Technical Report 1

University Hospitals
Case Medical Center
Cancer Hospital

11100 Euclid Avenue Cleveland, Ohio

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Structural Option
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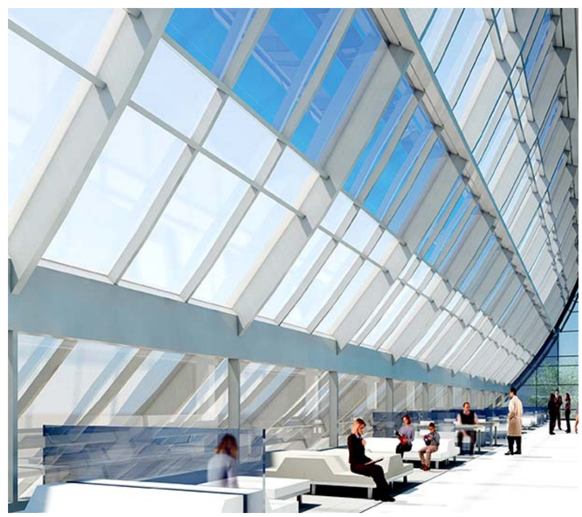
http://www.engr.psu.edu/ae/thesis/portfolios/2009/dcm234

Executive Summary

The University Hospitals Case Medical Center Cancer Hospital is a 9 story research and patient care facility located in Cleveland, Ohio. Its infrastructure consists of steel and steel composite members which have been carefully arranged in order to conform to the modular architectural design system known as the *Universal Grid*, allowing full optimization of available space for varying use. Sloped curtain walls envelope the Cancer Hospital, consisting of exterior glazing and curved steel. The new Cancer Hospital will serve as an addition to the adjacent Case Medical Center which will integrate medical services once spread through 7 different buildings.

The purpose of this report is to analyze the design and construction of the University Hospitals Case Medical Center Cancer Hospital using current design codes and requirements, in effort to gain an understanding of its structural system. The building design is controlled by the *The Ohio Building Code 2006* referencing *The International Building Code 2006*. For this report, calculations and load determinations were conducted using the *IBC 2006*.



The lateral forces resulting from wind and seismic loads were found and broken down specifically to the individual levels of The Cancer Hospital. The controlling force was found to be a distributed wind load of 22.63psf producing an 82.53k story force to the top of the structure in the north/south direction. This is consistent with the low seismic region in which the building is located. The lateral forces are resisted by concentric braced frames which will be analyzed in a future report. Gravity loads were calculated using a unique Load Key Diagram provided by the engineering consultant in order to accurately determine the highly varying distributed loads of the Cancer Hospital. This method was efficient in determining a base shear which was within 1% error of the engineering consultant's calculated value.

Spot checks were conducted in order to gain an understanding of the gravity load distribution to members and their corresponding design. The results revealed an overdesign of the composite slab system which may be due to the constricting vibration criteria required for the imaging and surgery rooms on the lower floors. The column design was also found to conservative by a strength factor of two for axial load. This most likely is due the without-holding of snow loads from the spot-check. The composite beams and girders were found to be controlled by deflection and adequate to the required load when given a 1" camber during construction. All additional calculations and assumptions are provided in Appendix A.

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Introduction

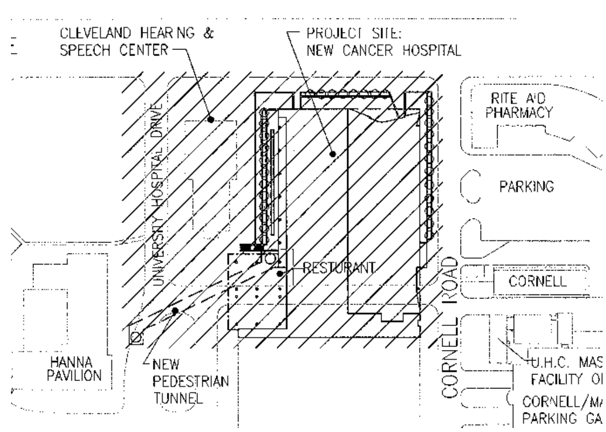
The University Hospitals Case Medical Center Cancer Hospital will integrate patient care and cancer research in a new and innovative way. Architecturally, the Cancer Hospital will reflect this cutting edge link by joining adjacent buildings together while serving as a primary gateway to the UHCMC campus located in Cleveland, Ohio.

The Cancer Hospital design fulfills the wishes of former facility cancer patients in creating an appealing and comfortable environment as opposed to the sterile feel of the past. This is accomplished through use of strong architectural accents including the Cancer Hospital's most dominating feature, its curved facade. A universal grid system consisting of 31'-6" modular bays has been incorporated into design to optimize floor space for varying uses. Clinical pods have been designed for treatment of specific patient populations.

Medical services which were previously distributed among seven facilities will now be performed under one roof to optimize cancer research, education, and patient care while providing an architecturally appealing exterior as well as a warm and inviting natural interior.

Structural System Overview

Foundation



The Cancer Hospital consists of drilled piers transferring load to caissons for the gravity columns with the combined use of grade beams for the lateral force resisting frames. The drilled gravity piers/caissons range 30" to 60" in diameter depending on location. The drilled piers/caissons receiving lateral load are typically 66" in diameter. Along the south side, 36" thick spread footings, typically 48" by 72", have been used to carry gravity load along the existing adjacent Case Medical Center Hospital. The grade beams which carry the lateral load to the drilled piers/caissons are typically 24" by 24" and consist of Grade 60, #7 reinforcement bars. All foundations are made from concrete having a compressive strength of 4000psi with the exception of the caissons and spread footings, which have a strength of 3000psi.

The soil on site has been classified as hard shale. Thus, giving the caissons used in the foundation an end bearing capacity of 50kpf with a skin friction capacity of 10psi below the first 5’ of shale. The typical minimum penetration depth for the gravity piers/caissons is 3’-0” and for the lateral, 16’-6”.

Floor System

Being a primarily steel structure, the Cancer Hospital has a fairly typical composite steel beam and girder framing system. The typical composite floor slab is 5-1/4” thick using 3000psi lightweight composite concrete, an 18 gauge 2” galvanized steel deck, and 3-1/2” metal studs. This composite floor slab is used on all but the 2nd and 8th floors. The second floor requiring a thicker slab with normal weight concrete due the vibration requirements of the surgery and imaging rooms and the 8th due to the increased load from the mechanical system. The slab used on these floors consists of 6-1/2” thick 3000psi normal weight concrete, an 18 gauge 2” galvanized steel deck, and 3-1/2” metal studs. Both decks are reinforced with 6x6 Welded Wire Fabric; W4.5xW4.5 for the first floor, W3.5xW3.5 for the second and eighth floors, and W2.1x2.1 for the remaining floors.

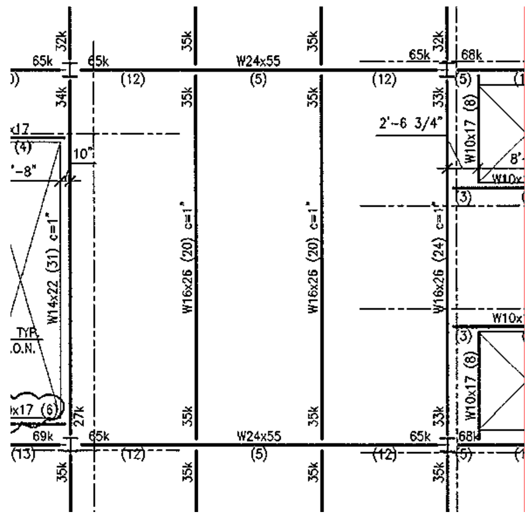


Figure 1

Framing

Bay sizes conform to the universal grid, having a typical size of 31’-6” by 31’-6”. Infill beams are typically W16x26 around the interior and W14x22 around the exterior framing into W24x68 girders (see Figure 1). For the larger breaks in the slab, such as the elevator shafts, HSS 8x4x1/4 tubes have been used. On the 4th and roof level, moment connections are utilized in conjunction with cantilevered beams in order to support the curved exterior façade. Smaller breaks used for mechanical, plumbing, etc., consist typically of W10x17. Columns consist of a typical W14 member decreasing in size with elevation and spliced every other floor starting with the second. All steel members conform to ASTM A-992, Grade 50 unless otherwise noted.

Ground Level

At the ground level, a 6” thick slab-on-grade is used with Grade 60 #5 reinforcement bars spaced @ 18”oc EW. The slab rests on a 10 mils min. vapor barrier on compacted granular material over a 2000psi mud slab. In the northeastern and southeastern section of the building special research equipment has been placed requiring a 12” thick slab-on-grade with Grade 60 #5 reinforcement bars placed @ 12”oc EW.

Machine Room

A 31'-0" by 63'-0" machine room is located on the 8th floor. Framing is similar to the rest of the structure however with shorter spans and larger members to account for the additional weight. Beams range from W21 beams to W40 beams depending on specific equipment.

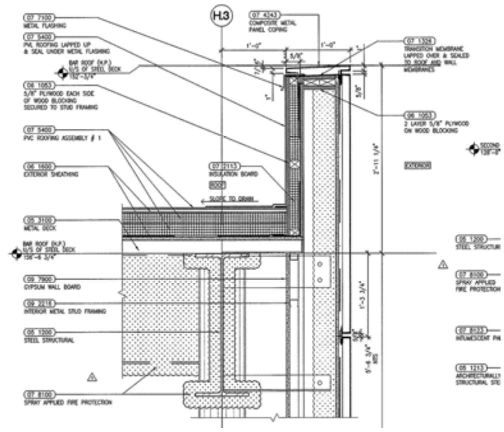


Figure 2

Roof System

The roof of the Cancer Center is a sloped deck with a 63'-0' by 63'-0" elevator penthouse perched at the southern corner. The roof slopes downward along the east and west sides of the building and allows drainage to the center third. The roof system consists of a 3"x20ga type 'N' galvanized steel deck. The roof deck rests typically rests on W14x22 beams framing into W21x44 girders with W18x35 beams being used to support mechanical equipment spaced uniformly across the building's center. Roof decks lower than the top of the 8th level consist of 1.5"x20ga. type 'B' galvanized steel deck (see Figure 2).

