Technical Report 1: Structural Concepts/Structural Existing Conditions Report

SMILOW CANCER CENTER – YALE-NEW HAVEN HOSPITAL

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EXECUTIVE SUMMARY

After going through and determining certain types of lateral loads and quickly checking typical member sizes, it is evident that the designers of the Smilow Cancer Center were under a different set of restrictions when deciding on what type and size of members to use. From the member spot-checks done on a representative bay, it is obvious that gravity loading was not the controlling limit state for beam and column sizes. Some of the beams had capacities up to *six* times the factored load. One possible explanation is that lateral load was the controlling factor, or perhaps the structural engineer was under stricter limitations regarding the amount of tolerable deflection or vibration in the building. The space is, after all, a hospital where sensitive equipment operates and important surgical procedures take place.

Also, the set of assumptions made by the student in calculating gravity and lateral loads may have somewhat skewed the results, leading to discrepancies in member sizes. For example, in assuming the building footprint continues on as a rectangle past the fifth floor, the calculation of wind loads becomes much simpler. But, at the same time, that assumption neglects the significant effects of having horizontal as well as vertical irregularities in the building.

And finally, another possible cause for the variation in the loads calculated is the different set of codes used by the structural engineer and the student. The Connecticut State Building Code, the one used by the designer, is inherently much more specific to the region than the national ASCE Standard. Consequently, the state code would be more accurate in describing conditions around that area, whereas the national standard would be broader in scope and more general. Ultimately, the purpose of this report was for the student to become more familiar with the structure of the building. With a more intimate knowledge of the existing structure, the student can begin to develop ideas about an alternative structural system.

INTRODUCTION

The Structural Concepts/Structural Existing Conditions Report (Tech Report 1) is an overview of the current design for the Smilow Cancer Center in New Haven, CT. As a preliminary study of the building, Tech Report 1 contains a general description of the hospital's structural system. Two of the major lateral loads on the structure—wind and seismic—are calculated according to the 2005 version of the American Society of Civil Engineers (ASCE) Standard for Minimum Design Loads for Buildings and Other Structures. The calculated values are then compared with any available load information from the construction documents. Also, a simplified spot-check for typical structural members is included in the report; note that this check considers only gravity loads as a limit state.

SMILOW CANCER CENTER: Overview of Architecture, Project Information, and Code Analysis

Located in the middle of New Haven, the addition of the Smilow Cancer Hospital to the Yale-New Haven Hospital complex will feature a state-of-the-art building with the latest equipment for the treatment of the disease. The several areas of specialization are separated among the sixteen stories of the building, with the larger equipment (i.e. MRIs, ultrasound, operating rooms) housed primarily on the lower floors and the 112 inpatient rooms—all single rooms—starting on the eleventh floor. As for the exterior, the façade emulates that of the surrounding buildings in the complex with its glass and terra cotta curtain walls. For ease of installation, a unitized curtain wall panel system was used: the glass and terra cotta come in pre-installed panels ready to be attached to the structure. The hospital's roof is a combination of cast-in-place concrete roof deck and metal (steel) decking. The insulation and waterproofing are comprised of fully adhered thermoplastic polyolefin (TPO) sheet membrane roofing over mechanically attached insulation and cover board.

Construction on the 497,000 square foot project began in September of 2006 and is projected to be completed by early 2009. Overall cost is estimated at about $253 million. The architect is Shepley Bulfinch Richardson & Abbott of Boston, and Turner Construction Company is the construction manager [see "Building Statistics Part 1 for a full list of the primary project team]. Structural design was headed by Spiegel Zamecnik & Shah of New Haven, CT. The design of the building follows the 1999 Connecticut State Building Code which adopts mostly from "The BOCA National Building Code/1996." Other codes and standards used in the design of the structure are listed below:

- ASCE 7-02: Load combinations for consideration of future vertical expansion
- ACI 318-02: "Building Code Requirements for Structural Concrete"
- ACI 315-latest edition: "Details and Detailing of Concrete Reinforcement"
- AISC LRFD Steel Manual (2nd Edition): "LRFD Specification for Structural Steel Buildings"
- AISC 341-02: Seismic Provisions for Structural Steel Buildings
- Latest Specifications of the Steel Deck Institute
- "Specification for Welded Steel Wire Fabric for Concrete Reinforcement" (Latest Edition) by the Wire Reinforcement Institute

The hospital's structure and curtain walls were designed for wind loads using the Main Wind Force Resisting System (MWFRS) method and Components and Cladding (C&C) method as prepared by RWDI, Inc. of Guelph, Ontario. As for seismic loads, the structural engineer used the Equivalent Lateral Force Procedure (ELFP).

STRUCTURAL SYSTEM: Summary

The structural system of Smilow Cancer Center consists of a concrete slab on metal deck floor system supported on a steel framing system (moment, lateral braced, and regular gravity frames) and four reinforced concrete (RC) shear walls. On the first level, concrete beams of varying sizes run along three edges of the building. The floor slab and steel beams act in composite action with each other, while the moment frames and shear walls share the lateral load. The whole structure rests on a 4-foot thick mat slab foundation (the slab is 8 feet thick at shear wall locations). A relatively simple structure, the footprint of the building through the first five levels is almost square (210 ft. x 176 ft.). At the beginning of the seventh floor[1], however, the northeast "corner" of the building ends in a rooftop garden, and the rest of the building rises to the roof as an L-shape.

Normal weight concrete is used for the shear walls and the foundation, while lightweight concrete is used for the floor slabs. Concrete strength ranges from 3000 psi to 8000 psi depending on the location and use. All reinforcement is A615 Grade 60 steel. A range of steel W-shapes are used for the framing system, but all are of the standard A992 grade steel (F_y = 50 ksi). Additionally, Hollow Structural Shapes (HSS) conform to ASTM A500 Grade B, while all other steel shapes (i.e. plates, channels, etc.) conform to ASTM A36 (F_y = 36 ksi).

STRUCTURAL SYSTEM: Foundation + Columns

As mentioned above, the foundation for Smilow Cancer Center is a 4-foot thick mat slab with different types of column base plates down at the basement level. These columns vary from W-shapes, HSS, and even cruciform columns consisting of a wide flange plus two T-shapes[2]—all of which are encased in concrete. Some columns are regular reinforced concrete columns. Starting on the first floor, the columns continue up the structure as regular steel columns.

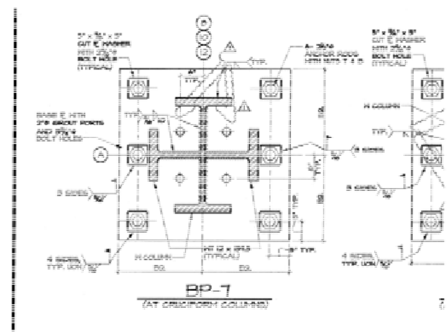


Figure 1: BP-7 Column Base Plate Detail

STRUCTURAL SYSTEM: Floor Slabs + Beams

The typical floor slab for Smilow Cancer Center is a 4-1/2" thick lightweight concrete slab on a 3" deep, galvanized, 18 gage composite steel floor deck with a 3 span minimum. Reinforcement consists of one layer of 6 x 6 – D4 x D4 welded bar mesh and top reinforcing bars. The slab is supported on steel framing and concrete shear walls at some locations. As per ASCE 05, the floor slabs are considered as rigid diaphragms when taking into account lateral loads.

