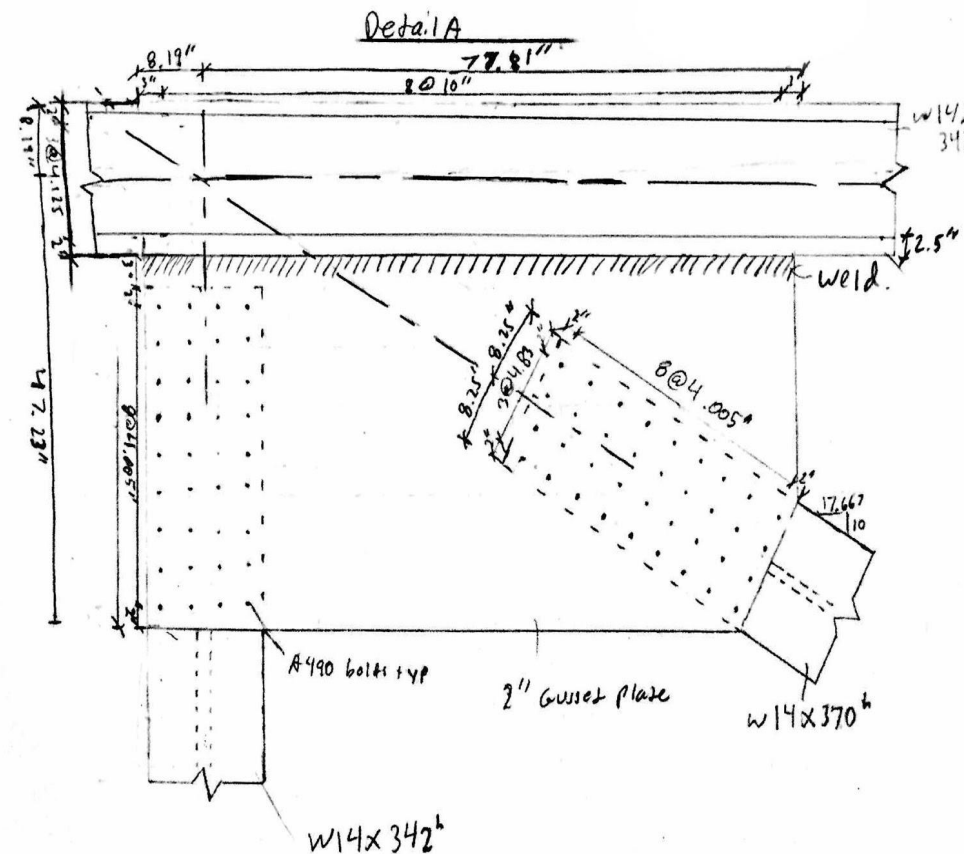


Fig. A-1. Detail of cantilever truss connection (circled in Fig. 23).

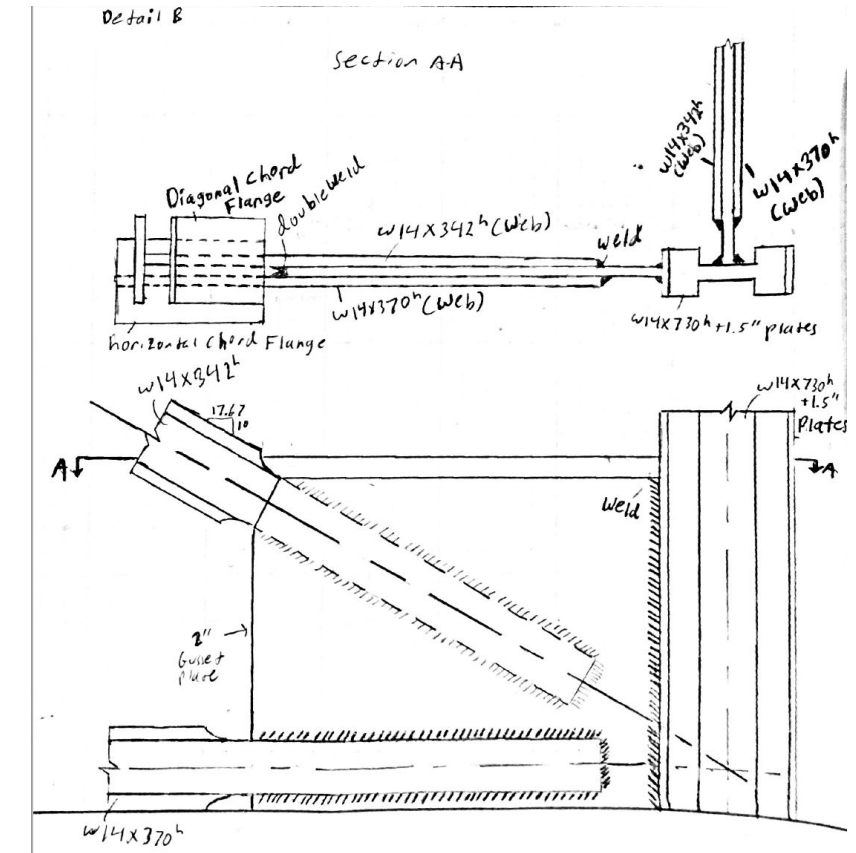


Fig. A-2. Typical braced core connection.

APPENDIX 8: FOUNDATION DESIGN CALCULATIONS

$$1 \quad A_{req} = \frac{\sum P}{q_a}$$

$$2 \quad q_u = \frac{\sum P}{A_{req}^2}$$

$$3 \quad b_0 = 4 * (a + d) \quad a = \text{column width}$$

$$4 \quad V_{u1} = q_u * [b^2 - (\frac{a+d}{12})^2] \quad b = \text{spreading width}$$

$$5 \quad \phi V_c = \phi * 4 * \lambda * \sqrt{f'_c} * b_0 * d$$

$$6 \quad \phi V_c \geq V_{u1} \quad \text{iterate through } d \text{ values.}$$

$$7 \quad V_{u2} = q_u * c * b \quad c = \text{distance to edge, } d \text{ in. from col. abs.}$$

$$8 \quad \phi V_c \geq V_{u2}$$

$$9 \quad M_u = q_u * b * (\frac{g^2}{2}) * 12 \quad g = \text{distance to edge, } 0 \text{ in. from col. abs.}$$

$$10 \quad A_s = \frac{M_u}{0.9 * 60 * (d - 1)}$$