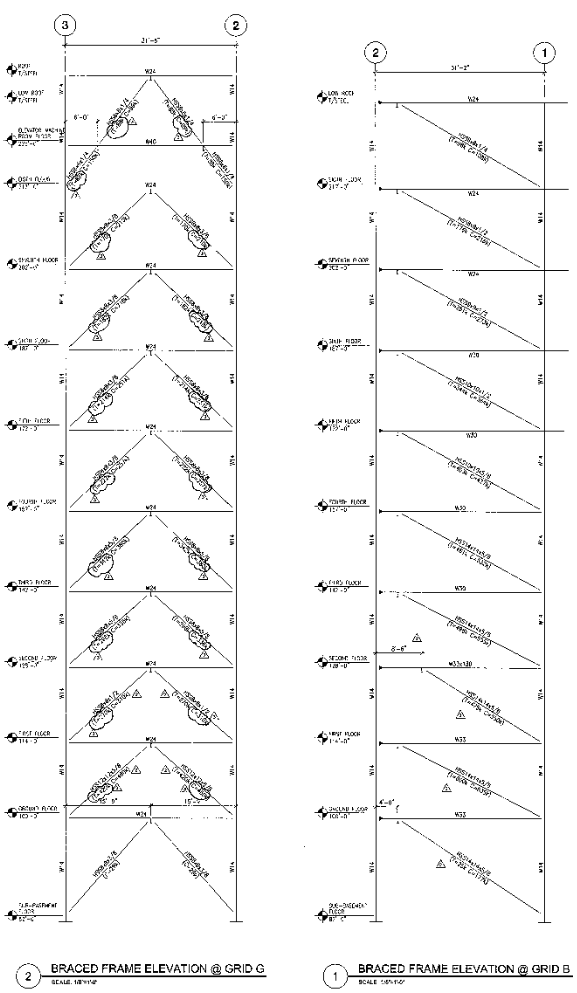


Figure 3

Lateral System

Lateral forces are resisted by a series of concentrically braced frames located at the center of the building near the main elevator core and along isolated points of the exterior bays (see Figure 4). This system consists of four chevron braces and single diagonal brace, which are used both in the north/south direction as well as the east/ west direction. Each brace typically consists of a 31'-6" wide W24 beam, a 15'-0' tall W14 column, and two HSS8 size diagonal members (see Figure 3). Structural brace members beyond the 8th floor increase in size due to increased lateral loads.

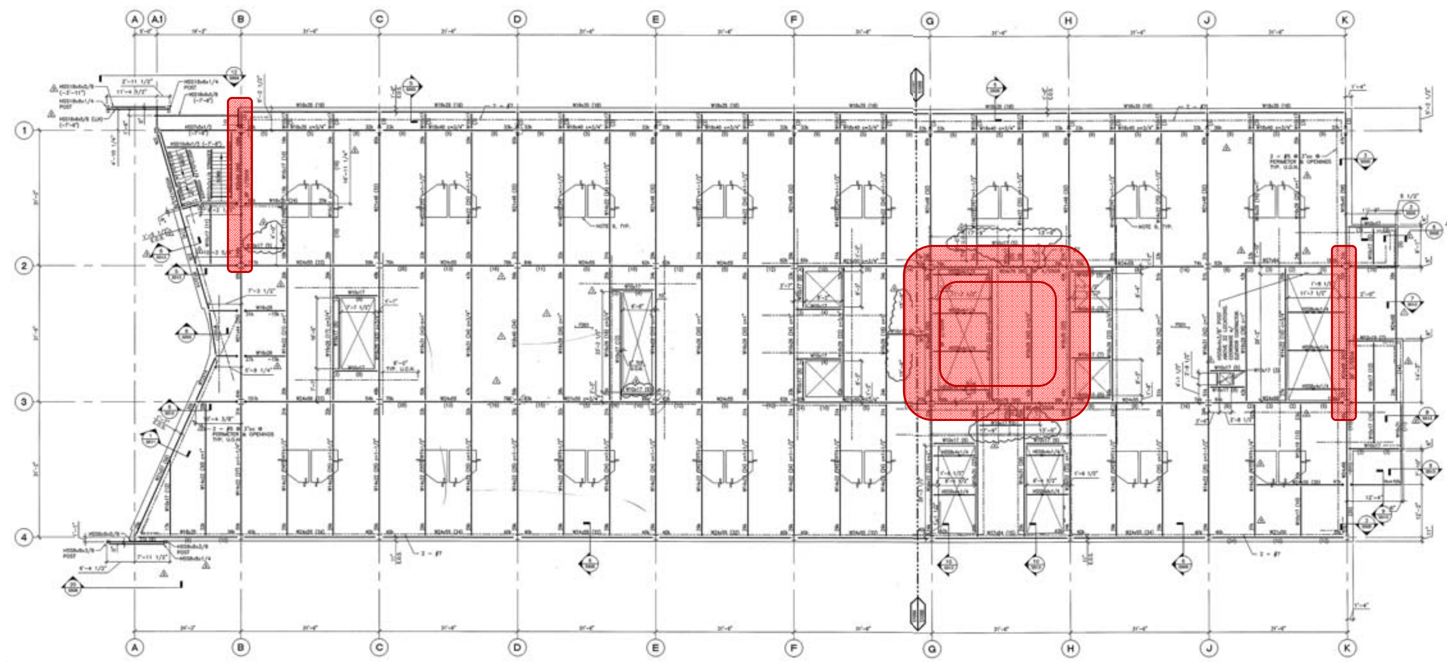


Figure 4

Codes and Design Requirements

Codified Ordinances of the City of Cleveland:

Land Use Code – Planning and Housing 6/3/07

Zoning Code 6/3/07

Land Use Code – Fire Prevention Code 6/3/07

Building Code 6/3/07

2007 Ohio Building Code (w/ 2006 International Building Code)

2006 International Mechanical Code

2006 International Plumbing Code

ACI 318-05 (Reinforced Concrete Design)

AISC LRFD 13th Edition (Steel Construction Manual)

ASCE 7-05 (Minimum Design Loads for Buildings and Other Structures)

Lateral and Gravity Load Analysis

Seismic Analysis

All tables, figures, and equations used in calculation of seismic loads were done so in accordance with Chapter 12 of ASCE 7-05. Although the structure falls under design category A allowing simplified procedure, the Equivalent Lateral Force Procedure was utilized in order to gain greater accuracy in the development of results. Due to the complexity and diversity of loads on each floor of the Cancer Center, a Load Key Diagram was obtained from the structural consultant in order to accurately calculate effective story weight to be used in the Equivalent Lateral Force Procedure (see Appendix B).

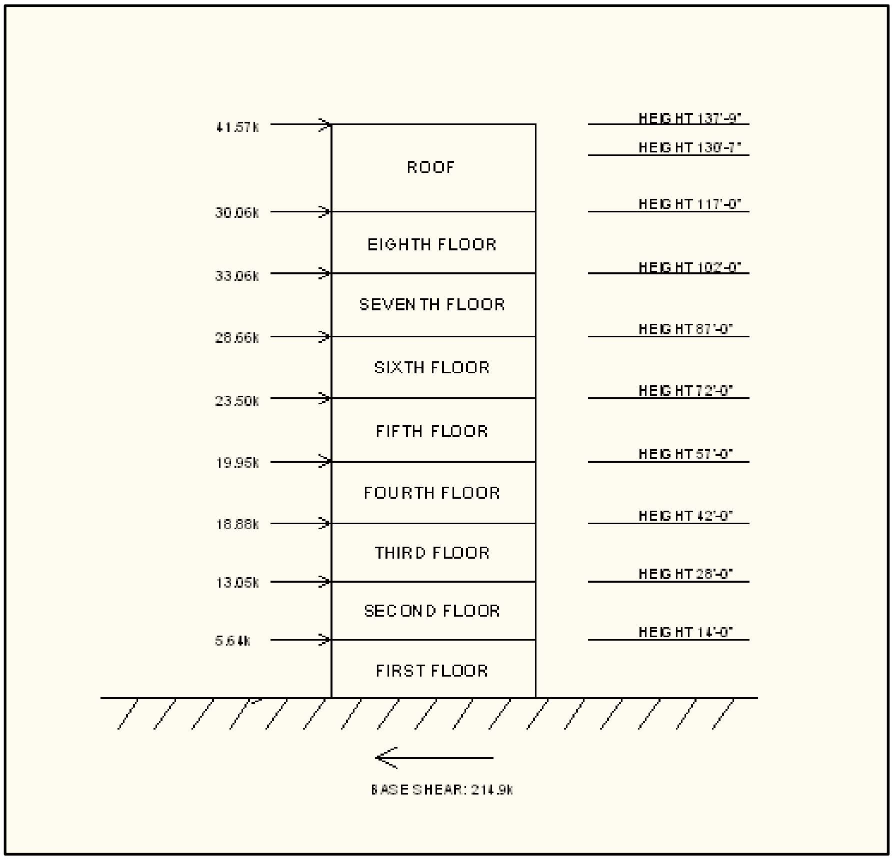


Figure 5

Superimposed line and area dead loads from the diagram can be applied to each respective zoned area on each of the 9 levels. The penthouse level weight has been neglected due to its small amount of contribution to the period. After calculation, these loads were determined to include self-weight. The dead load distribution is shown in the following Tables 1 through 3: (see Appendix A for calculations).