The hospital's typical bay[3] is a 30 ft. x 30 ft. square with W-shape columns at the corners, W24 girders along the perimeter, and two W18/21/24 beams spaced evenly at 10 ft. on-center. As discussed in the following section, most of the beams frame into simple gravity columns, while moment frames and shear walls are dispersed throughout the structure to effectively resist lateral loads.

[1] Smilow Cancer Center does not have floors labeled 6th or 13th for superstition purposes.
[2] See Figure 1: BP-7 Plan Detail.
[3] See Figure 2: Typical Bay Plan.

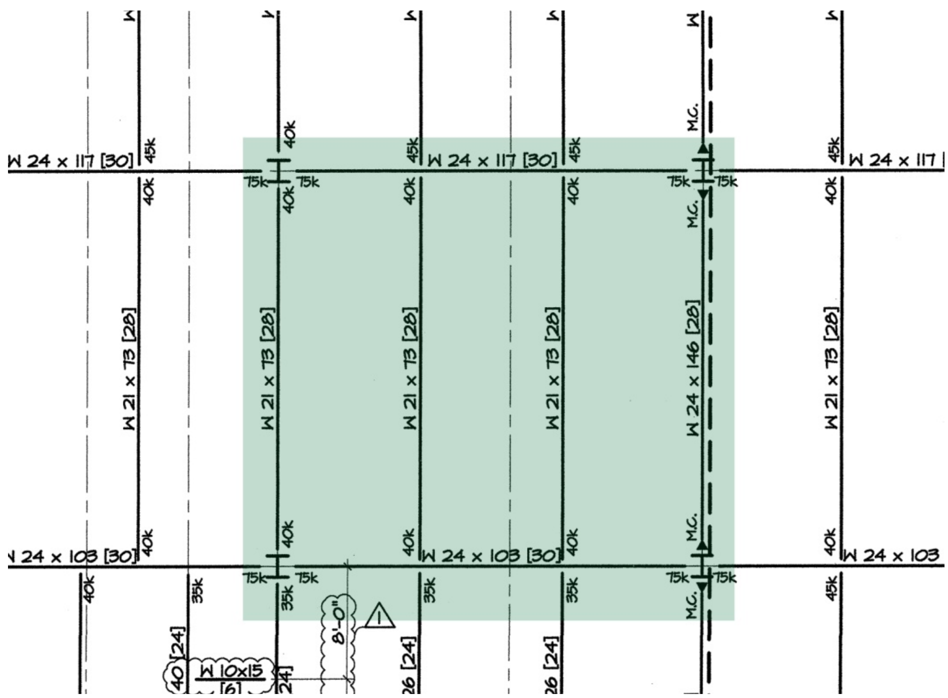


Figure 2: Typical Bay Plan – The typical 30’x30’ bay is shown here highlighted in green.

STRUCTURAL SYSTEM: Lateral Resisting System

Smilow Cancer Center’s lateral resisting system is a combination of six primary moment frames, several smaller lateral braced frames on the roof, and four C-shaped RC shear walls. Four of the six main moment frames are located at the edges of the building, while the remaining two run along the east-west direction at approximately one-third points of the building’s length[4]. The four shear walls are all located towards the southeast quadrant of the building, strategically placed around central elevator and mechanical openings. All four shear walls rise up to either the sixteenth or seventeenth floor, ending where the lateral braced frames of the roof begin. Refer to page A1 of the Appendix for sketches of the shear wall sections.

[4] See Figures 3a & 3b: Moment Frame Locations.

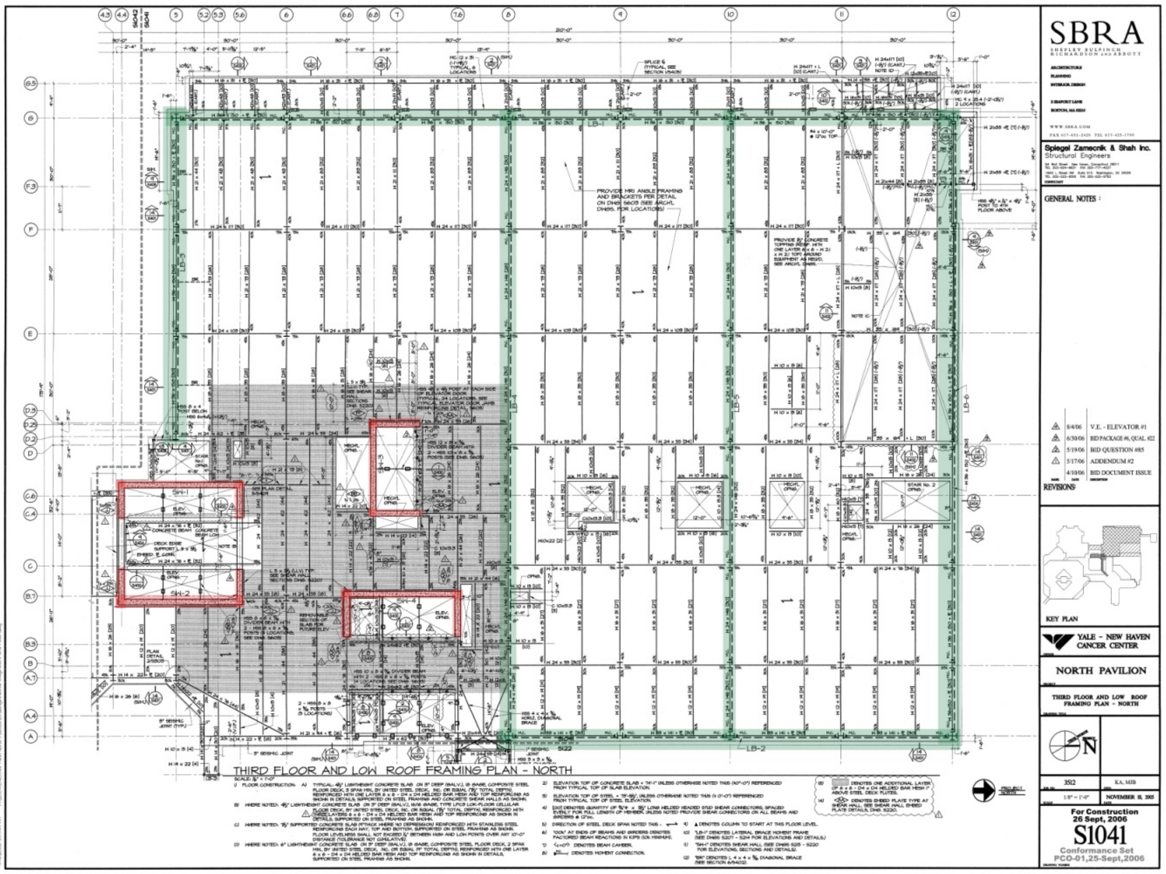


Figure 3a: Typical Framing Plan for Levels 1-5. Green denotes Moment Frames; Red denotes Shear Walls.

