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STRUCTURAL OPTION

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AE 481 W

TECHNICAL REPORT ONE
STRUCTURAL CONCEPTS / EXISTING CONDITIONS

INTRODUCTION

This first technical report presents a detailed description and preliminary analysis of the existing structural system found within Memorial Sloan-Kettering Cancer Center. Located in Somerset County, New Jersey, this four story health-care facility will open its doors in the summer of 2006 to serve as one of the premiere cancer treatment centers in nation. A combination of steel, concrete, and masonry, MSK is designed to hold 38 exam rooms, 27 offices, 23 chemotherapy bays, a Laboratory, Pharmacy, and radiotherapy treatment area. From an exterior perspective, the building's brick and stone façade accents the surrounding mountains. Curved exterior walls with large windows allow for an interior layout filled with dynamic hallways, maximizing natural lighting, and scenic views. Structural steel framing creates beam spans between 30' and 45' long, opening up spaces where needed.

The structure below grade consists of foundation walls and columns exclusively made of reinforced concrete. This concrete system ties into the steel framing at grade which continues throughout the rest of the building. The floor systems consist of one-way slab on the first floor and slab on composite metal deck for the remaining four stories. The building envelope is made up of a brick and glass curtain wall which is attached directly to the structural components of Memorial Sloan- Kettering. Steel cross bracing frames are positioned in both directions to take lateral loads.

This report will discuss and analyze the structural system of Memorial Sloan-Kettering through a combination of design analysis, required loading, and code criteria directly related to the structural design. An in-depth description of the foundation, steel framing, lateral bracing, and all floor systems are provided. To maximize clarification, a number of diagrams are referenced and can be found in Appendix E. Design analysis of both wind and seismic loads on the building were calculated with consideration of all proper modifiers. In addition, structural spot checks have been performed on random beams, girders, and columns. Complete sets of wind and seismic load calculations can be found in Appendix D and handwritten calculations referring to the spot checks made in this report are in Appendix C. Sketches of floor framing plans can be seen by referring to Appendix A.

CODES AND CODE REQUIREMENTS

The following codes were used in the structural design of Memorial Sloan-Kettering:

National Code: *International Building Code 2000 – New Jersey Edition*

Design Codes:

- *American Society of Civil Engineers* (ASCE 7)
- *American Concrete Institute* (ACI 318-02)
- *American Institute of Steel Construction* (AISC - 3rd Edition)
- *ASTM Standards – Properties of Building Materials*

LOADS AND CALCULATIONS

Dead Loads:

1st Floor:

75 psf	concrete slab
15 psf	superimposed
45 psf	concrete beam
50 psf	concrete girder
185 psf	

2nd through 4th Floor:

56 psf	concrete slab
2 psf	metal deck
15 psf	steel framing
15 psf	superimposed
88 psf	

Roof:

44 psf	concrete slab
2 psf	metal deck
15 psf	steel framing
20 psf	MEP equipment
81 psf	

Live Loads:

100 psf (Table 4-*1 ASCE 7-02*)

Snow:

Ground Snow Load: 30 psf
Flat Roof Snow Load: 23 psf

*** **Please refer to Appendix D for all load calculations*****

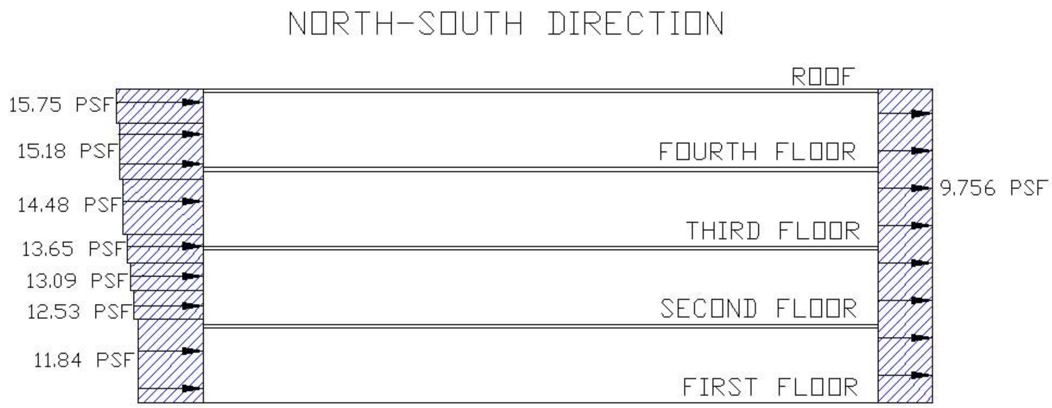
LATERAL LOADS

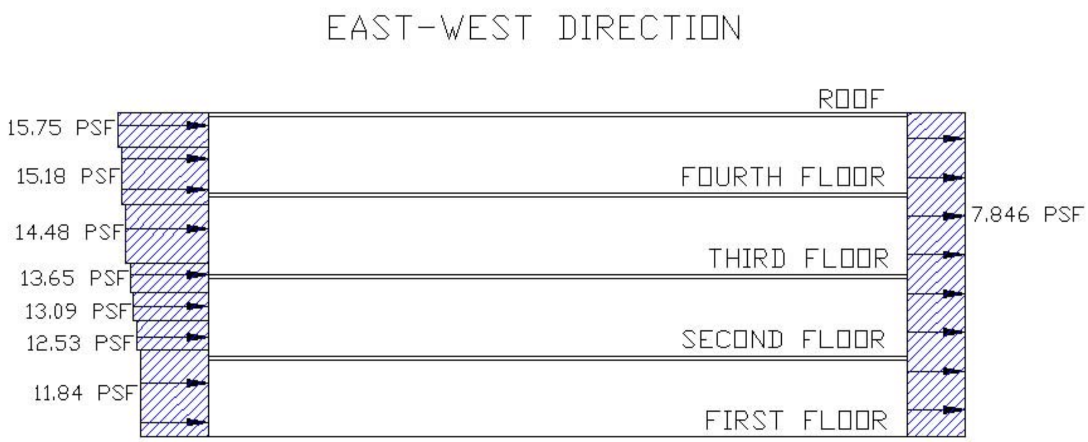
Following the guidelines set by IBC 2000, the lateral loads for Memorial Sloan-Kettering have been computed by the methods provided in ASCE 7-02. Wind forces were found using the Analytical Approach (Method 2) located in Chapter 6 of the design code. Similarly, seismic forces were found using the Equivalent Lateral Force Method, which is given in Chapter 9. Summarized results of both methods are provided below. Please refer to Appendix D for the complete version.

Wind: Method 2 (ASCE 7-02)

Velocity Pressure Envelope						
	Windward		Leeward		Max psf	
Z(ft)	N-S	E-W	N-S	E-W	N-S	E-W
0-15	11.84	11.84	-9.77	-7.84	21.60	19.68
15-20	12.53	12.53	-9.77	-7.84	22.30	20.38
20-25	13.09	13.09	-9.77	-7.84	22.86	20.93
25-30	13.65	13.65	-9.77	-7.84	23.41	21.49
30-40	14.48	14.48	-9.77	-7.84	24.25	22.33
40-50	15.18	15.18	-9.77	-7.84	24.95	23.02
50-60	15.74	15.74	-9.77	-7.84	25.50	23.58
58	15.63	15.63	-9.77	-7.84	25.39	23.47

Shear Summary		
(kips)	N-S	E-W
Shear @ Roof	34	20.86
Shear @4	67.6	40.5
Shear @3	64.47	38.43
Shear @2	59.97	35.5
Shear @1	0	0
Base Shear	226.04	135.301
Overturning Moment	7651.80	4601





Seismic: Equivalent Lateral Force Method (ASCE 7-02)