GRAVITY LOAD				
Level	**Description**	**Load**	**Area / Dist**	**Total(lb)**
Roof	Roof Load	25psf -41psf	28200 ft^2	800747
	Building Envelope	300plf - 500plf	1921 ft	713100
	Ceiling Partition	5psf	26791 ft^2	133955
	Suspended Mechanical Equipment	10psf	26791 ft^2	267910
	Interior Shafts	225plf	812ft	182700
8th	Floor Load	30psf -70psf	28315 ft^2	883095
	Building Envelope	300plf	814 ft	244200
	Ceiling Partition	5psf	26791 ft^2	133955
	Suspended Mechanical Equipment	10psf	26791 ft^2	267910
	Interior Shafts	225plf	812ft	182700
7th	Floor Load	47psf	28516 ft^2	1340252
	Building Envelope	300plf	814 ft	244200
	Ceiling Partition	5psf	28516 ft^2	142580
	Suspended Mechanical Equipment	10psf	28516 ft^2	285160
	Interior Shafts	225plf	812ft	182700
6th	Floor Load	47psf	28518 ft^2	1340346
	Building Envelope	300plf	814 ft	244200
	Ceiling Partition	5psf	28518 ft^2	142590
	Suspended Mechanical Equipment	10psf	28518 ft^2	285180
	Interior Shafts	225plf	812ft	182700
5th	Floor Load	47psf	28188 ft^2	1324836
	Building Envelope	300plf	814 ft	244200
	Ceiling Partition	5psf	28188 ft^2	140940
	Suspended Mechanical Equipment	10psf	28188 ft^2	281880
	Interior Shafts	225plf	812ft	182700
4th	Floor Load	47psf	28062 ft^2	1318914
	Building Envelope	300plf - 360 plf	1289 ft	409740
	Ceiling Partition	5psf	28062 ft^2	140310
	Suspended Mechanical Equipment	10psf	28062 ft^2	280620
	Interior Shafts	225plf	812ft	182700
3rd	Floor Load	47psf	40492 ft^2	1903124
	Building Envelope	300plf	1006 ft	301800
	Ceiling Partition	5psf	40492 ft^2	202460
	Suspended Mechanical Equipment	10psf	40492 ft^2	404920
	Interior Shafts	225plf	812ft	182700
2nd	Floor Load	47psf	41393 ft^2	1945471
	Building Envelope	300plf - 560plf	1006 ft	357180
	Ceiling Partition	5psf	41393 ft^2	206965
	Suspended Mechanical Equipment	10psf	41393 ft^2	413930
	Interior Shafts	225plf	812ft	182700
1st	Floor Load	47psf	41662 ft^2	1958114
	Building Envelope	300plf	336 ft	100800
	Ceiling Partition	5psf	41662 ft^2	208310
	Suspended Mechanical Equipment	10psf	41662 ft^2	416620

Table 1

GRAVITY LOAD	
Level	**Load(lb)**
Roof	2098412
8	1711860
7	2194892
6	2195016
5	2174556
3	2995004
2	3106246
1	2683844
Total Wt.	19159830

Table 2

SEISMIC FACTORS			
S_s	19.20%	S_{ds}	0.1536
S_1	5.10%	S_{d1}	0.058
Site Class	C	Occupancy Cat.	IV
F_a	1.2	Seismic Des. Cat.	A
F_v	1.7	C_s	0.01
S_{ms}	0.2304	T_a	1.31
S_{m1}	0.0867	V	214.9k
R	7	I	1.5

Table 3

The maximum lateral story force was determined to be 41.57k at the roof level (see Table 4). This value is reasonable due to the low amount of seismic activity in Cleveland, Ohio. Base shear was found to be 214.9 kips, placing it within 1% error when compared to value of 213.6k calculated by the consultant engineer (see Figure 5). For further calculation details see Appendix A.

SEISMIC FORCES					
Level	wx	hi	Σwihi	Cvx	Story Force(k)
Roof	2098412	132	1431970472	0.19	41.57
8	1711860	117	1431970472	0.14	30.06
7	2194892	102	1431970472	0.16	33.60
6	2195016	87	1431970472	0.13	28.66
5	2174556	72	1431970472	0.11	23.50
4	2332284	57	1431970472	0.09	19.95
3	2995004	42	1431970472	0.09	18.88
2	3106246	28	1431970472	0.06	13.05
1	2683844	14	1431970472	0.03	5.64

Table 4

Wind Analysis

As in the seismic analysis, all tables, figures, and equations for calculation of wind loads were done so in accordance with chapter 6 of ASCE 7-05. Method 2 of the Main Wind-Force Resisting Systems, also known as the Analytical Method, was used in determination of lateral wind pressures. Wind forces on the Cancer Hospital were analyzed both in the north/south direction as well as the east/west direction which resulted in different gust factors due to flexibility (see Figure 6-7). A conservative approach was taken in east/west direction in order to account for the vertical "L" shape caused by the lower 4 story, southern wing of the Cancer Hospital. Since the code is unclear about applying wind pressures to non-uniform shapes, a rectangular shape was used in calculation. This will cause the lateral forces to be larger than in actuality (see Appendix B for North Elevation).

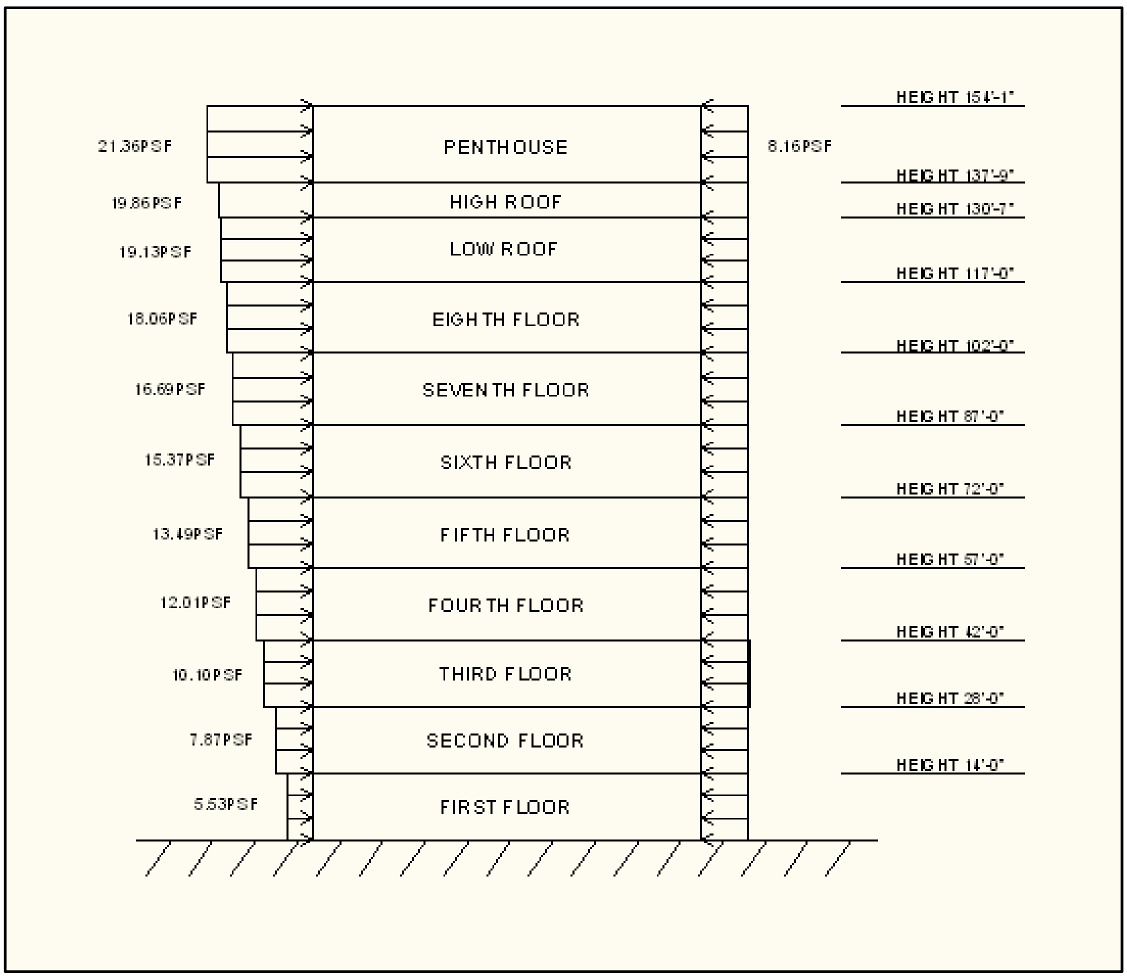


Figure 6

East – West Direction

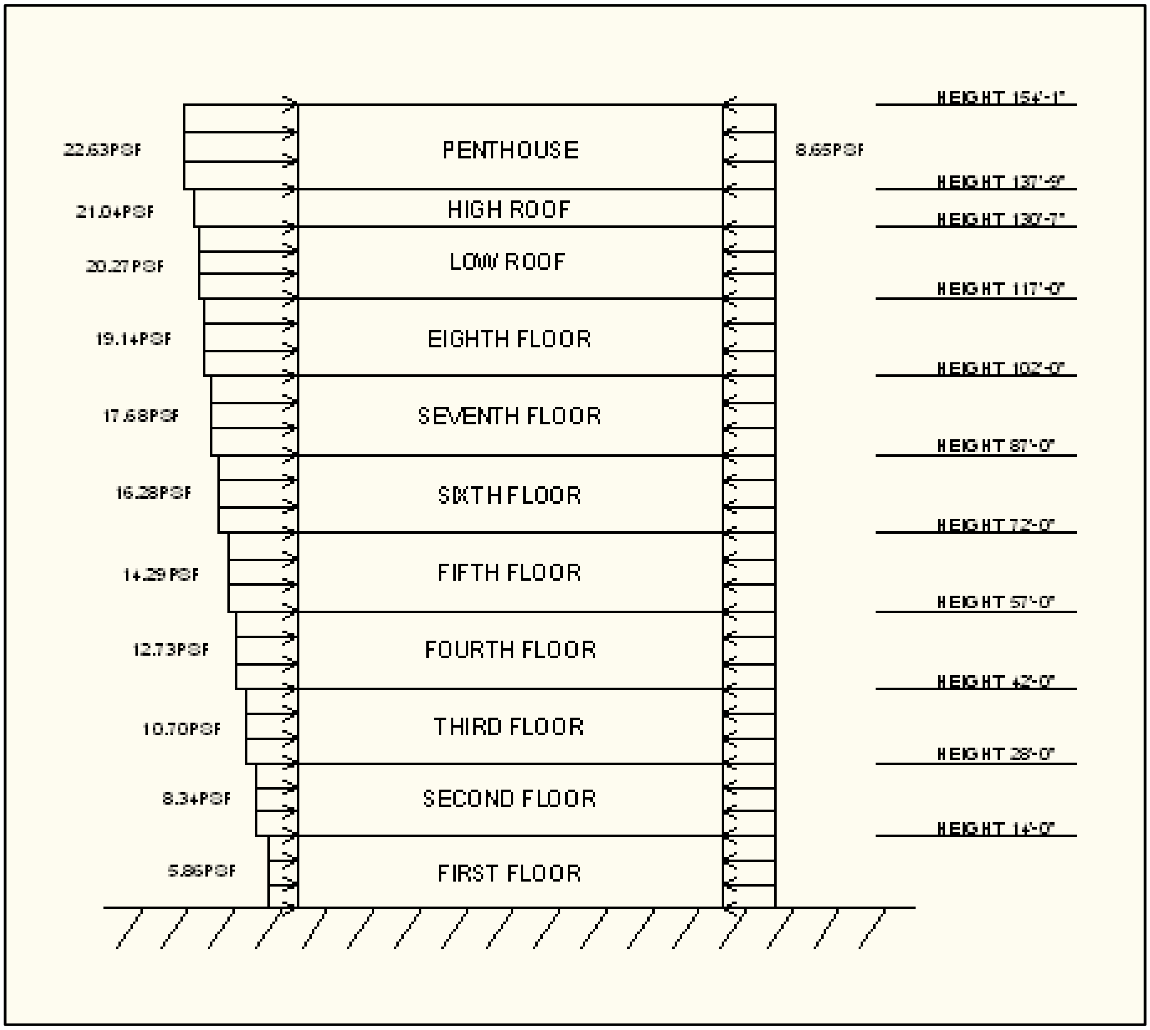


Figure 7

North – South Direction

For this analysis internal pressures and roof top uplift pressures have been ignored, however, overturning moment has been determined for future calculation. The maximum wind pressure was found to be 22.63 psf in the north/south direction . When compared to the seismic forces, wind controls the lateral design with a story force of 82.53k. The wind pressure and story shear for each in the following Tables 5 through 8: (see Appendix A for calculations).