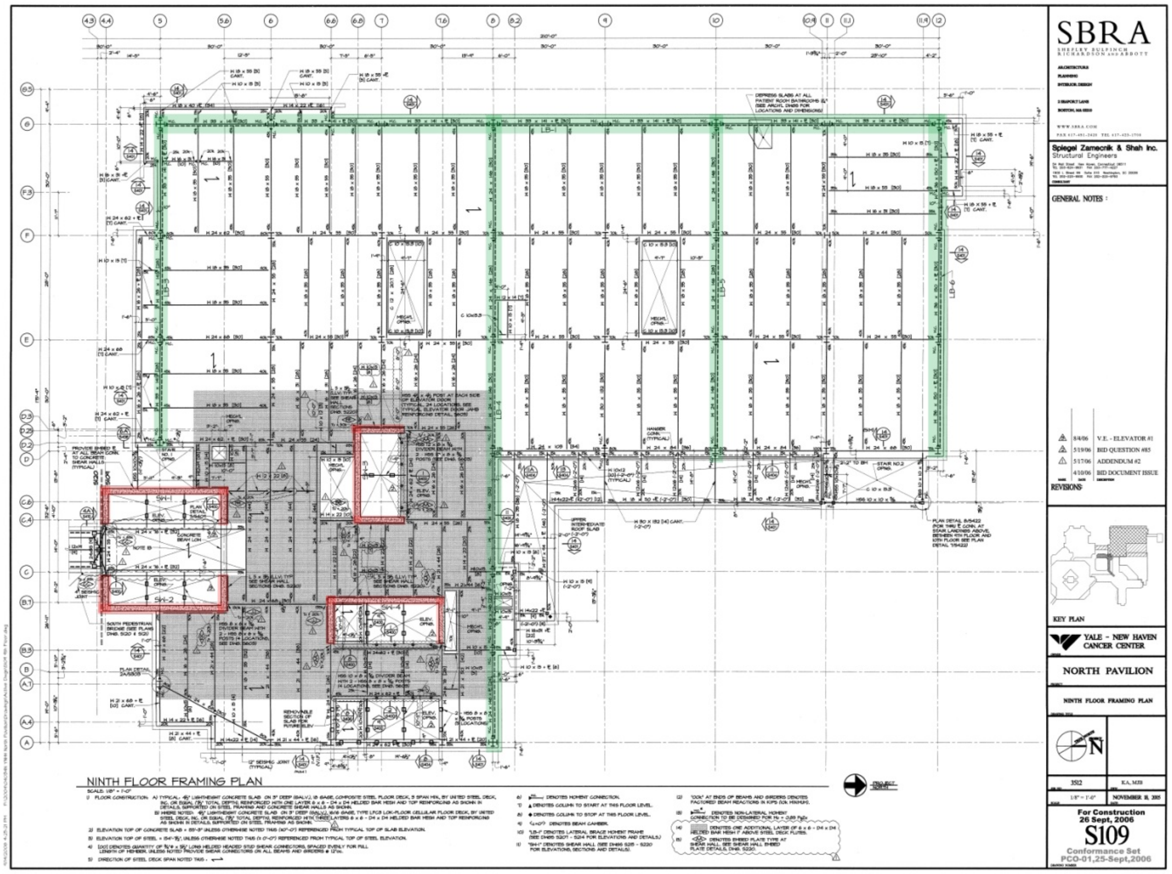


Figure 4: Typical Framing Plan for Levels 7-17. Green denotes Moment Frames; Red denotes Shear Walls.

BUILDING DESIGN LOADS: Gravity Loads

For the student's preliminary calculations, gravity loads were determined as per ASCE 7-05, 13th Edition of the AISC Steel Manual, other relevant publications, and a few assumptions on the student's part. Construction documents (CD) also provided some insight into code compliant loads. Table 1 below summarizes loads by type and material.

Table 1: Gravity Loads

FLOOR LOADS			
Type	**Material/Occupancy**	**Load**	**Reference**
Dead Load	Normal Weight Concrete	145 pcf	[Assumed]
	Light Weight Concrete	110 pcf	[Assumed]
	Steel	per shape	AISC 13th Edition
	Partitions	20 psf	[Assumed]
	Superimposed	10 psf	CD: S605 – S606
	Common Areas	100 psf	CD: S605 – S606

Live Load	Lobbies	100 psf	CD: S605 – S606
	Corridors (1F)	100 psf	ASCE 7-05
	Corridors (Above 1F)	80 psf	ASCE 7-05
	Operating Rooms	80 psf	CD: S605 – S606
	Exam Rooms	80 psf	CD: S605 – S606
	Mechanical	150 psf	CD: S605 – S606
	Stairs	100 psf	CD: S605 – S606
ROOF LOADS			
Dead Load	Normal Weight Concrete	145 pcf	[Assumed]
	Light Weight Concrete	110 pcf	[Assumed]
	Steel	per shape	AISC 13th Edition
	Superimposed	25 psf	CD: S605 – S606
Live Load	Roof Live Load	33 PSF	CD: S605 – S606

Note: Snow and rain loads were not a requirement for Tech Report 1 and as such were not included in the load calculations.

BUILDING DESIGN LOADS: Lateral Loads

As per ASCE 7-08, lateral loads—specifically wind and seismic—were calculated to compare against design loads used by the structural engineer. The methods used for calculating wind and seismic loads were the Main Wind Force Resisting System (MWFRS) and the Equivalent Lateral Force Procedure (ELFP), respectively. Other references include IBC 2006 and the United States Geological Service website, usgs.gov. Refer to the following spreadsheets for a summary of wind and seismic load calculations. Also, pressure (wind) and story shear (seismic) diagrams are included in pages A4-A7 of the Appendix.

WIND LOAD CALCULATIONS

Basic Wind Speed (mph) *Fig. 6-1*	V=	120
Wind Directionality Factor *Table 6-4*	K_d=	0.85
Occupancy Category, IBC		IV
Importance Factor *Table 6-1*	I=	1.15
Exposure Category *6.5.6*	EC=	B

Topographic Factor *6.5.7.1*	K_{zt}=	1		
Velocity Coefficient *Table 6-3*	K_z	[see table]		
Velocity Pressure *Eq. 6-15*	q_z	[see table]		
Building Frequency *C6-15 (Steel MRF)*	n_1=	0.282 *(flexible)*	<1	
Peak Factors *6.5.8.2*	g_Q=g_v=	3.4		
Peak Factor *Eq. 6-9*	g_R=	3.88		
Turbulence Factor *6.5.8.1*	z=	141	> z_{min} = 30'	
Intensity of Turbulence *Eq. 6-5*	I_z=	0.236	c = 0.3 *Table 6-2*	
Integral Length *Eq. 6-7*	L_z=	519	l = 320' *TABLE 6-2*	ϵ =1/3.0 *TABLE 6-2*
Background Response *Eq. 6-6*	Q=	0.806		
Mean Wind Speed *Eq. 6-14*	V_z=	113.9	α = 0.25 *TABLE 6-2*	b = 0.45 *TABLE 6-2*

Reduced Frequency	$N=$	1.28		
Eq. 6-12				
Eq. 6-11	$R_n=$	0.115		
Eq. 6-13	$R_h=$	0.305	$\eta=$	2.67
Eq. 6-13	$R_B=$	0.386	$\eta=$	1.93
Eq. 6-13	$R_L=$	0.116	$\eta=$	8.11
Resonant Response	$R_{(N\text{-}S)}=$	0.089	$\beta=$	1
Eq. 6-10	$R_{(E\text{-}W)}=$	0.054		
Gust Effect Factor	$G_f=$	0.83	*(N-S)*	
Eq. 6-8	$G_f=$	0.82	*(E-W)*	

ENCLOSED?	YES
LOW-RISE?	YES
RIGID?	NO

External Pressure Coefficients *(Fig. 6-6)*

Windward	$C_p=$	0.8		
Leeward (N-S)	$C_p=$	-0.45	L/B=	1.25
Leeward (E-W)	$C_p=$	-0.5	L/B=	0.80

Table 2a: Wind Pressure N-S

North-South Wind Direction

(short side of building)

Height=	235	ft.
B=	170	ft.
L=	213	ft.