North - South Direction							
Level, x	w_x	h_x	$w_x h_x^k$	C_{VX}	F_X	V_x	M_x
	kips	feet			kips	kips	ft-kips
Roof	2346	58	115,763	0.368	123.4		7155.2
4	2620	43.5	98,063	0.312	104.5	123.4	4545.9
3	2620	29	66,440	0.211	70.8	227.9	2053.3
2	2620	14.5	34,150	0.109	36.4	298.7	527.7
1						**335.1**	
Σ	10206		314416	1	**335.1**		**14282.1**

East - West Direction							
Level, x	w_x	h_x	$w_x h_x^k$	C_{VX}	F_X	V_x	M_x
	kips	feet			kips	kips	ft-kips
Roof	2346	58	115,763	0.368	123.4		7155.2
4	2620	43.5	98,063	0.312	104.5	123.4	4545.9
3	2620	29	66,440	0.211	70.8	227.9	2053.3
2	2620	14.5	34,150	0.109	36.4	298.7	527.7
1						**335.1**	
Σ	10206		314416	1	**335.1**		**14282.1**

DESCRIPTION OF STRUCTURAL SYSTEM

Basement /Foundation

The basement floor of Memorial Sloan-Kettering is located 360 ft. above sea-level on the east side of the building, and gradually rises to 362 ft. above sea level on the west side of the building. The floor consists of 5" slab on grade concrete reinforced with 6x6-W4.0xW4.0 welded wire fabric. All slab on grade concrete in MSK has a minimum compressive strength of 3000 psi. The steel reinforcement for these slabs is to be ASTM A615 Grade 60. All welded wire fabric shall be ASTM A185. During construction, this floor system is divided in sections and each section is poured modularly. These pours are separated by construction/control joints. The welded wire fabric discontinues at each joint.

Concrete columns provide the primary structural support below grade. These columns are 2' by 2' in dimension and are typically spaced in 30' x 30' bays from center line to center line. This spacing fluctuates for spans close to exterior foundation walls. Each of these columns in has a typical vertical reinforcement of eight #8 bars and horizontal reinforcement of #4 ties spaced 12" on center (See diagram X-3 in Appendix). Each column rests on a 6' by 6' footing which typically descends four feet below grade. These footings are reinforced with twelve #6 bars spanning in both directions. All footings have a compressive strength of 4000 psi and are reinforced with A615, Grade 60 steel.

The exterior foundation walls of MSK are comprised of 1'4" thick, reinforced concrete and have a compressive strength of 4000 psi. These walls contain 2' wide concrete pilasters, typically spaced 30' on center, these extend 1' beyond the face of the foundation wall into the interior space. The footings that support the foundation walls are 3'-2" wide, extending 10" beyond the wall on each side. The concrete pilasters footings are similar to those supporting the columns, dimensioned at 6' by 6'. All foundation wall footings descend three beneath the basement's concrete slab on grade. The footings located on the northern side of Memorial Sloan-Kettering bear on undisturbed decomposed rock having a minimum allowable bearing pressure of 10 KSF. The footings located on the southern side of the building are to bear on undisturbed competent rock having a minimum allowable bearing pressure of 20 KSF.

Memorial Sloan-Kettering's front entrance, located on the building's west side, is covered by a cantilevered canopy. Because of this, the canopy piers descend along the exterior side of the foundation walls to exactly the same depth. Each canopy pier is 1'-2" wide by 7'-3" long with footings extending 6" beyond on each side.

One noticeable feature pertaining to the basement's structural system is where building ends above grade on the north side. In this area, two concrete columns are placed side by side. Because of this, instead of individual footings per column, there is one large footing approximately 8' wide by 63' long. This footing descends almost 6' below grade and has seven concrete columns and two shear walls tying into it.

One-Way Slab System

The first floor of Memorial Sloan-Kettering Cancer Center is constructed as a one-way concrete slab system that is structurally supported by the foundation walls and concrete columns below. The 6" concrete slab lies on top of concrete beams spanning in the E – W direction and concrete girders spanning in the N – S direction.

The concrete beams have a typical tributary width of 10' and span 30' between girders. These girders also span 30' from column to column. A typical concrete beam size is 18" wide by 24" deep and is reinforced by four #8 bars on top and four #7 bars on the bottom. These bars are held together horizontally with #4 ties spaced 12" on center and contain #4 stirrups spaced 9" on center.

A typical concrete girder size is 24" wide by 30" deep and has top reinforcement of eight #9 bars and bottom reinforcement of six #8 bars. This reinforcement is held together horizontally with #4 ties spaced 12" on center and contain #4 stirrups spaced 6" on center.

Slab on Deck Floor System

The second, third, and fourth floors of Memorial Sloan-Kettering all share the same composite floor system. This system consists of 4 ½" normal weight concrete slab poured on 2" 20-gauge galvanized metal decking. The concrete slab is reinforced with 6x6-W2.9 x W2.9 welded wire fabric. The metal floor deck spans in the E – W direction and must be continuous over a minimum of two or more spans. In the event where spanning less then that distance is unavoidable, the floor must be shored at midspan until the concrete slab is fully cured.

The metal deck connects into the wide flange steel beams through equally spaced ¾" diameter by 4" long headed shear studs welded into the center of the flange. The roof of this building is made of 3 1/2" thick normal weight concrete slab on 2" 20-gauge galvanized composite metal decking and follows the same composite criteria as the floor below.

Structural Framing

The structural steel skeleton in MSK begins at the first floor level and continues for the remainder of the building. Because each steel column sits directly on top of a concrete column, the typical bay size remains at 30'x 30' throughout the first floor. However, beginning on the second floor, a number of columns near the south end of the building are removed in order to create more of an open floor plan. This causes some bays to span 30' x 45' in the upper level floors. A number of bays are also reduced in size near the exterior walls of the building due to Memorial Sloan Kettering's curved exterior façade. The structural steel grade for all framing members is ASTM A992 Grade 50.

The steel columns vary in size throughout the building according to their location and purpose. These columns remain constant in size between the first floor and fourth floor. Typical exterior columns vary anywhere from a W12 x 53 found on the west elevation of the building to a W12 x 106 found on the northeast corner. The steel columns which support the cantilevered canopy are W10 x 33's and only span one story. As the columns get closer to the building's central axis, they are required to support tributary area and thus are larger is size. A typical interior column ranges between W12 x 87 and W12 x 96.

These steel columns connect into the concrete columns below through ASTM A572, Grade 50 steel base plates. A typical base plate used for these connections is 18"x 18" and 1-½" thick. These plates are kept in place by four ¾" A449 anchor bolts embedded 2' into the concrete column (See diagram X-4 in Appendix E)

The second, third, and fourth floors of MSK make use of steel beams and girders to support their floor systems. The second floor is the last floor to maintain the typical bays sizes and because of this, has the most consistent steel sizes throughout the floor. A typical interior beam is a W16 x 26 while a typical interior girder is a W24 x 96. For smaller bays near the exterior walls, beam sizes fall to a W12 x 16. The exterior girder sizes do not show much consistency, ranging anywhere from a W18 x 35 to a W30 x 108. The third floor and fourth floor both have the same layout. Where the interior spans continue from the second floor, the structural design is maintained with W16 x 26 beams connecting into W24 x 96 girders. For those spans which became 30' x 45', the beams become W24 x 62 and girders become W30 x 90. The roof of the building follows the same framing as the fourth floor, only with slighting smaller beams and girders that support the 30' x 45' bays.