WIND FACTORS			
V	90mph	n	0.4
K_d	0.85	G	.89/.84
I	1.15	q_z	20.27
Exposure Cat.	B	q_i	22.7
K_{zt}	1	q_h	22.7
K_h	1.2	C_p	0.8

Table 5

WIND ANALYSIS					
	Story	Tributary Height (ft)	Height Above Ground Level, z (ft)	Kz	qz (psf)
Windward	Penthouse	16.33	154.08	1.12	22.7
	High Roof	7.17	137.75	1.08	21.9
	Low Roof	13.58	130.58	1.06	21.5
	8	15	117	1.03	20.9
	7	15	102	0.99	20.1
	6	15	87	0.95	19.3
	5	15	72	0.89	18.0
	4	15	57	0.84	17.0
	3	14	42	0.77	15.6
	2	14	28	0.68	13.8
	1	14	14	0.57	11.6
Leeward		154.1	All	1.12	22.7
Side		154.1	All	1.12	22.7

Table 6

NORTH - SOUTH DIRECTION						
	Story	Tributary Height (ft)	External Pressure qGC_p (psf)	Forces (k)	Story Shear (k)	Overturn Moment (ft-k)
Windward	Penthouse	16.33	22.63	82.53	523.95	41.27
	High Roof	7.17	21.04	33.69	441.42	1764.38
	Low Roof	13.58	20.27	61.48	407.73	3387.63
	8	15	19.14	64.11	346.25	6407.83
	7	15	17.68	59.23	282.14	10479.28
	6	15	16.28	54.54	222.90	15404.04
	5	15	14.29	47.87	168.36	21096.88
	4	15	12.73	42.64	120.49	27468.57
	3	14	10.70	33.44	77.85	34171.12
	2	14	8.34	26.08	44.41	41067.29
	1	14	5.86	18.33	18.33	48274.31
Leeward	All	154.1	-8.65	N/A	N/A	N/A
Side	All	154.1	-14.14	N/A	N/A	N/A

Table 7

EAST - WEST DIRECTION						
	Story	Tributary Height (ft)	External Pressure qGC_p (psf)	Forces (k)	Story Shear (k)	Overturn Moment (ft-k)
Windward	Penthouse	16.33	21.36	77.89	494.52	38.95
	High Roof	7.17	19.86	31.80	416.62	1665.26
	Low Roof	13.58	19.13	58.02	384.82	3197.32
	8	15	18.06	60.51	326.80	6047.84
	7	15	16.69	55.90	266.29	9890.56
	6	15	15.37	51.48	210.38	14538.64
	5	15	13.49	45.18	158.90	19911.66
	4	15	12.01	40.25	113.72	25925.39
	3	14	10.10	31.56	73.48	32251.39
	2	14	7.87	24.62	41.91	38760.13
	1	14	5.53	17.30	17.30	45562.27
Leeward	All	154.1	-8.16	N/A	N/A	N/A
Side	All	154.1	-13.35	N/A	N/A	N/A

Table 8

Gravity Spot-Checks

A typical section was selected between column lines E and F on the 6th level of the structure for a gravity load spot-check (see Figure 8). The selected section conforms to the 31'-6" *Universal Grid* and complies with that of the typical bay described in the *Structural System Overview.* All tables, figures, and equations used in calculation were consistent with the AISC *Steel Construction Manual 13thed* and *ASCE7-05*. Values are calculated using LRFD design factors, specifically load combination 1.2D+1.6L. A deflection criterion of L/240 was applied for the total load with a L/360 limit being applied for live loads only.

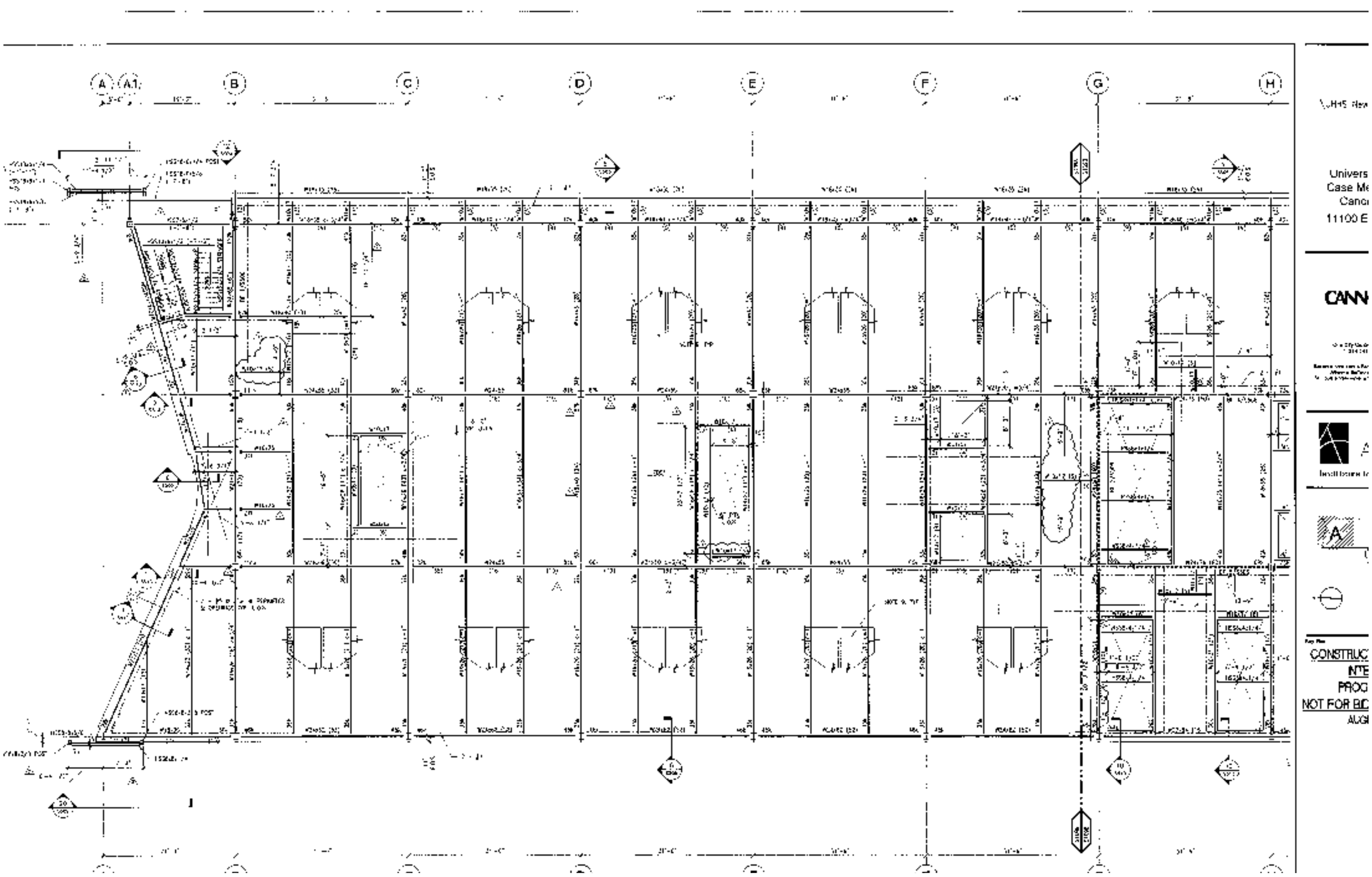


Figure 8

Using the *Unified Steel Deck Catalogue*, the slab system was analyzed and determined to be over-designed in excess of 100psf. A strong possibly for this excess strength may be due to the vibration consideration for the imaging and surgery rooms on the lower floors (see Appendix B for Profile Section). The composite beam and girder system were found to be controlled by deflection and have been cambered by 1" for construction. The W14x90 column at grid location E-3 was analyzed for axial loading. This column is spliced at level 5 and level 7 and carries the load of all above levels (see Table 9).