Location	Height (ft.)	K_z	q_z	p_z (psf)	P_z (kips)	Overturning Moment, M_o (ft-kips)
Windward	30	0.70	25.2	16.6	42.45	1273.59
	40	0.76	27.4	18.1	46.09	1843.68
	50	0.81	29.2	19.3	49.12	2456.22
	60	0.85	30.6	20.2	51.55	3093.01
	70	0.89	32.1	21.2	53.98	3778.33
	80	0.93	33.5	22.1	56.40	4512.16
	90	0.96	34.6	22.8	58.22	5239.93
	100	0.99	35.7	23.5	60.04	6004.08
	120	1.04	37.5	24.7	63.07	7568.78
	140	1.09	39.3	25.9	66.11	9254.78
	160	1.13	40.7	26.9	68.53	10965.03
	180	1.17	42.2	27.8	70.96	12772.32
	200	1.2	43.2	28.5	72.78	14555.35
	235	1.26	45.4	30.0	76.42	17957.67
Leeward	ALL	1.26	45.4	-16.9	-42.98	-5050.59

K_h=	**1.26**
q_h=	**45.4**

Eq. 6-15

Table 2b: Wind Pressure E-W

East-West Wind Direction
(long side of building)

Height=	235	ft.
B=	213	ft.
L=	170	ft.

Location	Height (ft.)	K_z	q_z	p_z (psf)	P_z (kips)	Overturning Moment, M_o (ft-kips)
Windward	30	0.70	25.2	16.6	53.05	1591.59
	40	0.76	27.4	18.0	57.60	2304.02
	50	0.81	29.2	19.2	61.39	3069.50
	60	0.85	30.6	20.2	64.42	3865.29
	70	0.89	32.1	21.1	67.45	4721.72
	80	0.93	33.5	22.1	70.48	5638.78
	90	0.96	34.6	22.8	72.76	6548.26
	100	0.99	35.7	23.5	75.03	7503.22
	120	1.04	37.5	24.7	78.82	9458.60
	140	1.09	39.3	25.9	82.61	11565.56
	160	1.13	40.7	26.8	85.64	13702.84
	180	1.17	42.2	27.8	88.67	15961.38
	200	1.2	43.2	28.5	90.95	18189.61
	235	1.26	45.4	29.9	95.50	22441.43
Leeward	ALL	1.26	45.4	-18.7	-59.68	-7012.95

K_h=	**1.26**	
q_h=	**45.4**	*Eq. 6-15*

Notes:

1. Building footprint is assumed to be rectangular throughout height of structure.

2. All equations, tables, and sections cited are from ASCE/SEI 7-05.

SEISMIC LOAD CALCULATIONS

Table 3a: SEISMIC DESIGN CRITERIA	VALUE	SOURCE
Occupancy Category	IV	*IBC Table 1604.5*
Importance Factor	1.5	*Table 11.5-1*
Site Class	B	*usgs.gov*
Spectral Acceleration for Short Periods, S_s	0.243	*usgs.gov*
Spectral Acceleration for 1 Sec. Periods, S_1	0.062	*usgs.gov*
Site Coefficient, F_a	1	*usgs.gov*
Site Coefficient, F_v	1	*usgs.gov*
Seismic Design Category (SDC)	A	*Table 11.6-1,2*
Response Modification Coefficient, R	3.5	*Table 12.2-1*
(Ordinary Steel Moment Frame)		
$S_{MS} = F_a * S_s$	0.243	*Eq. 11.4-1*
$S_{M1} = F_v * S_1$	0.062	*Eq. 11.4-2*
$S_{DS} = (2/3) * S_{MS}$	0.162	*Eq. 11.4-3*
$S_{D1} = (2/3) * S_{M1}$	0.041	*Eq. 11.4-4*
C_s	0.008	*< 0.01*
	∴ C_s =	**0.01**

Table 3b: BUILDING CRITERIA	VALUE	SOURCE
$T_a = C_t * h_n^x$	2.21	*Eq. 12.8-7*
$T_0 = 0.2 * (S_{D1}/S_{DS})$	0.05	*11.4.5*
$T_s = S_{D1}/S_{DS}$	0.26	*11.4.5*
C_t	0.03	*Table 12.8-2*
x	0.80	*Table 12.8-2*
h_n (ft.)	235.00	
T_L	6.00	*Fig. 22-15*

T_a	**<**	**T_L**

Table 3c: **EFFECTIVE SEISMIC WEIGHT, W**			
Dead Load			
Superimposed	lbs/ft	Area (ft^2)	Total Weight (kips)
First Level	35	36210	1267.35
Second Level	25	36210	905.25
Third Level	25	36210	905.25
Fourth Level	35	36210	1267.35
Fifth Level	25	36210	905.25
Seventh Level	35	27210	952.35
Intermediate Roof	575	9000	5175
Eighth Level	25	27210	680.25
Ninth Level	25	27210	680.25
Tenth Level	25	27210	680.25
Eleventh Level	25	27210	680.25
Twelfth Level	25	27210	680.25
Fourteenth Level	25	27210	680.25
Fifteenth Level	25	27210	680.25
Sixteenth Level	25	27210	680.25
Seventeenth Level	25	27210	680.25
Roof	25	680	17.00
[see S605-S606 of drawings for typical load diagrams]			
Special Equipment Loads			Weight (kips)
MRI Signa Twinspeed 1.5T			13.11
MRI Signa Signa Excite 3.0T			23.12
CT - Lightspeed PRO 16			4.20
PET/CT Discovery ST			7.92
RAD - Revolution XR/D			1.39
RAD/Flouro - Precision 500D			3.44
Gamma - Infinia w/ Hawkeye Option			6.39
Emergency Diesel Generators			35.00
[loads taken from S607 of drawings]			
Storage Space: 25% of LL		Area (ft^2)	Weight (kips)
First Level		600	18.75
Second Level		420	13.13
Third Level		404	12.63
Fourth Level	**0.25x125 psf**	328	10.25
Fifth Level		0	0.00
Seventh Level		1060	33.13
Eighth Level		200	6.25

Ninth Level	132	4.13
Tenth Level	132	4.13
Eleventh Level	240	7.50
Twelfth Level	240	7.50
Fourteenth Level	210	6.56
Fifteenth Level	210	6.56
Sixteenth Level	0	0.00
Seventeenth Level	0	0.00
Roof	0	0.00

[see S605-S606 of drawings for typical load diagrams]