Lateral System

The lateral system designed for Memorial Sloan-Kettering Cancer Center consists of shear walls in the basement floor which tie into steel cross-bracing for all levels above grade. Shear walls are located on the north and south sides of Memorial Sloan-Kettering surrounding the basement's concrete stairwells and framing into supporting columns. These 12" thick shear walls span in both the N-S and E-W directions and are approximately 14' long. Another four shear walls are located at the center of the basement floor plan. Two of these walls span in the N-S direction and two span in the E-W direction. Again, all four walls remain constant at 14' long. Each shear wall is reinforced vertically with #5 bars at 12" on center for both faces of the wall. These two faces are tied together with #4 ties spaced 12" on center. Similarly, the horizontal reinforcement on each wall face is made up of #5 bars at 12" on center (See diagram X-1 in Appendix E). The columns supporting these shear walls have sixteen #9 bars of vertical reinforcement, about twice as much as that found in a typical column.

Once above grade, the lateral system changes from concrete shear walls to steel cross-bracing. This cross bracing consists of two HSS 6 x 6 x ½'s which are connected together at mid span by a ¾" gusset plate. Each brace spans approximately at a 45 degree angle for one story, where it connects into the structural columns running vertically on both sides of the lateral system.

Linear Accelerator

The section of MSK which contains the linear accelerators follows a very unique design. A linear accelerator is a device used to destroy cancer cells by delivering a uniform dose of high-energy radiation to the patient. Because of the dangerous side effects that could result if these x-rays were to penetrate the exterior walls, this area of the building must be impenetrable.

The reinforced concrete walls surrounding these machines are between 4' and 8' thick, ensuring that even a slight crack will not allow the radiation to escape. Each wall is reinforced with ASTM A615 Grade 60 steel. A typical design is #7 bars spaced 8" apart for each face for horizontal reinforcement, and #7 bars space 12" apart on each face for vertical reinforcement (See Diagram X-2 in Appendix E). Also, the vertical reinforcement in the wall must extend to within 3" from the top of roof slab. To make certain that no horizontal or vertical joints exist in this area, all exterior and interior walls must be place in one continuous, monolithic pour.

The roof slab above the linear accelerators is also 4' thick, but is placed in two separate concrete pours instead of one. The first two pour add two feet of concrete to the roof slab, and in between pours the slab must be roughened as deep as ½" to create a proper bond between the top and bottom slabs. Reinforcement for the roof slab is #7 bars spaced 12" on center for both the top and bottom faces. Also, the shoring for the roof must be kept in place until both concrete slabs reach the compressive strength of 4000 psi.

STRUCTURAL ANALYSIS

Structural spot checks were performed on random beams, girders, and columns in order to determine how accurate the design assumptions used to develop the loads for this building were. The results from each spot check could then be compared to the existing structural design to evaluate the effectiveness of those vales. Overall, the member sizes designed with the design assumptions came relatively close to that in the building. This reveals that the assumptions made in this report which deal with the structural analysis of Memorial Sloan-Kettering must be somewhat close to the actual design values.

First Floor System

Both a beam and girder were checked in this system and compared with the actual beam and girder characteristics. A factored load of 1.2D + 1.6L was applied to the deal and live load, respectively. The maximum positive and negative moments for this span were taken from ACI 318 with the assumption that the beam is on two interior supports. After calculating for the beam, it was apparent that actual beam was stronger then necessary. The same held true when computing the girder for this typical interior bay. Because both the beam and girder were slightly larger then necessary, it is possible that a small amount of dead load was neglected.

Remaining Floors

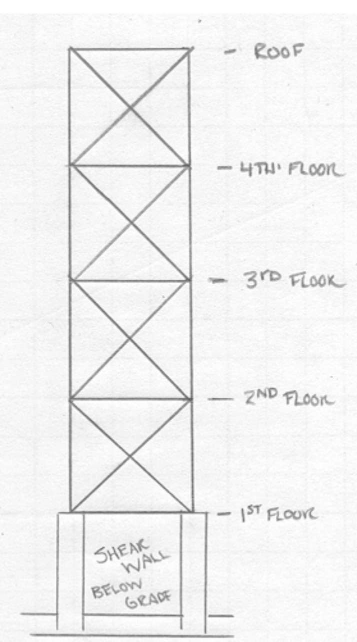
Another spot check was done on a beam and girder designed to support the slab on metal deck. Because this floor is a composite system, shear also had to be checked to see whether or not the member would receive the full moment. With the assistance of Table 5-14 in the LRFD Manual, it was determined that these beams were calculated with the same assumptions as those used in this report.

Column:

A random interior column was chosen from the second floor. Gravity loads were calculated for all the floors above and collected down to the second floor column. A live load reduction factor was computed and applied for this check. After going into the LRFD, a 12x65 was chosen as a suitable column size. The actual column designed for is a W21x94, but there could also be number of factors that were never taking into consideration.

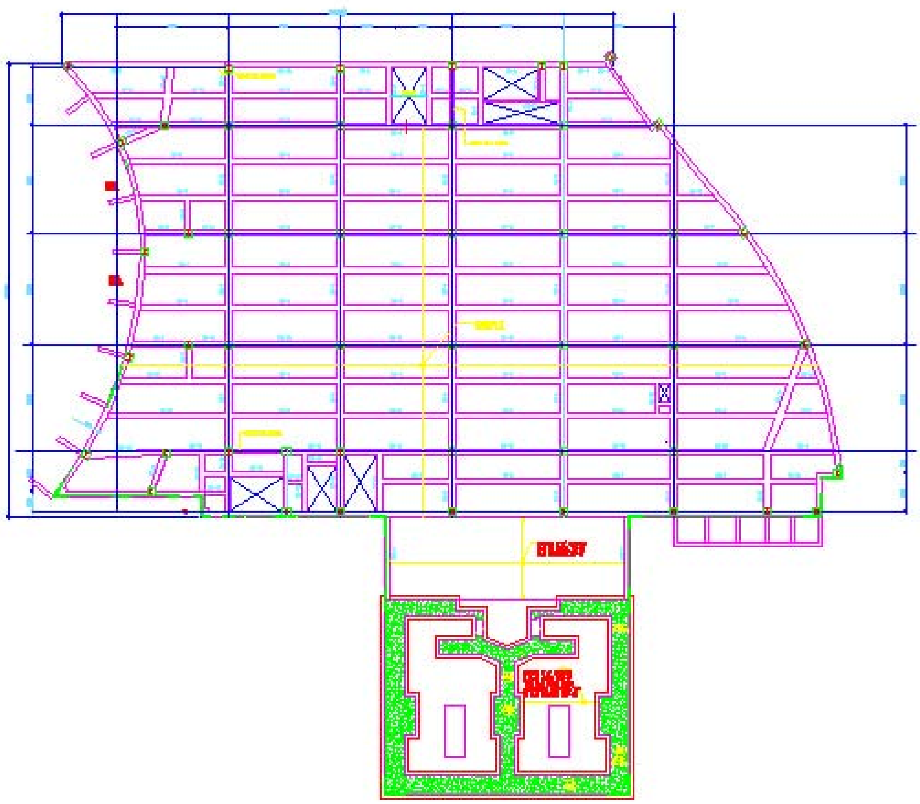
Lateral Bracing Check:

The lateral bracing system was also chosen to perform a spot check with. After calculating the lateral loads and determining the total base shears for both wind and seismic, it was apparent that seismic loads created a larger base shear in my building with 328 kips. The lateral bracing was checked between the ground and second floor to determine whether or not it was large enough to withstand that shear force. A HSS 6x6x3/8 proved to work. The lateral bracing design specifies HSS 6x6x1/2" so this calculation was very close.