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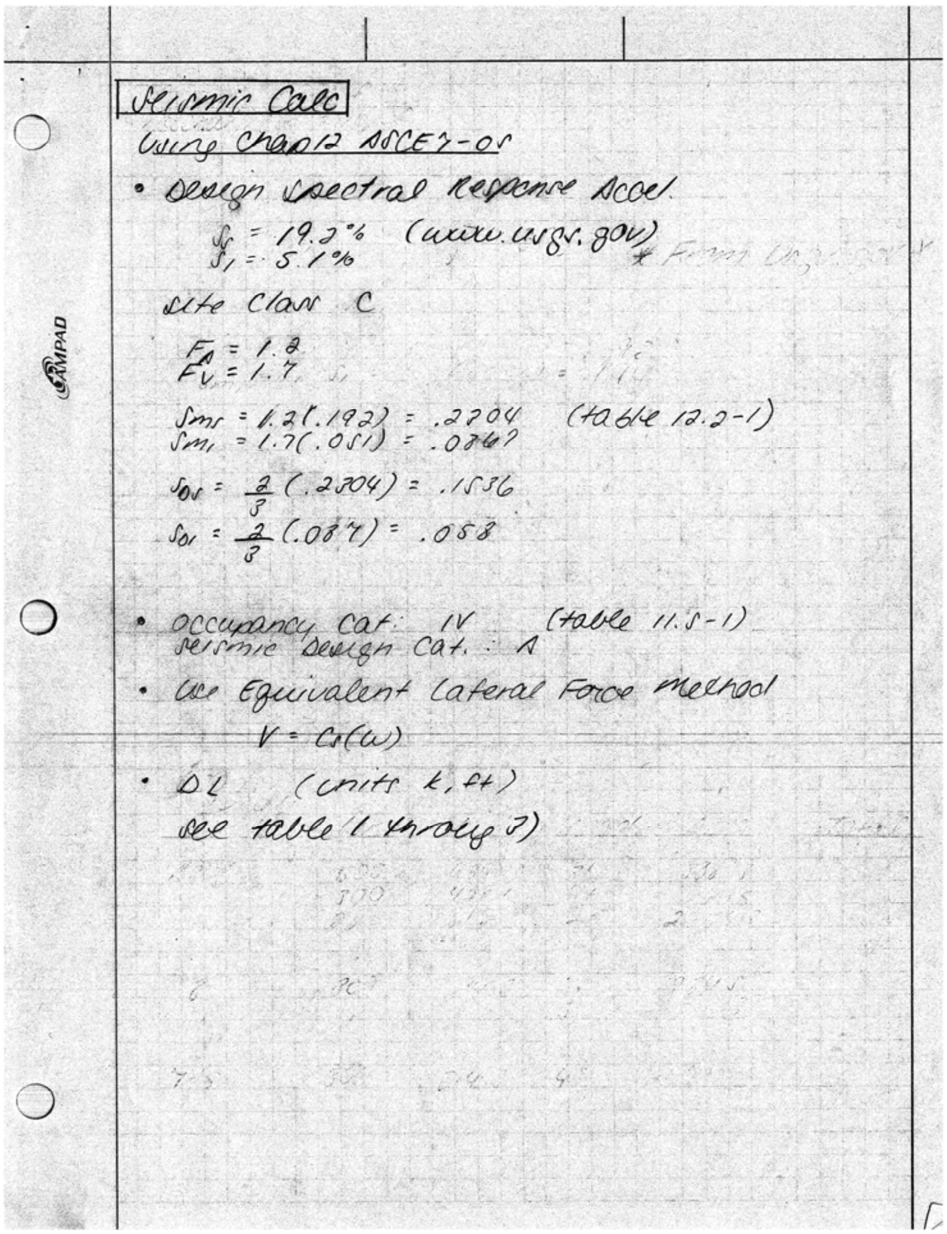
COLUMN SPOT-CHECK LOADS							
Level	Tibutary Area (In^2)	Dead Load (psf)	Live Load (psf)	Live Load Reduction	Total Load (psf)	Factored Load (psf)	Axial Load (kip)
Roof	992.25	25	30	30	55	78	77.4
8	992.25	30	150	150	180	276	273.9
7	992.25	47	60	60	107	152.4	151.2
6	992.25	47	60	24	71	94.8	94.1

Table 9

The capacity for this column was found to be twice that of the required gravity load. A possible explanation for this is that rain and snow load were not considered during this analysis. All members analyzed for gravity were adequate to carry their required load (see Appendix A for calculations).

Appendix A

Seismic Calculations


Seismic Calc
Using Chap 12 ASCE7-05
• Design Spectral Response Accel.
S_s = 19.2% (www.usgs.gov) * From USGS
S_1 = 5.1%
Site Class C
F_a = 1.2
F_v = 1.7
S_{MS} = 1.2(.192) = .2304 (table 12.2-1)
S_{M1} = 1.7(.051) = .0867
$S_{DS} = \frac{2}{3}(.2304) = .1536$
$S_{D1} = \frac{2}{3}(.087) = .058$
• Occupancy Cat. IV (table 11.5-1)
Seismic Design Cat.: A
• Use Equivalent Lateral Force Method
$V = C_s(W)$
• DL (units k, ft)
see table 1 through 3

Seismic Calculations

- Calculate seismic base shear

$I = 1.5$

$R = 7$ (eccentric braced frame, non-moment)

$C_s = \frac{S_{DS}}{(R/I)} = \frac{.1536}{(7/1.5)} = .033$

$T_a = C_t h_n^x$ $\quad C_t = .03 \quad x = .75 \quad h_n = 172.08'$

$T_a = .03(154.08)^{.75}$ (eq. 12.8-7)

$= 1.87$ sec (table 12.8-2)

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

$\leq \frac{.058}{1.87(7/1.5)}$

$= .009$

$.009 \leq .01$

$C_s = .01$ (12.8-5)

$V = C_s W$

$= .01(21482014.16)$

$= 206.7$

$V = 214.9k$ (eq. 12.8)

$V_{ACT} = 213.6^k$

Error ≤ 5% ✓ okay

Wind Calculations

Wind:

- $V = 90$ mph Fig 6-1 (6.5.4)
- $K_d = .85$ Table 6-4 (6.5.4.4)
- $I = 1.15$ Table 6-1 (6.5.5)
- Wind Exp. Cat. : B
- $K_{zt} = 1.0$ not situated on hill, ridge, or escarpment
- $K_h = 1.12$ (Table 6-3)
- $q_p = .00256 K_h K_{zt} K_d V^2 I$

  $= .00256(1.12)(1.0)(.85)(90)^2(1.15)$

  $= 22.7$ psf
- $p_p = q_p GC_{pn}$ → parapet

  $= 22.7(1.5) = 34.1$ psf for windward parapet

  $= 22.7(-1.0) = -22.7$ psf for leeward parapet

parapet height = 4'-3"

$F = 34.1(4.25) = 144.9$ plf windward
$F = 22.7(4.25) = 96.5$ plf leeward

- Determine whether flexible or rigid

  $n = \frac{22.2}{H^{.8}} = \frac{22.2}{(184)^{.8}}$ (ASCE 7-05; C6.5.8)

  $n = .40 < 1$ <u>flexible</u>

Wind Calculations

• Gust Factors [North-south]

$g_Q = g_v = 3.4$; $n_1 = .40$

$g_R = \sqrt{2 \ln(3600 n_1)} + \frac{.577}{\sqrt{2 \ln(3600 n_1)}}$

$= 3.82 + .15$

$g_R = 3.97$ Eq 6-9

$\bar{z} = .6(154)$

$= 92' > z_{min} = 30'$ ✓ okay (table 6-2)

$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6}$ $c = .3$ (table 6-2)

$= .3\left(\frac{33}{92}\right)^{1/6}$

$= .25$

$L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}}$ $\ell = 320'$; $\bar{\epsilon} = 1/3.0$ (table 6-2)

$= 320\left(\frac{92}{33}\right)^{1/3.0}$

$= 450.4$

* $Q = \sqrt{\frac{1}{1 + .63\left(\frac{B+h}{L_{\bar{z}}}\right)^{.63}}}$ $B = 304.25$

$= \sqrt{\frac{1}{1.63}}$

$= .781$

$\bar{V}_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}} V\left(\frac{88}{60}\right)$ $\bar{\alpha} = 1/4.0$; $\bar{b} = .45$ (table 6-2)

$= .45\left(\frac{92}{33}\right)^{1/4.0}(90)\left(\frac{88}{60}\right)$

$= 76.8$ mph

$N_1 = \frac{n_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}} = \frac{.46(450.4)}{76.8}$

$= 2.7$

Wind Calculations

$$R_n = \frac{7.47(N_1)}{(1+10.3(N_1))^{5/3}} = \frac{7.47(2.7)}{(1+10.3(2.7))^{5/3}} = .074$$

$$\eta_h = 4.6 n_1 h/\bar{V}_z = 4.6(.46)\left(\frac{154}{76.8}\right) = 4.24$$

$$R_h = \frac{1}{\eta} - \frac{1}{2\eta^2}(1-e^{-2\eta}) = \frac{1}{4.24} - \frac{1}{2(4.24)^2}(1-e^{-2(4.24)}) = .21$$

$$\eta_B = 4.6 n_1 B/\bar{V}_z = 4.6(.46)\frac{(304.20)}{76.8} = 8.38$$

$$R_B = \frac{1}{8.38} - \frac{1}{2(8.38)^2}(1-e^{-2(8.38)}) = .12 - .007(.999) = .113$$

$$\eta_L = 15.4(n_1)(L)/\bar{V}_z \qquad L = 228.33'$$

$$= 15.4(.46)\frac{(228.33)}{76.8} = 20.6$$

$$R_L = \frac{1}{20.6} - \frac{1}{2(20.6)^2}(1-e^{-2(20.6)}) = .048 - .0011(1) = .047$$

$$R = \sqrt{\frac{1}{\beta} R_n R_h R_B(.53+.47R_L)} \qquad \beta = 1.0 \text{ (steel)}$$

$$= \sqrt{\frac{1}{.01}(.074)(.21)(.113)(.53+.47(.047))} = .371$$

Wind Calculations

$$G_f = .925\left(\frac{1 + 1.7 I_{\bar{z}}\sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1 + 1.7 g_v I_{\bar{z}}}\right)$$

$$= .925\left(\frac{1 + 1.7(.25)\sqrt{(3.4)^2(.781)^2 + (3.97)^2(.871)^2}}{1 + 1.7(3.4)(.25)}\right)$$

$$= .925\left(\frac{2.4}{2.5}\right)$$

$$= .89$$

$$\underline{\underline{G_f = .89}}$$

Wind Calculations

• Gust Factors **East - West** (using previous)

$g_Q = g_v = 3.4$; $n_1 = .46$

$g_R = 3.97$

$\bar{z} = 92'$

$I_{\bar{z}} = .25$

$L_{\bar{z}} = 450.4$

$Q = .781$

$\bar{V}_{\bar{z}} = 76.8 \text{ mph}$

$N = 2.7$

$R_n = .074$

$\eta_h = 4.24$

$R_h = .21$

$$\eta_B = 4.6 n_1 B / \bar{V}_z = 4.6(.46)\left(\frac{223.33}{76.8}\right) = 6.15$$

$$R_B = \frac{1}{6.15} - \frac{1}{2(6.15)^2}(1 - e^{-2(6.15)}) = .16 - .013(.99) = .147$$

$$\eta_L = 15.4 n_1 L / \bar{V}_z = 15.4(.46)\frac{(304.25)}{76.8} = 28.06$$

$$R_L = \frac{1}{28.06} - \frac{1}{2(28.06)^2}(1 - e^{-2(28.06)}) = .035 - .00063(1) = .034$$

$$R = \sqrt{\frac{1}{\beta} R_n R_h R_B (.53 + .47 R_L)} = \sqrt{\frac{1}{.01}(.074)(.21)(.147)(.53 + .47(.034))} = .35$$