Partition Load		Area (ft^2)	Weight (kips)
First Level		36210	724.2
Second Level		36210	724.2
Third Level		36210	724.2
Fourth Level		36210	724.2
Fifth Level		36210	724.2
Seventh Level		27210	544.2
Eighth Level		27210	544.2
Ninth Level	**20 psf [assumed]**	27210	544.2
Tenth Level		27210	544.2
Eleventh Level		27210	544.2
Twelfth Level		27210	544.2
Fourteenth Level		27210	544.2
Fifteenth Level		27210	544.2
Sixteenth Level		27210	544.2
Seventeenth Level		27210	544.2
Roof		680	13.60

**NOTE: Snow load for area = 30 psf ∴ no 20% addition to effective seismic weight*

TOTAL EFFECTIVE SEISMIC WEIGHT, W =	**26818.72**	**kips**
SEISMIC BASE SHEAR, V =	**268.19**	**kips**

Linear interpolation for exponent *k*:

	Tmin	T_a	Tmax
	0.5	2.21	2.5
k	1	1.86	2

∴ k = 1.86

Table 3d: **Distribution of Seismic Forces**

**NOTE: In determining the vertical distribution of seismic forces, levels 7-17 are assumed to have 75% of the weight of levels 1-5. See Appendix Page A6 for hand calculation.*

Total Effective Seismic Weight ≈ 27000 kips

	Height (ft)	Story Weight, w (kips)	Vertical Distribution Factor, C_{vx}
First Level	0	2157.55	0.000
Second Level	15	2157.55	1.000
Third Level	30.5	2157.55	0.789
Fourth Level	45.5	2157.55	0.624
Fifth Level	60.5	2157.55	0.515
Seventh Level	80.5	1618.16	0.396
Eighth Level	95.5	1618.16	0.353
Ninth Level	110.5	1618.16	0.316
Tenth Level	125.5	1618.16	0.286
Eleventh Level	140.5	1618.16	0.261
Twelfth Level	155.5	1618.16	0.240
Fourteenth Level	170.5	1618.16	0.221
Fifteenth Level	185.5	1618.16	0.206
Sixteenth Level	200.5	1618.16	0.192
Seventeenth Level	217.5	1618.16	0.183
Roof	232	30.60	0.004

Table 3d (continued)

Lateral Seismic Force, F_x (kips)	Story Shear, V_x (kips)	Overturning Moment, M_o (ft-kips)
0.00	0.00	0.00
268.19	268.19	4022.81
211.65	479.84	6455.27
167.39	647.23	7616.24
138.02	785.25	8350.35
106.30	891.54	8556.75
94.56	986.10	9030.63
84.81	1070.92	9371.84
76.72	1147.64	9628.89

69.96	1217.60	9829.52
64.25	1281.85	9990.56
59.37	1341.22	10122.76
55.17	1396.39	10233.29
51.51	1447.89	10327.14
48.98	1496.87	10653.26
1.04	1497.92	241.36

MEMBER SPOT-CHECK

Refer to pages A8-A12 of Appendix for member spot-check calculations and interpretation.

CONCLUSION

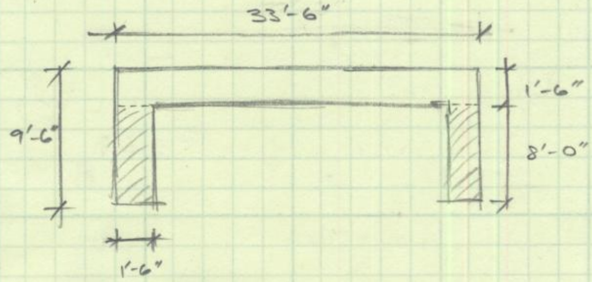
Being one of the more comprehensive cancer care facilities in the New England area, the Smilow Cancer Center features a relatively simple yet elegant and efficient structural system. The near-square bays (approx. 30 ft by 30 ft) form a fairly orderly grid towards the west end of the plan, while the southeast quadrant of the building boasts four shear walls that shoot straight up through the building, providing part of the lateral resistance of the structure. The rest of the lateral load is handled by the six major moment frames located around the perimeter of the building and across the center.

Calculating the wind and seismic loads of the building according to ASCE 7-05 and testing typical member sizes reveal that gravity loading most likely does *not* control the design of the members. More than likely it is the case that lateral loading or even deflection and vibration limitations control the size of the members used. It is understandable that a hospital with its operating rooms and critical machinery would have very strict guidelines regarding the amount of vibration allowed. Furthermore, the importance of a hospital as a safe shelter during times of emergency definitely warrants the apparent over-sizing of its structural members.

LIST OF FIGURES & TABLES:

SMILOW CANCER HOSPITAL | SHEAR WALL DIMENSIONS | 26 SEPTEMBER 2008 | A1

□ SW-1 & SW-2



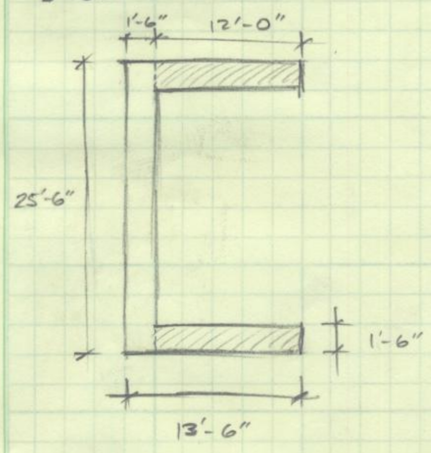
* SEE S216 - S219 FOR ACTUAL DRAWINGS

▷ CROSS-SECTIONAL AREA, A_{cs}:

$$A_{cs} = (33.5' \times 1.5') + 2(1.5' \times 8.0')$$

$$\boxed{A_{cs} = 74.25 \text{ ft}^2}$$

□ SW-3

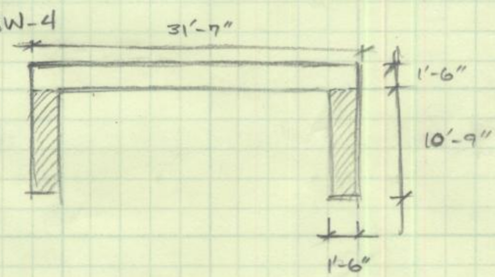


▷ CROSS-SECTIONAL AREA, A_{cs}:

$$A_{cs} = (1.5' \times 25.5') + 2(12.0' \times 1.5')$$

$$\boxed{A_{cs} = 74.25 \text{ ft}^2}$$

□ SW-4



▷ CROSS-SECTIONAL AREA, A_{cs}:

$$A_{cs} = (31.58' \times 1.5') + 2(1.5' \times 10.75')$$

$$\boxed{A_{cs} = 79.62 \text{ ft}^2}$$

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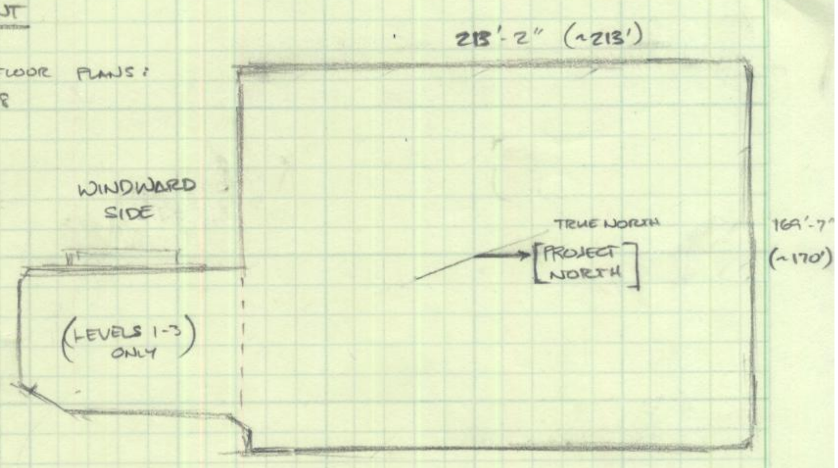
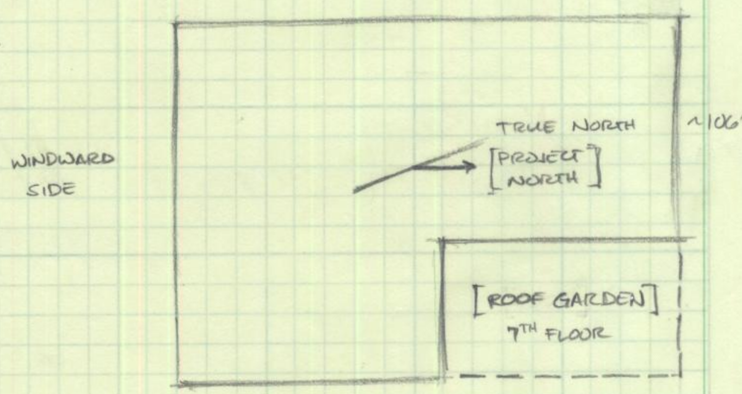
BUILDING FOOTPRINT

COMPREHENSIVE FLOOR PLANS:
AC101 – AC118

LEVELS 1 – 5:



LEVELS 7-18:



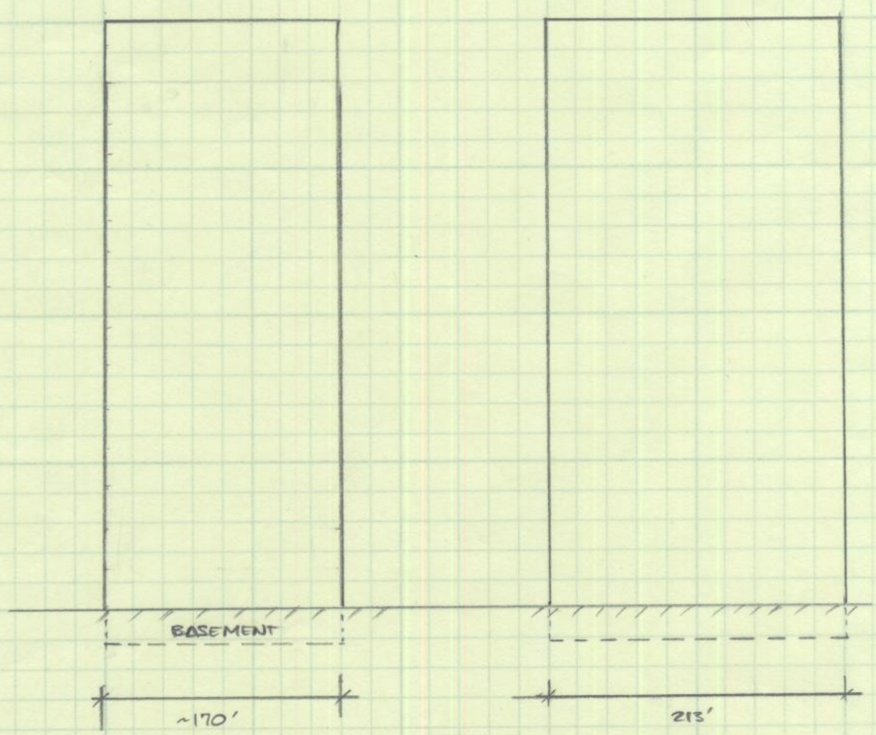
* NOTE: SMILOW CANCER HOSPITAL DOES NOT HAVE FLOORS LABELED 6TH & 13TH FOR SUPERSTITION PURPOSES.

BUILDING ELEVATION

OVERALL HEIGHT : ~235 ft

FLOOR LEVEL	FLOOR HEIGHT
1ST	15'-2"
2ND	15'-8"
3RD	15'-6"
4TH	15'
5TH	20'-6"
7TH	15'
8TH	15'
9TH	15'
10TH	15'
11TH	15'
12TH	15'
14TH	15'
15TH	15'
16TH	~16'
17TH	~14'
18TH	[ROOF]

* NOTE: FOR TECH REPORT 1, BUILDING IS ASSUMED TO HAVE A RECTANGULAR FOOTPRINT ALL THE WAY UP TO THE TOP FLOOR. (i.e. ROOF GARDEN ON 7TH FLOOR IS IGNORED).



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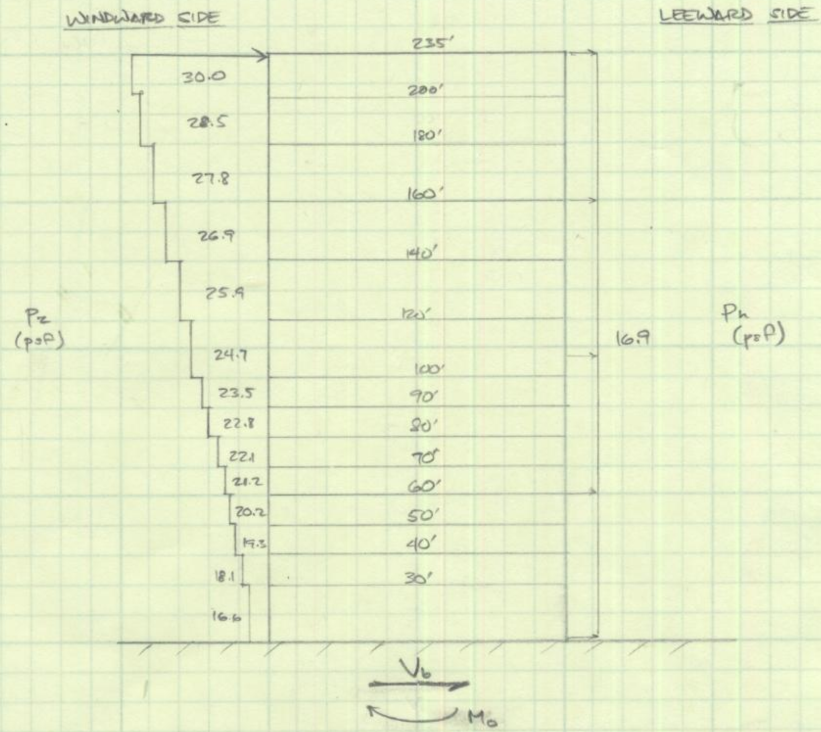
WIND LOAD PRESSURE DIAGRAM:



NORTH-SOUTH WIND DIRECTION

V_b = 878.70 k

M_o = 106,325.53 ft-k

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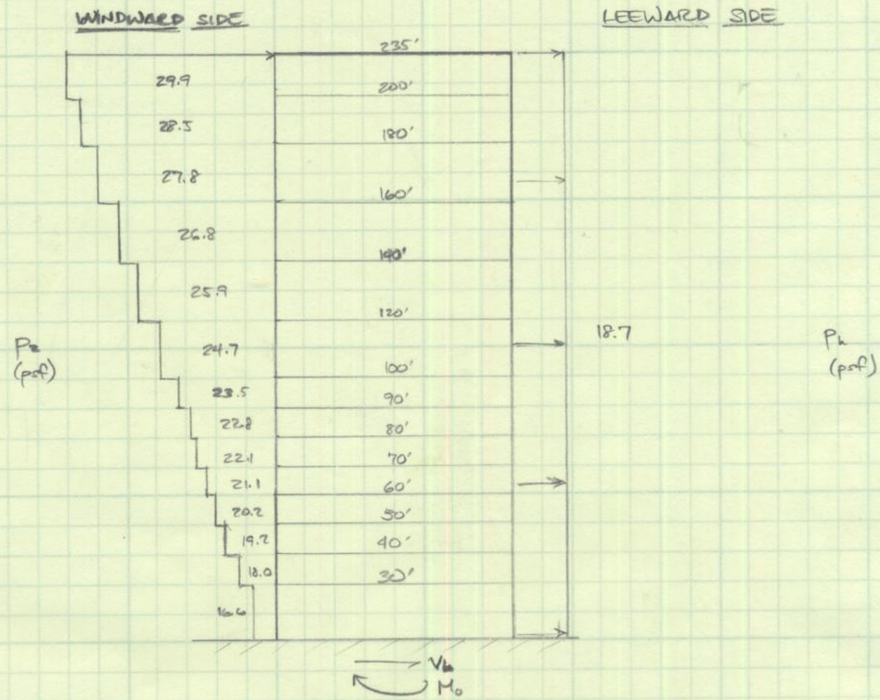
WIND LOAD PRESSURE DIAGRAM:



EAST-WEST WIND DIRECTION

$V_b = 1044.4^{k}$

$M_o = 126{,}561.81^{ft\text{-}k}$

▫ SEISMIC WEIGHT DISTRIBUTION :

$5L + 10U + R = 27000^k$

$[\text{LOWER FLOORS}] + [\text{UPPER FLOORS}] + [\text{ROOF}] = 27000^k$

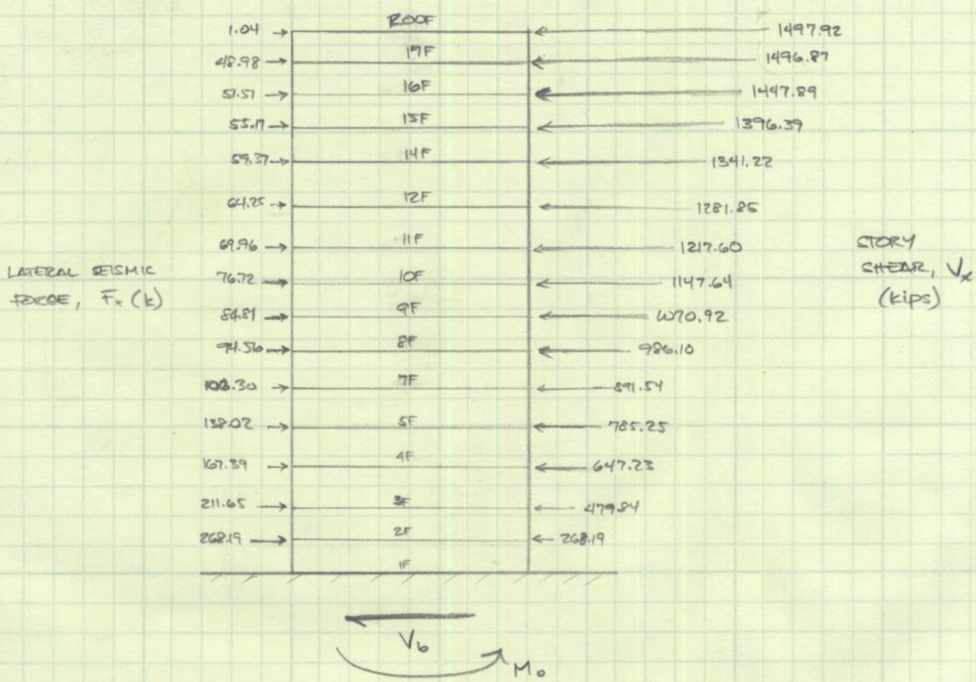
▹ $0.75L = U$

$5L + 10(0.75L) + 30.6 = 27000$

▸ $L = 2157.55^k$

▸ $U = 1618.16^k$

SEISMIC LOAD STORY SHEAR DIAGRAM:



V_b = 268.19 k

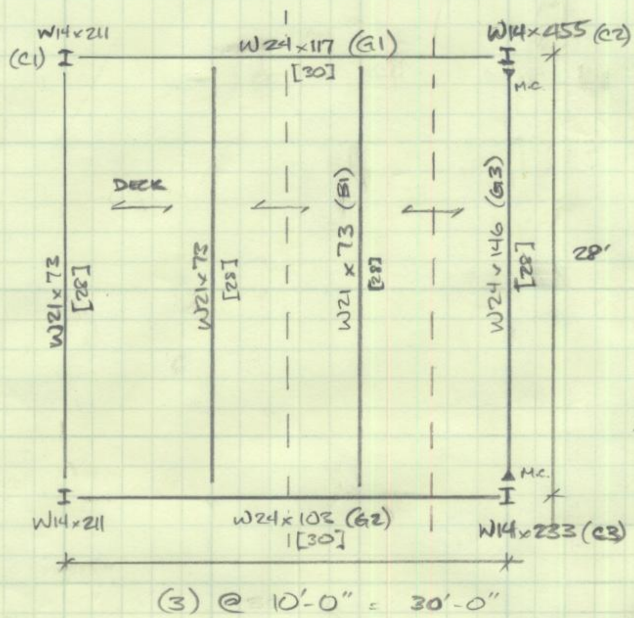
M_o = 124430.67 ft-k

* GRAVITY LOADS ONLY

TYPICAL BAY:

THIRD FLOOR INTERIOR BAY (EXAM ROOMS, OFFICES, LABS, OPERATING ROOMS)

[SEE S1041 & S605 EXCERPTS]



▷ LOADS:

LIVE LOAD = 80psf
SI. DEAD LOAD = 25psf

FACTORED LOAD:

$w_u = 1.2D + 1.6L$
$1.2(25) + 1.6(80)$

$w_u = 158$ psf
$w_u = 1580$ plf

▷ SHEAR STUDS:

1 SHEAR STUD per FOOT (TYP.)

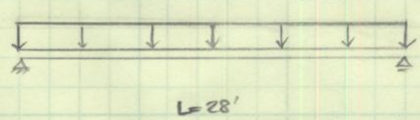
3/4" ⌀ STUDS
1 STRONG STUD per RIB

▷ CONCRETE

LIGHTWEIGHT CONC.
$f'_c = 4$ ksi

B1: W21x73

$w_u = 1580$ lbs/ft $= 1.58$ k/ft



$$M_u = \frac{w_u L^2}{8} = \frac{1.58\,(28)^2}{8} = 154.84 \text{ ft-k}$$

FROM STEEL MANUAL: (TABLE 3-21) $Q_n = 21.2^k$ → $\Sigma Q_n = 21.2^k$/STUD (14 STUDS)