ELEVATION OF LATERAL BRACING THROUGHOUT BUILDING

APPENDIX A: Typical Floor Plan



FIRST FLOOR FRAMING PLAN

APPENDIX B: Load Calculations

LOAD CALCULATIONS

DEAD LOADS

1st Floor

$(6''/12'')(150\ PCF) = 75\ PSF$ SLAB

$= 15\ PSF$ SUPER IMPOSED

$(18''/12)(24''/12)(150)(1/10') = 45\ PSF$ → BEAM → (450 PLF @ 10' O.C)

$(24/12)(30/12)(150)(1/30') = 50\ PSF$ → GIRDER → (750 PLF @ 30' O.C; FOR BOTH DIRECTIONS)

185 PSF

2ND FLOOR – 4TH FLOOR

$(4.5''/12'')(150\ PCF) = 56.25\ PSF$ SLAB

= 2 PSF METAL DECK

= 15 PSF STEEL FRAMING

= 15 PSF SUPERIMPOSED

88.25 PSF

ROOF

$(3.5/12)(150) = 43.8\ PSF$ CONCRETE

2 PSF METAL DECK

15 PSF STEEL FRAMING

20 PSF MECHANICAL

80.75 PSF

LIVE LOADS

ALL FLOORS → 100 PSF → TABLE 4-1 ASCE 7-02

SNOW LOADS

GROUND SNOW LOAD $P_g = 30\ PSF$ → TABLE

FLAT ROOF SNOW LOADS

$P_f = 0.7 C_e C_t I P_g$ ∴ $C_E = 0.9$ TABLE 7-2

$C_T = 1.0$ TABLE 7-3

$I = 1.2$ TABLE 7-4

$P_f = (.7)(0.9)(1.0)(1.2)(30)$

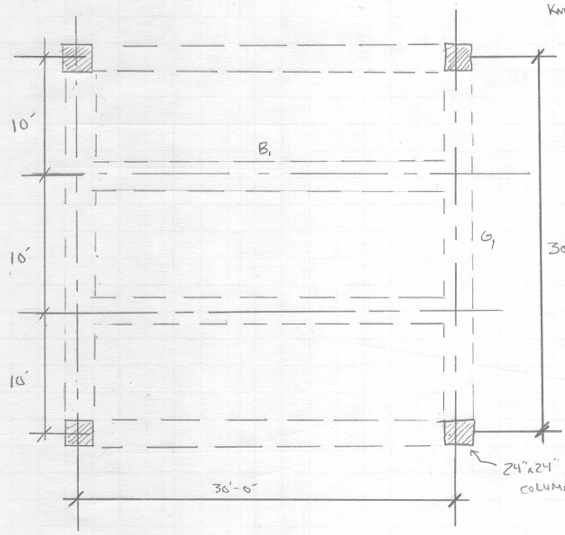
$P_f = 22.68\ psf$

APPENDIX C: Gravity Load Checks

First Floor Check:

1IT FLOOR SLAB 1

ONE WAY SLAB SYSTEM → 1ST FLOOR



KNOWN:

BEAM SIZE: 18"W x 24"D

REINFORCEMENT: 4#8 B = 4-#7"

BEAM SPAN: 30'

BEAM SPACING: 10' on CENTER

GIRDER SIZE: 24"W x 30"D

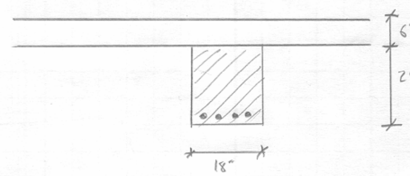
REINFORCEMENT: T → 4#9 6#8

SPAN: 30'

6" CONCRETE SLAB

* ASSUME SUPERIMPOSED OF 15 PSF

→ DEAD LOAD (BEAM)



BEAM = $(18/12)(24/12)(150) = .45$ KLF

SLAB = $(6/12)(150)(10) = 0.75$ KLF

SUPERIMPOSED = $(15\text{ PSF})(10') = .15$ KLF

LL = $(100)(10') = 1$ KLF

$1.2(1.35) + 1.6(1) = 3.22$ KLF

$b_{eff} \overset{\text{less of}}{=} \begin{cases} (8)(6)(2) + 18 = 114" \\ \text{or} \\ (30)(12)/4 = 90" \leftarrow G \end{cases}$

FIND MOMENTS

→ ACI 381 → CODE 8.3.3 (INTERIOR SPANS) $L_n = 30 - 2' = 28'$

MAX POSITIVE $\dfrac{w_u L_n^2}{16} = \dfrac{(3.22)(28')^2}{16} = 158$ k·ft

MAX NEG $\dfrac{w_u \cdot L_n^2}{11} = \dfrac{(3.22)(28')^2}{11} = 229.5$ k·ft

→ LOOK @ MIDSPAN

* MAX POSITIVE $M_n = 158/.9 = 175.6$

$d = (24 - 1.5 - .5 - .5) = 21.5"$

$A_{s,REQ} = \dfrac{175.6\,(12")}{(60)(.9 \cdot 21.5)} = 1.81" < 2.40"$ REINFORCEMENT WORKS

$a = \dfrac{(2.4)(60)}{.85(4)(90)} = .47"$

– DESIGN USES 4#7 BOTTOM REINFORCEMENT

$A_s = 2.40"$

$\phi M_n = (0.90)(2.4)(60)(21.5 - .47/2) = 2755\ \text{k}\cdot\text{in}/12 = 229 > 158 \therefore$ WORKS

LOOK NEGATIVE MOMENT AT SUPPORTS

229.5 K·ft $d \cong 24-1.5-.5-.5 = 21.5''$

$M_n = \frac{229.5}{.9} = 255\ K$ $A_s = \frac{(255)(12'')}{(60)(.9 \cdot 21.5)} = 2.63''$

BEAM DESIGNED W/ 4#8 @ SUPPORTS

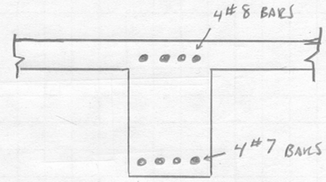
$A_s = 3.16''$

$a = \frac{(3.16)(60)}{(.85)(4)(18)} = 3.09$

$\phi M_n = (.9)(3.16)(60)(21.5 - \frac{3.09}{2}) = \frac{3405}{12} = 283 > 229.5$ ∴ WORKS

→ BEAM IS PROPERLY REINFORCED TO WITHSTAND MAX POSITIVE MOMENT @ MIDSPAN AND MAX NEGATIVE MOMENT AT THE SUPPORT.

BEAM REINFORCEMENT



→ GIRDER → USING SAME BAY AS SHOWN BEFORE

→ DEAD LOAD
- → SLAB $(6/12'')(150)(30) = 2250$ PLF
- → SUPERIMPOSED $(15\ PSF)(30') = 450$ PLF
- → BEAM $= (18/12)(24/12)(150)(30) = 13500/10' = 1125$ PLF
- GIRDER $= (24/12)(30/12)(150) = 750$ PLF

$DL_{TOTAL} = 4.58$ KLF

→ LIVE LOAD

$= (100)(30) = 3$ KLF $\omega = 1.2(4.58) + 1.6(3) = 10.296$ KLF

MOMENTS

$M_{AX}+ = \frac{\omega_u L_n^2}{16} = \frac{(10.296)(28^2)}{16} = 504\ K'$ $M_{AX}- = \frac{(10.296)(28)^2}{11} = 733\ K'$

→ LOOK @ MAX POSITIVE

$d = (30-1.5-.5-.5) = 27.5''$ $A_{s_{REQ}} = \frac{(560'k)(12'')}{(60)(.9 \cdot 27.5)} = 4.53''$

$a = \frac{(4.74)(60)}{(.85)(4)(90)} = 0.929$ GIRDER USES 6#8 $A_s = 4.74''$

$\phi M_n = (.9)(4.74)(60)(27.5 - \frac{.929}{2}) = \frac{6920}{12} = 576'k > 504'k$ ✓