Wind Calculations

$$G_f = .925\left(\frac{1 + 1.7 I_{\bar{z}} \sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1 + 1.7 g_v I_{\bar{z}}}\right)$$

$$= .925\left(\frac{1 + 1.7(.25)\sqrt{(3.4)^2(.781)^2 + (3.97)^2(.85)^2}}{1 + 1.7(3.4)(.25)}\right)$$

$$= .925\left(\frac{2.27}{2.5}\right)$$

$$= .84$$

$$G_f = .84$$

Wind Calculations

- Determine velocity pressures q_z ; q_h

$q_z = .00256 K_z K_{zt} K_d V^2 I$
$= .00256 K_z 1.0(.85)(90)^2(1.15)$
$= 20.27$

$q_z = 20.27 K_z$

(see excel chart)

- Determine C_p

E-W
- Windward wall: $C_p = .8$
- Leeward wall ($L/B = 304/222.3 = 1.368$) $C_p = -.428$
- Side wall: $C_p = .7$

N-S
- Windward wall: $C_p = .8$
- Leeward wall: ($L/B = 222.3/304 = .73$) $C_p = -.5$
- Side wall: $C_p = -.7$

- Determine q_i

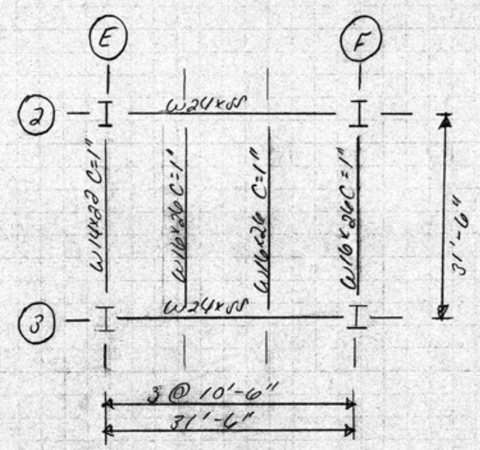
$q_i = q_h = 22.7$ psf

- Determine GC_{pi}

$GC_{pi} = +.18, -.18$ for enclosed structure

Gravity Calculations

Structural Spot Check:



• Slab Check - Level 6

Dead:

Floor Load + S.W.	47psf
Ceiling Partition	5psf
Suspended Mech Equipment	10psf
	62psf

Live:

Floor Load	60psf (reducible)

Live Load Reduction

$L = L_0\left(.25 + \frac{15}{\sqrt{K_{LL} A_T}}\right)$ $K_{LL} = 1$ (ASCE 7-05; Table 4-2)

$A_T = (10'\text{-}6'')(31'\text{-}6'') = 331 \text{ ft}^2$

$= 60\left(.25 + \frac{15}{\sqrt{1.0(331)}}\right)$

$= 64.4 \text{ psf}$

$\approx 60\text{psf}$ (no reduction)

Gravity Calculations

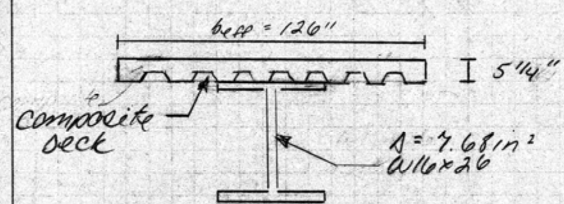
$w = (1.2)(62) + (1.6)(60)$
$w = 170.4 psf$

United Steel Deck catalog:

max unshored - 3 span =

uniform live load capacity = 285 psf > 152.4 psf ✓ okay

• Beam Check – Level 6 W16×26



f'c = 3000 psi 3/4" ⌀ shear studs
Fy = 50 ksi 2" rib

$w = 170.4 psf$
$w = \frac{(170.4 psf)(10.5 ft)}{1000} = 1.78 k/ft$

$M_u = \frac{wl^2}{8} = \frac{1.78(31.5)^2}{8}$
$M_u = 220.7 ft \cdot k$

$V_u = \frac{wl}{2} = \frac{1.78(31.5)}{2}$
$V_u = 28.05^k$

beff = min | span/4 = 31.5/4 = 7.88'
spacing = 10.5'

beff = 7.88(12) = 94.5"

Gravity Calculations

$C_c = .85(f'c)(b)(t)$
$= .85(3)(94.5)(3.25)$
$= 783.2^k$

$T_s = A_sF_y$
$= 7.68(50)$
$= 384^k$

Using AISC Steel Manual (Table 3-19)

Assume: $a = 1''$

$Y2 = 5\tfrac{1}{4}'' - \frac{a}{2} = 4.75''$

@ 7
$\phi M_n = 245$
$\Sigma Q_n = 96$

Check Assumption:

$a = \frac{\Sigma Q_n}{.85 f'c \cdot b_{eff}}$
$= \frac{96}{.85(3)(94.5)}$
$= .39 < 1$ ✓ okay

Find capacity

studs = 20 studs — Studs spaced ≤ 24" O.C.

$Q_n = 20(17.1^k)$ AISC Steel Manual (Table 3-21)
$= 342^k$

$a = \frac{342}{.85(3)(94.5)} = 2.41$

$Y2 = 5.25 - \frac{2.41}{2} = 3.9''$

$\phi M_n = 237.0^{ft \cdot k} > 220.7^{ft \cdot k}$ ✓ okay

Gravity Calculations

$$\Delta_{TOT} = \frac{5\left(\frac{22(10.5)}{1000}\right)(31.5)^4(1728)}{384(29000)I} = \frac{(31.5)(12)}{240} = 1.58$$

w/ 1" of camber

$I_{REQ} = 296.7\,in^4 < I_{ACT} = 301.1\,in^4$ ✓ okay

$$\Delta_{C.L.} = \frac{5\left(\frac{60(10.5)}{1000}\right)(31.5)^4(1728)}{384(29000)I} = \frac{(31.5)(12)}{360} = 1.05$$

w/ 1" camber

$I_{REQ} = 234.7\,in^4 < I_{ACT} = 301.1\,in^4$ ✓ okay

W16x26 okay for serviceability, deflection

- Girder spot W24x55

$f'_c = 3000\,psi$
$F_y = 50\,ksi$

3/4" Ø shear studs
2" rib
$A_s = 16.2\,in^2$

$P = 56.1^k$

$M_u = P(a)$
$= (56.1)(10.5)$
$= 589.1^{ft \cdot k}$

$V_u = 56.1^k$

$b_{eff} = \min \begin{cases} span/4 = \frac{31.5(12)}{4} = 94.5'' \\ spacing = 31.5(12) = 378'' \end{cases}$

$b_{eff} = 94.5''$

Gravity Calculations

$C_c = .85(3)(3.25)(94.5")$
$= 783.17^k$

$T_s = A_s F_y$
$= 16.2(50)$
$= 810^k$

Using AISC steel manual (Table 3-19)

Assume $a = 1"$

$Y2 = 5\frac{1}{4}" - \frac{a}{2} = 4.75"$

@7
$\phi M_n = 716^{ft \cdot k}$
$\Sigma Q_n = 208$

Check Assumption:

$a = \frac{\Sigma Q_n}{.85 f'_c \cdot b_{eff}}$
$= \frac{208}{.85(3)(94.5)}$
$= .84$

$Y2 = 5.25 - \frac{.84}{2} = 4.83"$

$\phi M_n = 717.38^{ft \cdot k} > 589.1^{ft \cdot k}$ ✓ okay

$\Delta_{TOT} = \frac{5\left(\frac{2552}{1000}\right)(31.5)^4(1728)}{384(29000)I} = \frac{(31.5)(12)}{240} = 1.56"$

$I_{REQ} = 1248.6 in^4 < I_{ACT} = 1350 in^4$ ✓ okay

$\Delta_{LL} = \frac{5\left(\frac{1260}{1000}\right)(31.5)^4(1728)}{384(29000)I} = \frac{(31.5)(12)}{360} = 1.05"$

$I_{REQ} = 916.6 in^4 < I_{ACT} = 1350 in^4$ ✓ okay

W24×55 okay for serviceability, deflection

Gravity Calculations

• Column spot Check Level 6 Col. Line E-2

Reduction for interior column

$L = L_o\left(.25 + \frac{15}{\sqrt{K_{LL} A_T}}\right)$ $K_{LL} = 4$ (ASCE 7-05; Table 4-2)

$= 60\left(.25 + \frac{15}{\sqrt{4(992.25)(3)}}\right) = 27.25\text{ psf} < .4L_o$

$L = .4(L_o)$
$= .4(60)$
$= 24\text{ psf}$

Loads Considered:

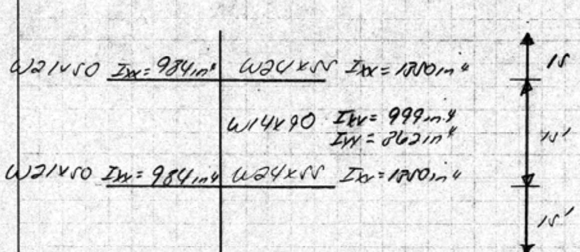
Floor Load (varies)
Ceiling Partition 5 psf
Susp. Mechanical 10 psf
Dist. Shaft Load

Shaft Load Distributed:

Line Load = .225 k/ft

.225(59.7 ft) = 13.4 k

Load to Column = $\frac{1}{2}(13.4)$
= 6.7 k



$r_y = 3.7$
$r_x/r_y = 1.66$
$A_g = 26.5\text{ in}^2$

$l_{beam} = 378\text{ in}$
$l_{col} = 180\text{ in}$

Gravity Calculations

Assume weak axis controls

$$G_A = \frac{2(362\,in^4/180)}{(984/378)+(1350/378)} = \frac{4.02}{6.17} = .65$$

$$G_B = G_A = .65$$

$$k = .725$$

$$\frac{KL_y}{r_y} = \frac{(.725)(15\times12)}{3.7} = 45.17 < 113 \quad \underline{\text{inelastic}}$$

$$4.71\left(\sqrt{\frac{29000}{50}}\right) = 113$$

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = \frac{3.14^2(29000)}{(45.17)^2} = 140.1\ ksi$$

$$F_{cr} = .658^{(F_y/F_e)}(F_y)$$
$$= .658^{(50/140.1)}(50)$$
$$= 43.06\ ksi$$

$$\phi P_n = (.9)(43.06)(26.5)$$
$$= 1026.9\ k$$

P_u (From excel chart)

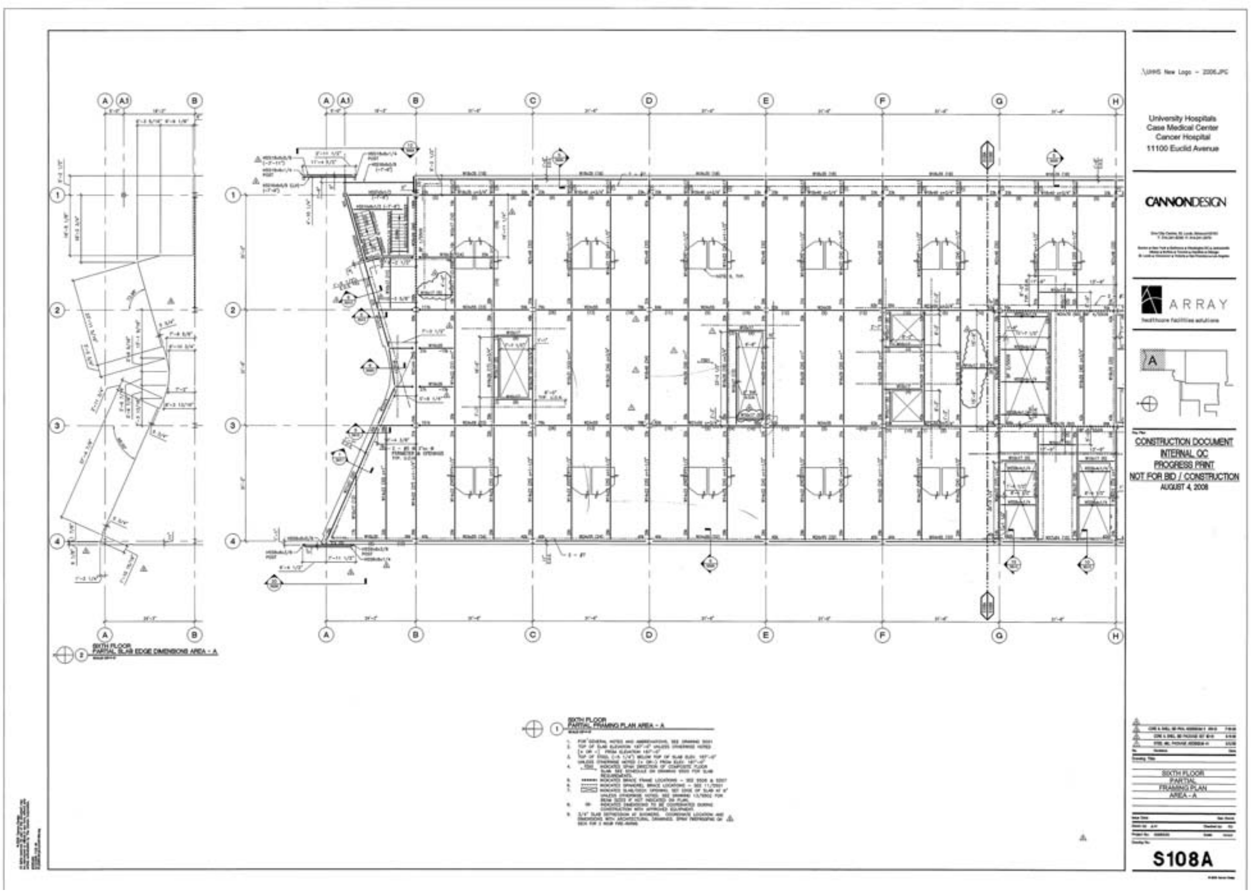
$$P_u = 596.5^k + 6.7^k$$

$$P_u = 603.2^k < \phi P_n = 1026.9^k \quad \checkmark \underline{\text{okay}}$$

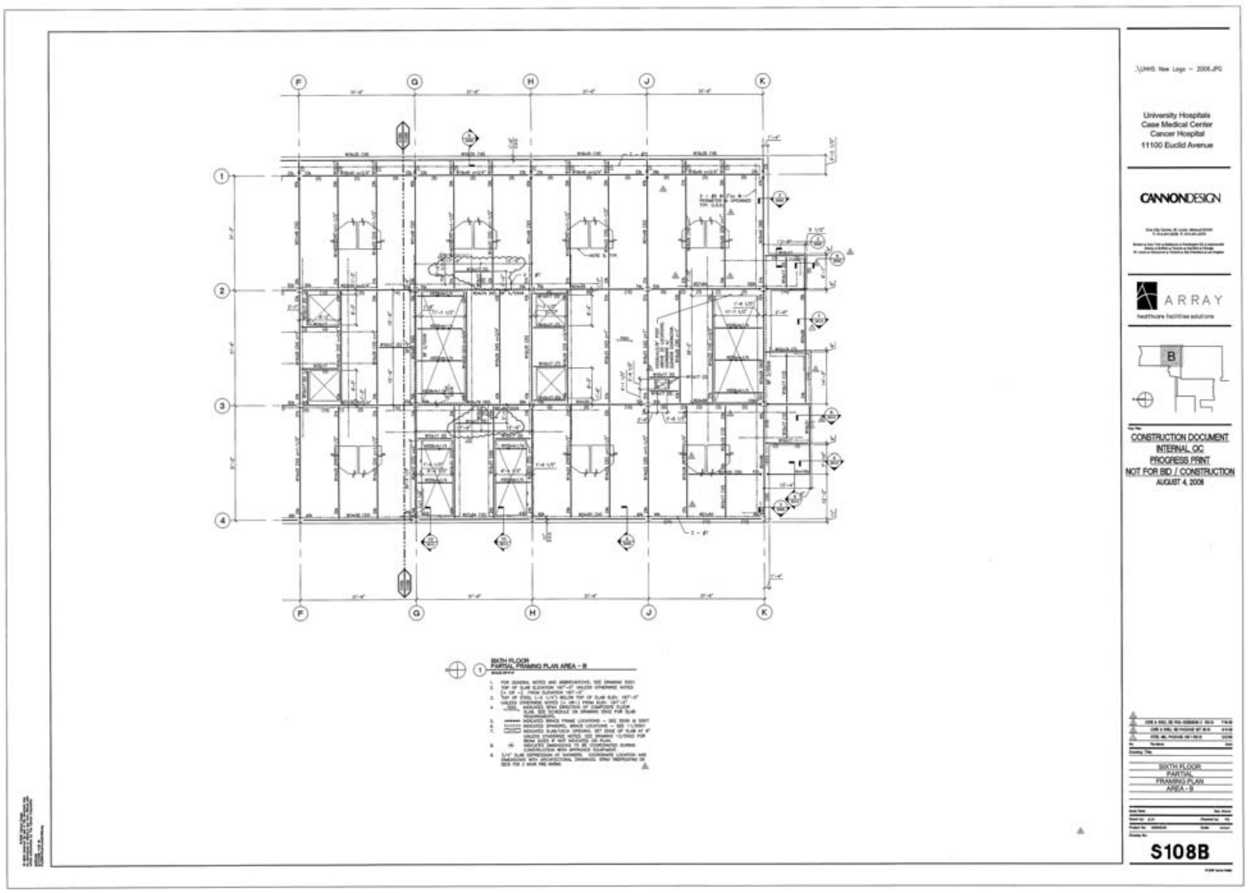
<u>Column meets criteria for axial loading</u>