$\Sigma Q_n = 296.8^k$

CONT'D→

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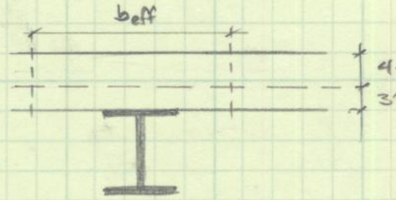
B1 (CONT'D)



$b_{eff} \le \frac{10(12)}{2} = 60$ in

$\le \frac{28(12)}{8} = 42$ in

$$a = \frac{\Sigma Q_n}{0.85 f'_c b} = \frac{296.8^k}{0.85(4)(42)} = 2.08 \text{ in}$$

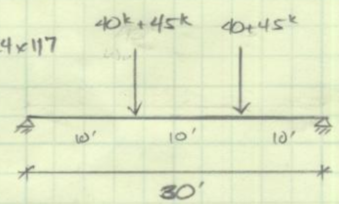
$$Y_2 = Y_{con} - \frac{a}{2} = 7.5'' - \frac{2.08 \text{ in}}{2}$$

$$\approx 6.5''$$

FROM STEEL MANUAL TABLE 3-19 : $\phi M_p = 866$ ft-k >> $M_u = 155$ ft-k

* GRAVITY LOAD OBVIOUSLY DID NOT CONTROL IN THE DESIGN OF THIS MEMBER. PERHAPS DEFLECTION CONTROLLED, OR MAYBE THE DESIGNER USED ONE SHEAR STUD PER FOOT AS A RULE FOR SIMPLICITY.

G1: W24x117

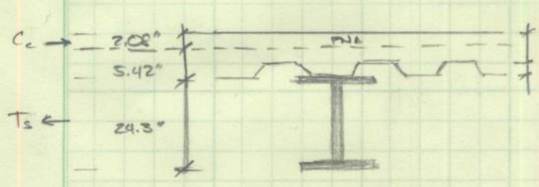


$M_u = Pa = 85(10)$

$M_u = 850$ ft-k

STEEL MANUAL (TABLE 3-21) : $Q_n = 21.2^k$ ⟶ $\Sigma Q_n = 21.2^k/\text{stud}$ (15 studs)

$\Sigma Q_n = 318$ k



$b_{eff} \le \frac{28(12)}{2} = 168''$

$\le \frac{30(12)}{8} = 45''$

$$a = \frac{\Sigma Q_n}{0.85 f'_c b} = \frac{318^k}{0.85(4)(45)} = 2.08 \text{ in}$$

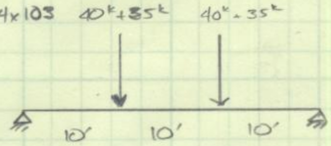
$$Y_2 = Y_{con} - \frac{a}{2} = 7.5'' - \frac{2.08 \text{ in}}{2} \approx 6.5''$$

CONT'D ⟶

G1 (CONT'D).

* W24x117 NOT LISTED IN TABLE 3-19 FOR COMPOSITE MEMBERS. BUT, BY CHECKING TABLE 3-2 (Z TABLES), ϕM_p FOR THE STEEL BEAM ALONE IS 1230 ft-kips > M_u = 850 ft-kips. THEREFORE, MEMBER DESIGN IS ADEQUATE. ANOTHER POSSIBLE REASON FOR THE APPARENT OVERSIZING OF THE MEMBERS IS VIBRATION CONSIDERATIONS. SOME OF THE SPACES IN THE BAY ARE OPERATING ROOMS, AFTER ALL.

G2 W24x103 $40^k + 35^k$ $40^k - 35^k$



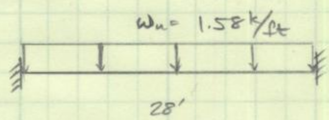
$M_u = Pa = 75\,(10)$

$M_u = 750$ ft-k

FROM TABLE 3-2 (Z TABLES), ϕM_p OF STEEL ALONE = 1050 ft.

$\therefore\ \phi M_p > M_u$ GOOD.

G3 W24x 146



$M_u = \dfrac{wL^2}{12} = \dfrac{1.58\,(28)^2}{12} = 103$ ft-k

FROM TABLE 3-2 (Z TABLES), ϕM_p OF STEEL ALONE = 1570 ft-k

$\therefore\ \phi M_p \gg M_u$ GOOD FOR GRAVITY.

(G3 IS PART OF A MOMENT FRAME)

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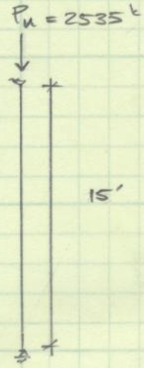
C1: W14 × 211

$P_u = 2535^k$



* STR. DRAWINGS INCLUDE LOADS FROM EACH GIRDER FRAMING INTO THE COLUMNS.

$$P_u = 2(75^k) + 2(40^k) + 2(70^k) + 2(40^k) + 2(90^k)$$
$$+ 50^k + 55^k + 2(70^k) + 2(40^k) + 2(70^k)$$
$$+ 2(40^k) + 2(70^k) + 40^k + 45^k + 2(70^k)$$
$$+ 2(40^k) + 2(70) + 40 + 45 + 2(70) + 40$$
$$+ 45 + 2(70) + 40 + 45 + 2(145) + 70 + 105$$

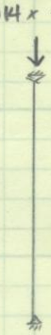
$$P_u = 2535^k$$

$kL = 15'$ (ASSUMED)

STEEL MANUAL: $\phi P_n = 2420^k < P_u$ $\therefore$ NO GOOD
(TABLE 4-1)

* THE ASSUMPTION OF THE 15' UNBRACED LENGTH COULD BE WRONG. IT IS VERY POSSIBLE THAT THE COLUMN IS BRACED ALONG IN BETWEEN FLOORS.

C2: W14 × 455



$kL = 15'$ (ASSUMED)

$$P_u = 14(40^k + 45) + 2(75) + 2(70) + 2(90) + 2(70)$$
$$+ 2(70) + 2(70) + 2(70) + 2(70) + 2(70)$$
$$+ 2(70) + 2(70) + 2(145)$$

$$P_u = 3070^k$$

STEEL MANUAL: $\phi P_n = 5330^k > P_u$ $\therefore$ GOOD
(TABLE 4-1)

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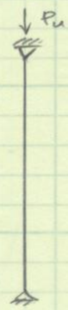
C3. W14×233



KL = 15' ASSUMED

$P_u = 14(40+45) + 2(75) + (70+75) + 2(90)$

$+ 2(70)(8) + 142 + 162$

$P_u = 3089^k$

STEEL MANUAL: (TABLE 4-1) $\phi P_n = 2680^k < P_u = 3089^k$ ∴ NO GOOD.

* UNBRACED LENGTH MAY NOT BE CORRECT. ALSO, NO LIVE LOAD REDUCTIONS WERE ASSUMED.