– CHECK MAX NEGATIVE MOMENT

$M_u = 733$ $\quad M_n = 733/.9 = 814.44$ $\quad A_s = \frac{(814.44)(12)}{(60)(.9)(27.5)} = 6.58$

TRY 8 #9 BAR $A_s = 8$

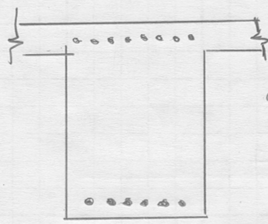
$$a = \frac{(8)(60)}{(.85)(4)(90)} = 1.56$$

$$\phi M_n = (.9)(8)(60)(27.5 - 1.56/2) = 11543.04/12 = 981\ 'k$$

$$961.92\ 'k > 733\ 'k$$

→ REINFORCEMENT WORKS

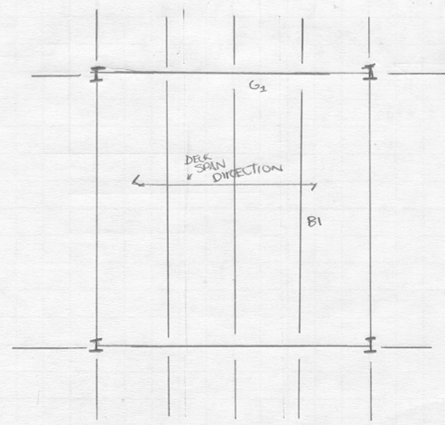
→ THE GIRDER IS EASILY ABLE TO SUPPORT THE MOMENTS IT WILL EXPERIENCE WITH THE DESIGNED REINFORCEMENT. IT ACTUALLY SEEMS THAT THE TOP REINFORCEMENT COULD BE REDUCED BUT PERHAPS THE STEEL IS POSITIONED IN TWO ROWS INSTEAD OF ONE. THAT WOULD DECREASE d and cause a slightly smaller moment.



GIRDER REINFORCEMENT

Slab on Deck Check:

Composite slab on metal deck



B1

Beam → W16x26

→ 30' span

7.5' tributary area

<15> 3/4" ϕ diameter shear studs

G_1 → W24 x 94

→ 30' span

30' tributary area

<32> 3/4" ϕ shear studs

Slab

4 1/2" normal weight galvanized concrete on 2" metal deck

Check gravity loads

DL

Concrete = (150 pcf)(4.5/12) = 56.25 psf

Metal deck = 2 psf

Superimposed 15 psf

73.25

* Beam wt already calc'd into table

LL = 100 psf (entire building)

$w_u = (1.2)(73.25\ psf) + 1.6(100) = 247.9\ psf$

b_{eff}

$(30 \cdot 12)/4 = 90"$

or

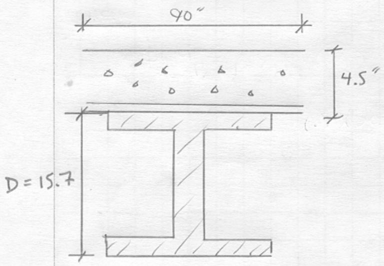
$(7.5)(4) = 90"$

$b_{eff} = 90"$

$W_u = (247.9)(7.5') = 1.86\ k/ft$

$M_u = \frac{(1.86)(30')^2}{8} = 209.3\ 'k$

D = 15.7



→ Assume W16x26

$C = (.85)(4.5)(90)(4) = 1377\ ksi$

$T_s = (7.68)(50) = 384\ ksi$ Steel controls

$a = \frac{384}{(.85)(4.5")(90)} = 1.115"$ → PNA

$Y_1 = 1.115$

$Y_2 = 4.5 - 1.115/2 = 3.94$

→ Look @ Table 5-14 on pg 5-143

@ $Y_2 = 3.5$ (conservative)

Y_1 = @ #6

$M_n = 243\ ft \cdot k > 209.25\ ft \cdot k$

works

∴ check shear

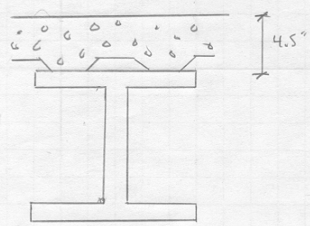
CHECK SHEAR

ASSUME (1) SHEAR STUD $Q_n = 26.1$ kips → TABLE 5-13

TABLE 5-14 $\sum Q_n = 145 / 26.1 \text{ kips} = 5.55 \therefore 6 \cdot 2 = 12$ STUDS REQUIRED FOR FULL MOMENT DEVELOPMENT ✓

∴ a W16x26 works in this design

→ GIRDER → use same bay as shown before



W24x84

$d = 24.3$

$DL = 73.25$ psf (slab and deck)

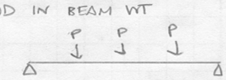
$LL = 100$ psf

$w_u = (1.2)(73.25) + (1.6)(100) = 247.9\text{psf}$

$W_u = (247.9 \text{ psf})(30') = 7.44$ klf

$$M_u = \frac{(7.44)(30')^2}{8} = 837\ 'K$$

ADD IN BEAM WT



$P = (26 \text{ plf})(30') = .78$ K

TABLE 5-16 in LRFD

Max Moment $= (.5)(.78^K)(30') = 11.7$ K'

$M = 837 + 11.7 = 848.7\ 'K$

→ FIND M_n

$b_{eff} = (30 \cdot 12)/4 = 90"$

$C_c = (.85)(4.5)(90)(4) = 1377$ ksi

$T_s = (27.7)(50) = 1385$ ksi

P.N.A. is in steel

$$a = \frac{1385}{(.85)(4)(90)} = 4.52$$

$T = C$

$T_s = C_c + C_s$

$C_s = 8$

* PNA IS in FLANGE

$8/2 = (9.07)(50)(x)$

$x = .001 \therefore$ in Flange

3

→ LOOK @ TABLE 5-14
ON PG 5-140

$M_n = 1687\,'k > 788\,'k$ works

Check Shear

$Q_n = 26.1\,k$

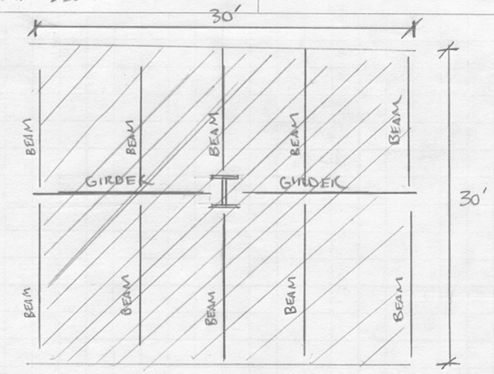
$32 \text{ studs}/2 = 16$

$\Sigma Q_n = 16(26.1) = 417.6^k > 394^k$ ∴ full moment developed

Column Check:

PRELIMINARY
COLUMN DESIGN (2ND FLOOR)

K R



TRIBUTARY AREA
30' x 30'

W12 x 96 COLUMN

INTERIOR COLUMN

2ND FLOOR

LOADS

LIVE LOADS : 100 PSF

DEAD LOADS : 56.25 PSF CONCRETE
2 PSF METAL DECK
15 PSF SUPERIMPOSED
15 PSF STEEL FRAMING (ASSUMED
88.25 PSF

3RD AND 4TH FLOOR ABOVE

SAME FLOOR SYSTEM AS 2ND FLOOR

LL = 100 PSF

DL = 83.25 PSF

ROOF =

DL = 43.8 PSF CONCRETE
2 PSF METAL DECK
15 PSF STEEL FRAMING
20 PSF MEP
80.75 PSF

→ TRIBUTARY AREAS

$A_T = (30')(30') = 900\ ft^2$

for COLUMN ON 2ND LEVEL (3 FLOORS ABOVE)