Appendix B

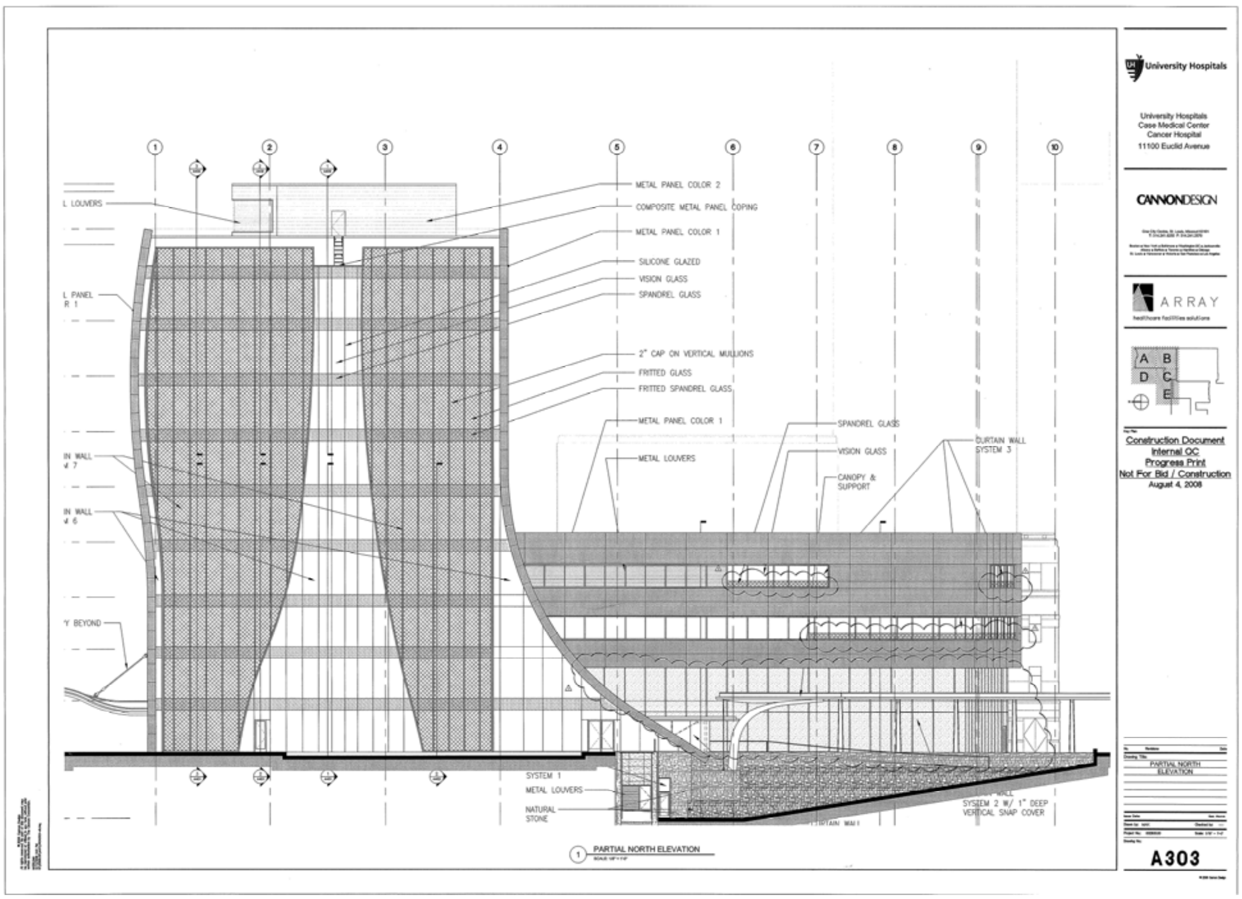
Sixth Floor Plan



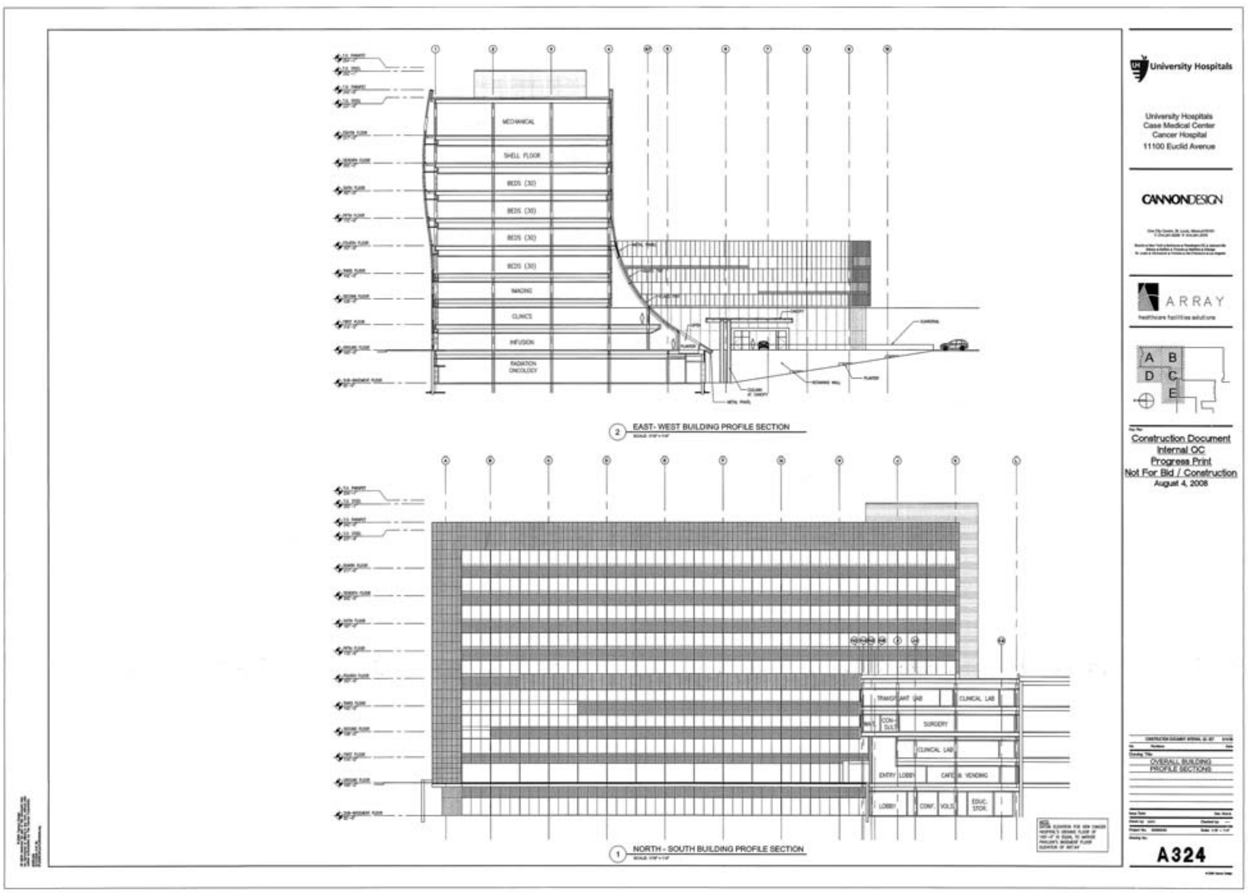
Sixth Floor Plan



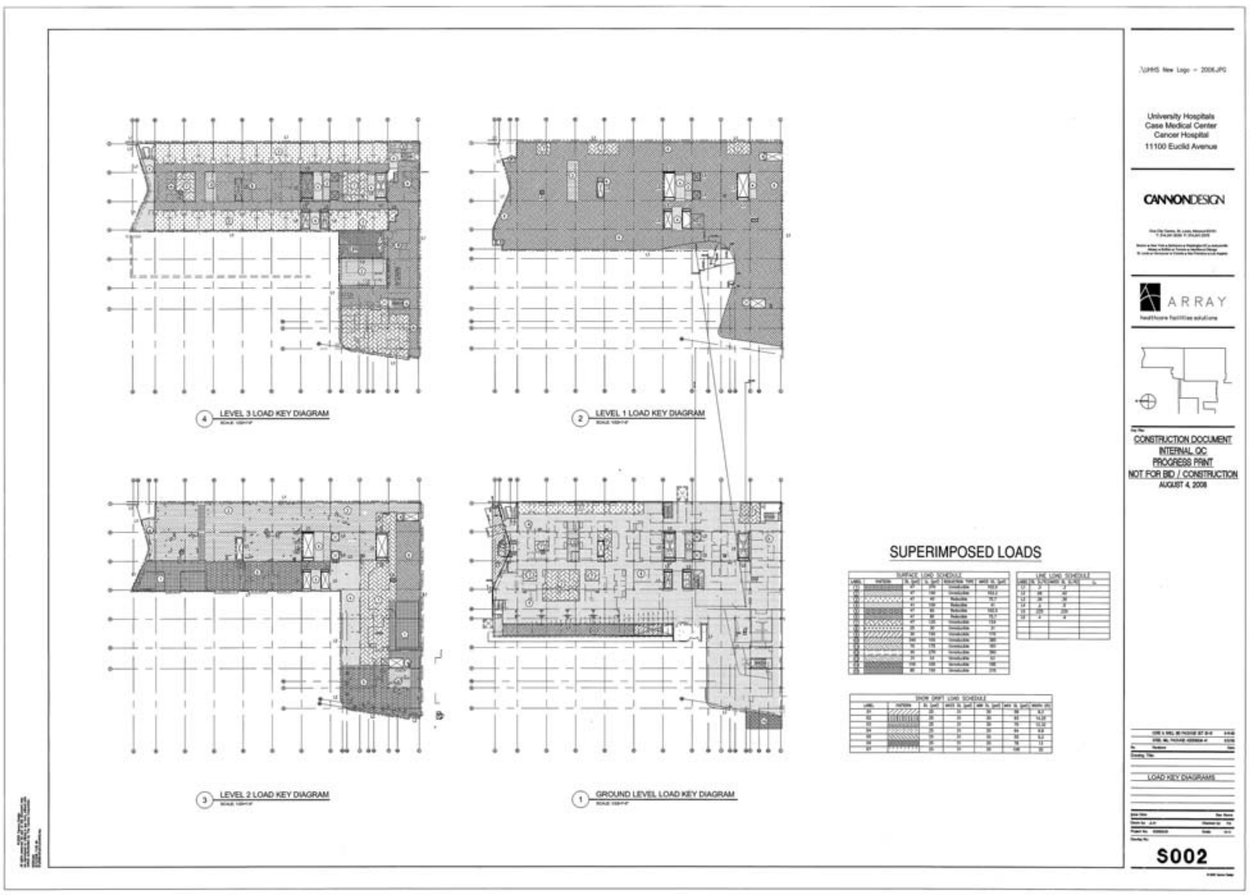
Elevations


METAL PANEL COLOR 2
COMPOSITE METAL PANEL COPING
METAL PANEL COLOR 1
SILICONE GLAZED
VISION GLASS
SPANDREL GLASS
2" CAP ON VERTICAL MULLIONS
FRITTED GLASS
FRITTED SPANDREL GLASS
METAL PANEL COLOR 1
METAL LOUVERS
SPANDREL GLASS
VISION GLASS
CANOPY & SUPPORT
CURTAIN WALL SYSTEM 3
SYSTEM 1
METAL LOUVERS
NATURAL STONE
PARTIAL NORTH ELEVATION
University Hospitals
University Hospitals
Case Medical Center
Cancer Hospital
11100 Euclid Avenue
CANNONDESIGN
ARRAY
Construction Document
Internal QC
Progress Print
Not For Bid / Construction
August 4, 2008
PARTIAL NORTH ELEVATION
A303

Elevations


MECHANICAL
SHELL FLOOR
BEDS (30)
BEDS (30)
BEDS (30)
BEDS (30)
IMAGING
CLINICS
INFUSION
RADIATION ONCOLOGY
EAST- WEST BUILDING PROFILE SECTION
TRANSPLANT LAB
CLINICAL LAB
SURGERY
CLINICAL LAB
ENTRY LOBBY
CAFE & VENDING
LOBBY
CONF
EDUC. STOR.
NORTH - SOUTH BUILDING PROFILE SECTION
University Hospitals
University Hospitals
Case Medical Center
Cancer Hospital
11100 Euclid Avenue
CANNONDESIGN
ARRAY
A B
D C
E
Construction Document
Internal QC
Progress Print
Not For Bid / Construction
August 4, 2008
OVERALL BUILDING PROFILE SECTIONS
A324

Loading Diagram



Loading Diagram