$(900)(3) = 2700$

$K_{LL} = 4$ for interior column

$A_I = (2700)(4) = 10800$

$L = L_o\left(.25 + \frac{15}{\sqrt{10800}}\right) = 0.39 < .40 \rightarrow .4$ controls

$L = (2700)(100)(.4) = 108^k$

$D_L = (2)(900)(88.25) + (900)(80.75) = 231.5^k$

$P_u = (1.2)(231.5) + (1.6)(108) = 450.6^k$

→ Go INTO LRFD → TABLE 4-2

ASSUME $KL = 15'$

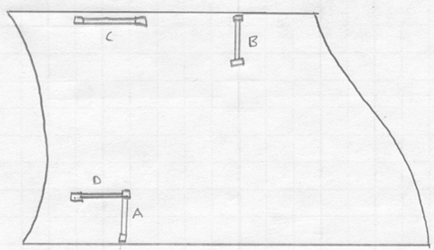
→ TRY A W12

→ ANY ABOVE A W12 x 65 works ∴ $\phi P_n = 626^k$

→ DESIGN USES A W12 x 96 COLUMN which is larger then necessary according to these calculations, but there are a few logical reasons for why a larger column is needed. FOR one, the building columns do not change from 1st to 4th floor, so columns were probably designed for 1st floor instead of the 2ND. Also, I used a LL reduction of .4, which significantly reduces the LL value. Perhaps a smaller LL reduction factor was used which would result in a larger ϕP_n. A larger DL could also have been used, or perhaps the column was chosen to be reoccurring. FOR THESE REASONS, I FEEL THAT A W12 x 94 was an adequate column size.

Lateral Bracing Check:

→ CHECK LATERAL BRACING FOR STEEL CROSS-BRACING ABOVE GRADE



SHEAR WALLS:

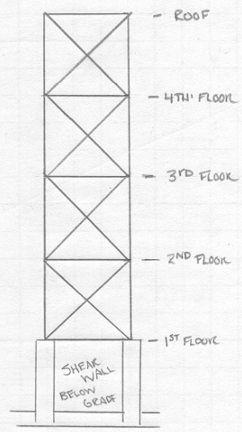
A: 14' long × 1' thick

B: 14' long × 1' thick

C: 13'-8" long × 1' thick

D: 13'-8" long × 1' thick

TYPICAL FLOOR PLANS SHOWING LATERAL BRACING AREAS

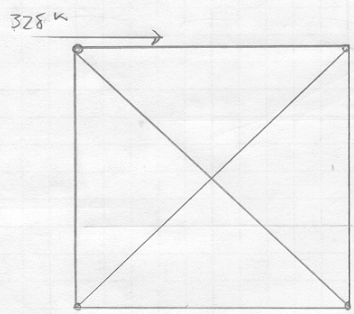


ELEVATION OF LATERAL BRACING THROUGHOUT BUILDING

→ Analyze Shear Wall A

→ Shear Force experienced from SEISMIC LOADS in N-S DIRECTION

$V_{x_{max}} = 328^k$ @ 1^{ST} FLOOR



$A_g = \frac{328^k}{50\,ksi} = 6.56\ in^2$ $\therefore$ A_g must be $\geq 6.56\ in^2$

TRY AN HSS $6 \times 6 \times 3/8"$ $A_g = 7.58\ in^2$

ACTUAL DESIGN USES HSS $6 \times 6 \times 1/2"$

DESIGNS ARE VERY CLOSE WHICH LEADS ME TO BELIEVE MY SEISMIC ANALYSIS IS CONSIST WITH THAT DESIGNED FOR

APPENDIX D: Lateral Load Check

Wind:

(1)

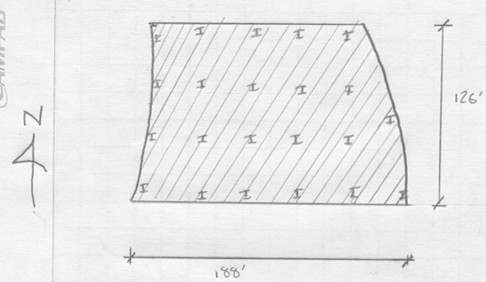
- Wind Analysis

- IBC 2006 refers to ASCE 7 for wind load calculations
  - → TRY USING METHOD TWO - ANALYTICAL PROCEDURE

ASSUMPTIONS:

- Rigid Structure
- HEIGHT = 58′ < 60′ ∴ QUALIFIES AS LOW-RISE BUILDING
- REGULAR SHAPED BUILDING

→ TYPICAL FLOOR PLAN SIZE



$B = 126'$

$h = 58'$

$L = 188'$

Exposure Catagory = C

$V = 90$ MPH

$K_d = 0.85$ (MAIN WIND FORCE RESISTING COLUMN)

$I = 1.15$ where $V = 85-100$ MPH

$K_z =$ TABLE 6.3 for EXPOSURE C

$K_{zt} = 1.0$

$G = 0.858821$

Enclosure CLASSIFICATION = ENCLOSED

$C_p =$ ∴ LOW RISE BLDG. FIG. 6-10

$q_h =$ → computed in TABLE

FIND GUST EFFECT FACTOR

$$G = 0.925\left(\frac{(1+1.7g_q I_{\bar{z}} Q)}{1+1.7g_v I_{\bar{z}}}\right) \qquad I_{\bar{z}} = (.2)\left(\frac{33}{34.8}\right)^{1/6} = 0.1982 \qquad L_{\bar{z}} = 500\left(\frac{34.8}{33}\right)^{.2} = 505.339$$

$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}} = 0.866$$

∴ $G = 0.858821$

→ CALCS

∴ MAIN WIND FORCE RESISTING Systems (PERTAINING TO LOW-RISE BUILDING)

$$p = q_h(GC_{pf} - GC_{pi})$$

FIND q_h

0-15 ft: $0.00256 \cdot (.85)(1)(0.85)(90^2)(1.15) = 17.229$ lbs/ft²

15-20 ft: $0.00256 \cdot (.9)(1)(0.85)(90^2)(1.15) = 18.2425$ lb/ft²

20-25 ft: $0.00256 \cdot (.94)(1)(0.85)(90^2)(1.15) = 19.053$ lbs/ft²

25-30 ft: $0.00256 \cdot (.98)(1)(0.85)(90^2)(1.15) = 19.864$ lbs/ft²

30-40 ft: $0.00256 \cdot (1.04)(1)(0.85)(90^2)(1.15) = 21.0802$ lbs/ft²

40-50 ft: $0.00256 \cdot (1.09)(1)(0.85)(90^2)(1.15) = 22.09$ lbs/ft²

50-60 ft: $0.00256 \cdot (1.13)(1)(0.85)(90^2)(1.15) = 22.9045$ lbs/ft²

INTERPOLATION REQUIRED FOR 58′ → $= 22.742$ lb/ft²

FIND WIND PRESSURES

NOTE: $GC_p = 0.687$ ∴ $GC_{p_i} = -.429$ N-S

$= -.345$ E-W

$$p = qGC_p - q_i(GC_{p_i})$$

∴ FIRST FIND P in N-S DIRECTION

$0'-15' = (17.229)(.687) - (22.742)(-.429) = 21.60 \text{ lbs/ft}^2$

$15'-20' = (18.2425)(.687) + (9.756) = 22.30 \text{ lbs/ft}^2$

$20'-25' = (19.0533)(.687) + (9.756) = 22.86 \text{ lbs/ft}^2$

$25'-30' = (19.8641)(.687) + (9.756) = 23.41 \text{ lbs/ft}^2$

$30'-40' = (21.0802)(.687) + (9.756) = 24.25 \text{ lbs/ft}^2$

$40'-50' = (22.0937)(.687) + (9.756) = 24.95 \text{ lbs/ft}^2$

$50'-60' = (22.9045)(.687) + (9.756) = 25.50 \text{ lbs/ft}^2$

@ 58' $\longrightarrow = 25.39 \text{ lbs/ft}^2$

→ P in E-W DIRECTION

$0'-15' \rightarrow (17.229)(.687) - (22.7433)(-0.345) = 19.68 \text{ lbs/ft}^2$

$15'-20' \rightarrow (18.2425)(.687) + (7.846) = 20.38 \text{ lbs/ft}^2$

$20'-25' \rightarrow (19.0533)(.687) + (7.846) = 20.93 \text{ lbs/ft}^2$

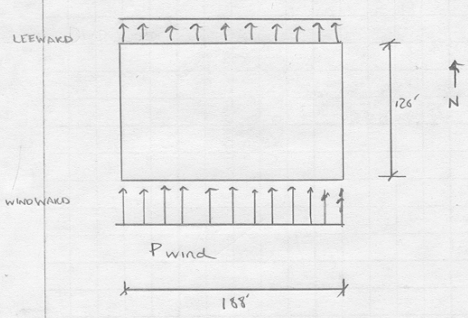
$25'-30' \rightarrow (19.8641)(.687) + (7.846) = 21.49 \text{ lbs/ft}^2$

$30'-40' \rightarrow (21.0802)(.687) + (7.846) = 22.37 \text{ lbs/ft}^2$

$40'-50' \rightarrow (22.0937)(.687) + (7.846) = 23.0233 \text{ lbs/ft}^2$

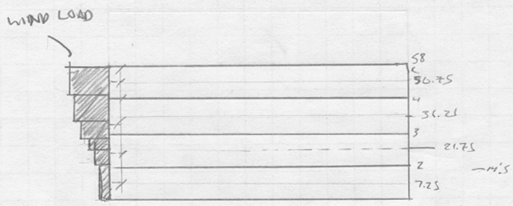
$50'-60' \rightarrow (22.9045)(.687) + (7.846) = 23.58 \text{ lbs/ft}^2$

@ 58' $\longrightarrow = 23.469 \text{ lbs/ft}^2$



Typical Floor Ht = 14.5'
$\frac{1}{2}$ = 7.25

→ North-South Direction



Floor 2
$= (7.75')(21.60\ lbs/ft^2) + (5')(22.30\ lbs/ft^2) + (1.75')(22.86\ lbs/ft^2) = 319\ lbs/ft \cdot 188' = 59.97\ K$

Floor 3
$= (3.25')(22.86) + (5')(23.41\ lbs/ft^2) + (6.25')(24.25\ lbs/ft^2) = 342.9\ lbs/ft = 64.47\ K$

Floor 4
$= (3.75')(24.25\ lbs/ft^2) + (10')(24.95\ lbs/ft^2) + .75'(25.50) = 359.56\ lbs/ft = 67.60\ K$

Roof
$= (7.25')(25.50\ lbs/ft^2) = 180.875\ lbs/ft \times 188' = 34\ K$

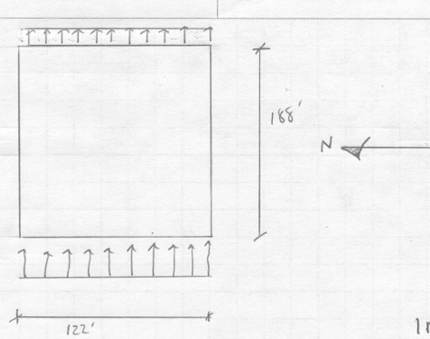
Total Base Shear = 225.91 K

Overturning Moments

Roof = $(34^K)(58') = 1,972\ 'K$
4^{th} Floor = $(67.60)(43.5) = 2,940\ 'K$
3^{rd} Floor = $(64.47)(29) = 1,869\ 'K$
2^{nd} Floor = $(59.97)(14.5) = 870\ 'K$

Overturning Moment = 7,651 'K

(4)



IN E-W DIRECTION

SAME WIND LOAD DISTRIBUTION, JUST DIFFERENT LOADS

CALCS

ROOF : $(7.25')(23.58') = 171 \text{ lbs/ft} \cdot 122' = 20.86$ kips

4^{TH} FLOOR : $(3.75')(22.33 \text{ lbs/ft}^2) + (10')(23.02 \text{ lbs/ft}) + (.75)(23.58 \text{ lbs/ft}) = 331.62 \text{ lbs/ft}^2 \cdot 122' = 40.5$ Kips

3^{rd} FLOOR : $(3.25')(20.93 \text{ lbs/ft}^2) + (5')(21.49) + (6.25')(22.33 \text{ lbs/ft}) = 315.035 \text{ lbs/ft}^2 \cdot 122' = 38.434$ Kips

2^{ND} FLOOR : $(7.75')(19.68 \text{ lbs/ft}) + (5')(20.38 \text{ lbs/ft}) + (1.75')(20.93 \text{ lbs/ft}) = 291.05 \text{ lbs/ft} \cdot 122' = 35.507$ Kips

TOTAL BASE SHEAR = 135.301 K

OVERTURNING MOMENTS

ROOF = (20.86)(58') = 1209.88 'K

4^{TH} FLOOR = (43.5')(40.5 k) = 1761.75 'K

3^{RD} FLOOR = (29')(38.43 k) = 1114.5 'K

2^{ND} FLOOR = (14.5')(35.507 k) = 514.75 'K

OVERTURNING MOMENT = 4601 'K

Velocity Pressure Envelope						
	Windward		Leeward		Max psf	
Z(ft)	N-S	E-W	N-S	E-W	N-S	E-W
0-15	11.84	11.84	-9.77	-7.84	21.60	19.68
15-20	12.53	12.53	-9.77	-7.84	22.30	20.38
20-25	13.09	13.09	-9.77	-7.84	22.86	20.93
25-30	13.65	13.65	-9.77	-7.84	23.41	21.49
30-40	14.48	14.48	-9.77	-7.84	24.25	22.33
40-50	15.18	15.18	-9.77	-7.84	24.95	23.02
50-60	15.74	15.74	-9.77	-7.84	25.50	23.58
58	15.63	15.63	-9.77	-7.84	25.39	23.47

Shear Summary		
(kips)	N-S	E-W
Shear @ Roof	34	20.86
Shear @4	67.6	40.5
Shear @3	64.47	38.43
Shear @2	59.97	35.5
Shear @1	0	0
Base Shear	226.04	135.301
Overturning Moment	7651.80	4601

Seismic:

SEISMIC LOAD DESIGN

Design Parameters		
# of stories	4	
h_s	14.5	**ft**
h_n	58	**ft**
Seismic Use Group	III	
Occ. Importance Factor	1.15	
S_S	0.39	**g-s**
S_1	0.09	**g-s**
F_a	1.00	
F_v	1.00	
S_{MS}	0.39	**g-s**
S_{M1}	0.09	**g-s**
S_{DS}	0.26	**g-s**
S_{D1}	0.06	**g-s**
Seismic Design Cat.	C	

Assumptions:

1) Assumed stiff soil
2) not specifically detailed for seismic resistance
3) Ordiinary Steel Concentrically braced
4) NO partition LL accounted for

Equivalent Lateral Force Procedure					
N-S Direction			E-W Direction		
	R_{N-S}	5		R_{N-S}	5
	$C_{S,N-S}$	0.060		$C_{S,E-W}$	0.060
	$C_{T,N-S}$	0.02		$C_{T,E-W}$	0.02
	X	0.75		X	0.75
	T_{N-S}	0.42		T_{E-W}	0.42
but not greater than:			but not greater than:		
	$C_{smax, N-S}$	0.033		$C_{smax, E-W}$	0.033
and	$C_{s\,min}$	0.0132		$C_{s\,min}$	0.0132
Therefore, ($C_{S, N-S}$) used is:		0.033	Therefore, ($C_{S, N-S}$) used is:		0.033

Loading Characteristics

Roof:		Slab Floors:	
DL (psf)		DL (psf)	
3.5" Concrete Slab	43.8	Concrete Slab	56.3
Metal Deck Roof	2	Metal Deck	2
Structural Framing	15	Structural Framing	15
MEP Sevices	25	MEP Services	20
Total:	85.8	Total:	93.3

Perimeter Wall:		Flat Roof Snow Load:	
DL (psf)		DL (psf)	
	45		4.6

Calculation Variables	
Building Width:	126 ft
Building Length:	188 ft
Floor Area:	23688 ft^2
Total weight of roof:	2346.3 kips
Total weight per floor:	2619.9 kips
Total Building Weight:	10205.9 kips
Seismic Shear, V_{N-S}:	**335.1 kips**
Seismic Shear, V_{E-W}:	**335.1 kips**

Vertical Distribution of Seismic Forces

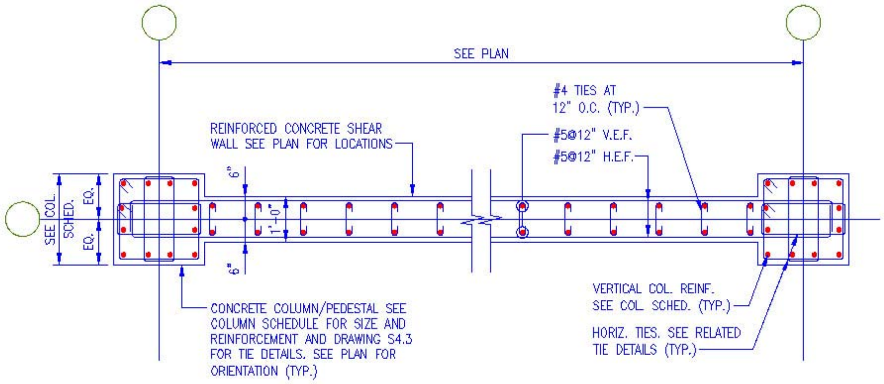
Exponent k_{N-S} : 0.960170131

North - South Direction							
Level, x	w_x	h_x	$w_x h_x^k$	C_{VX}	F_X	V_x	M_x
	kips	feet			kips	kips	ft-kips
Roof	2346	58	115,763	0.368	123.4		7155.2
4	2620	43.5	98,063	0.312	104.5	123.4	4545.9
3	2620	29	66,440	0.211	70.8	227.9	2053.3
2	2620	14.5	34,150	0.109	36.4	298.7	527.7
1						**335.1**	
Σ	10206		314416.4	1	**335.1**		**14282.1**

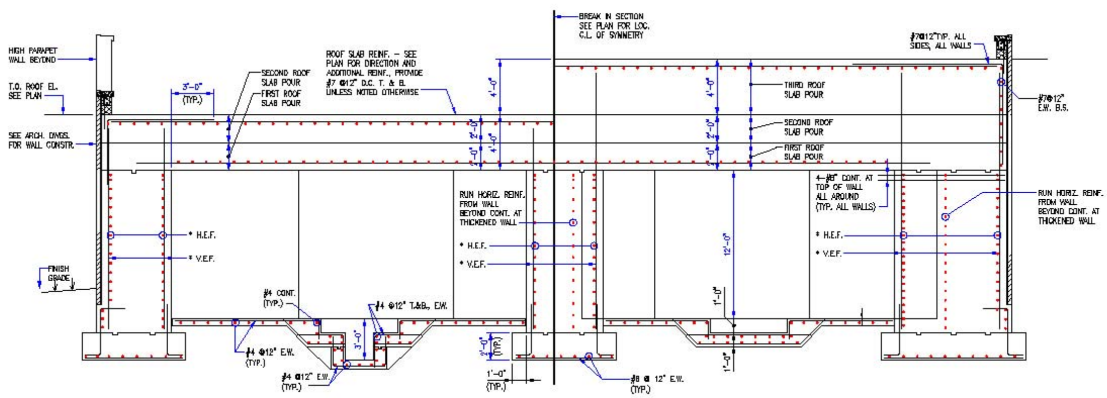
Exponent k_{N-S} : 0.96017

East - West Direction							
Level, x	w_x	h_x	$w_x h_x^k$	C_{VX}	F_X	V_x	M_x
	kips	feet			kips	kips	ft-kips
Roof	2346	58	115,763	0.368	123.4		7155.2
4	2620	43.5	98,063	0.312	104.5	123.4	4545.9
3	2620	29	66,440	0.211	70.8	227.9	2053.3
2	2620	14.5	34,150	0.109	36.4	298.7	527.7
1						**335.1**	
Σ	10206		314416.4	1	**335.1**		**14282.1**

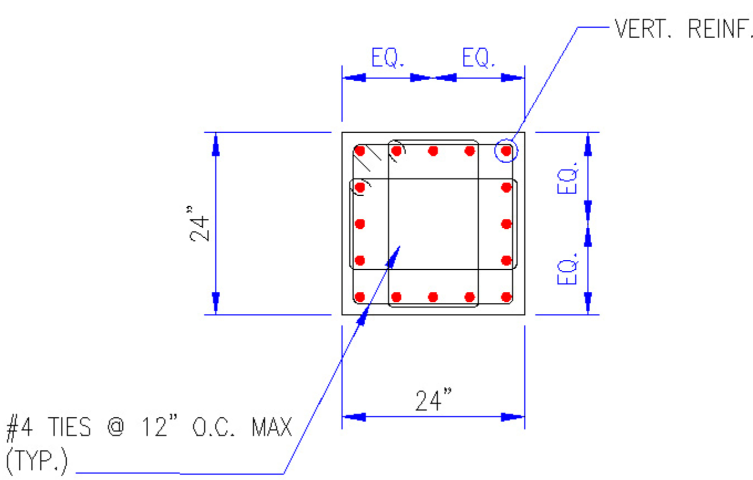
APPENDIX E:



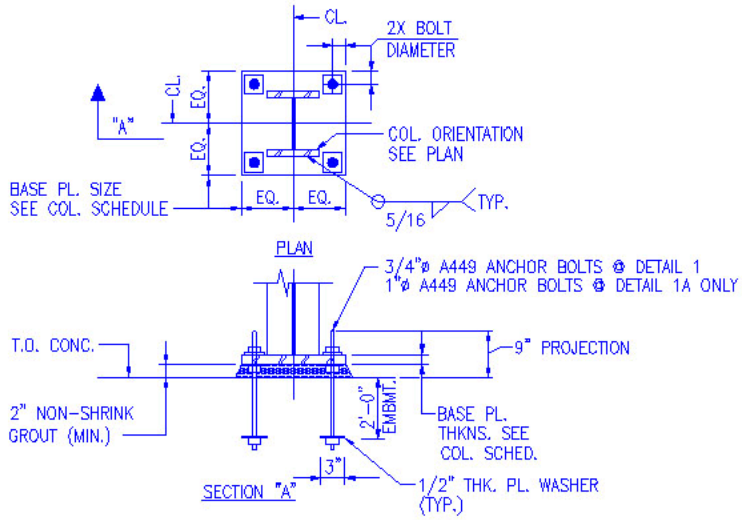
(X-1) SHEAR WALL SECTION



(X-2) LINEAR ACCELERATOR SECTION



(X-3) COLUMN REINFORCEMENT



(X-4) COLUMN CONNECTIONS