New Acute Care Hospital and Skilled Nursing Facility

San Francisco, CA

Technical Report 1

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October 5th 2009

Executive Summary

A thorough analysis was performed on the New Acute Care Hospital and Skilled Nursing Facility in San Francisco, CA in order to develop an understanding of how the structural system works. This analysis included a study of the structural system as shown in the structural plans, the codes used in the design of the building, as well as an analysis of the wind, seismic, dead, and live loads on the structure. Where appropriate, calculated loads were compared to those used by the designers.

The load analysis revealed that seismic loads will be the controlling lateral condition on the structure, resulting in a base shear of 1422 kips and an overturning moment of 110,750 ft-kips. The wind loads were determined to be much smaller in comparison to the seismic loads. A more in-depth analysis of the lateral system used to resist these loads will be undertaken in a future report.

After the load analysis was complete, spot checks were performed to verify the validity of the lateral loads on the structure. These spot checks indicated that the dead loads determined in this report were slightly larger than those used by the designers.

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Introduction

The New Acute Care Hospital and Skilled Nursing Facility will serve as an addition to the existing Chinese Hospital located in the historic Chinatown district of San Francisco (See Fig. 1). The site lies on the north flank of Nob Hill, at an elevation of approximately 110’ above sea level. Due to the slope of the site, the ground floor of the site is located partially below grade.

Figure 1: Site View of New Acute Care Hospital (blue) located adjacent to existing Chinese Hospital. Photo Courtesy of Google Maps.

This new addition will be connected directly to the existing Chinese Hospital, located at 845 Jackson Street. As part of the construction of this addition, the original portion of the hospital built in 1925 will be demolished. Then the new facility, which has seven stories above ground and one below, be constructed with a hard connection to a previous addition built in 1975. Therefore, the precast concrete panel exterior façade has been designed in a way that respects the 1975 design while providing a more modern look.

At approximately 92,000 SF, this new facility will provide additional patient rooms as well as well several new medical departments to serve the local community. Construction is expected to begin in 2010 and reach completion by Chinese New Year 2013.

Figure 2: Exterior view of New Acute Care Hospital and surrounding buildings

Structure Overview

The structure of the New Acute Care hospital rests on a mat foundation and consists primarily of composite steel decking with steel framing. A perimeter moment frame system is used to resist lateral loading.

Foundation System

According to the geotechnical report provided by Treadwell & Rollo, the soil conditions on the site can be described as "very stiff to hard sandy clay and clay with gravel," which rests on "intensely fractured, low hardness, weak, deeply weathered shale." Because of this, the New Acute Care Facility has been designed to bear on a 36" Mat foundation. Columns rest on concrete pedestals, typically sized at 3'-0" x 3'-0". Since the base of the structure will lay below the water table, the foundation was also designed for hydrostatic uplift.

The close proximity to nearby structures, particularly the 1975 addition to the Chinese Hospital provided a challenge to the designers. Underpinning was used to maintain the foundations of existing structures on either side of the building (see Fig.2).

Floor System

The New Acute Care hospital makes use of a composite floor system using a 3" Verco W3 Formlock deck with an additional 3 ¼" of concrete resulting in a total thickness of 6 ¼". This slab then rests on W-shapes ranging from W10x12's used as beams to sizes as large as W24x207's which also serve in the buildings lateral system. The slab is reinforced at mid-span as appropriate.

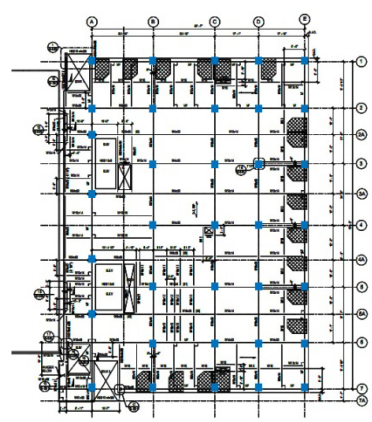

Figure 3: Typical Framing Plan with columns highlighted

There are several different bay sizes used in the New Acute Care Hospital. Larger bay typically exist on the plan east side of the building while smaller bay sizes are typically used in the western portion of the structure.

Framing System

The New Acute Care Hospital uses steel columns (See Figure 3) to support the buildings gravity loads. These columns range in size from W14x445 near the base of the structure to W8x40's near the roof level. As the columns rise vertically through the structure they are spliced together, usually at a distance of 22'-0". Aside from those used in the lateral system, most of the columns are connected to beams and girders using pinned connections.

Lateral System

As lateral loads move from through the frame of the structure, they are transferred to a series of special moment frames. These moment frames are used around the perimeter of the structure. As can be seen by the blue highlighting on Figure 3, there are 4 frames running east to west and two frames running north to south. See Figure 14 in Appendix D for a typical moment frame elevation.

Figure 4: Typical Framing Plans with lateral system highlighted in blue

Roof System

The roof system is supported in a similar manner to the floors below, with a concrete filled metal deck supported by beams and girders. However, beams at this level are typically spaced much closer together, at a distance of approximately 10-12 feet. The sizes of these roof beams generally vary from W10x12's to W24x104's.

Connection to Existing Structure

The structure of the New Acute Care Hospital is directly connected in several places with that of the existing Chinese Hospital. This connection generally consists of a fixed connection with a seismic joint between allowing minimum movement capability between zero inches to two feet. A typical joint is detailed in Figure 5.

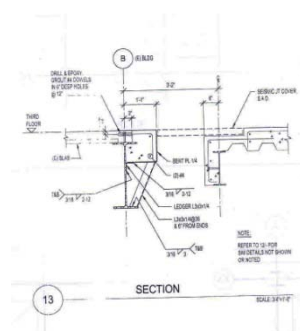

Figure 5: Typical connection between New Acute Care Hospital and existing structure

Materials Used

Concrete

Location	Weight	Strength f'c (ksi)
Foundation	Normal	4000
Drilled Piers	Normal	4000
Slab-on-Grade Walls, Columns, and Piers	Normal	4000
Fill in Metal Deck and Curbs at Ground Floor	Normal	4500
Fill in Metal Deck at First Floor and Above, Topping Slab, Curbs, and Pads	Light	4000
Fill in Stair Pans	Normal	2500
Fill in Over-Excavated Areas and Conduit Encasement	Normal	1500

Structural Steel

Type	Standard	Grade
W-Shapes	ASTM A992	Grade 50
Other Shapes	ASTM A992	Grade 50
Plates for Built-Up Members	ASTM A572	Grade 50
Steel Channels, Angles, Base Plates, Shear Tabs	ASTM A36	
Structural Steel Plates	ASTM A572	Grade 50
Steel Bars	ASTM A529	Grade 50
Square or Rectangular Steel Tubes	ASTM A500	Grade B
Round Steel Tubes	ASTM A500	Grade C
Pipe Sections	ASTM A53	Grade B

Reinforcing Steel

	ASTM A615	Grade 60

Applicable Codes

Original Design Codes Used

In addition to the following codes, the California State Government requires that all new government and hospital buildings are approved by the Office of Statewide Health Planning and Development (OSHPD).

- 2007 California Administration Code
  - Part 1, Title 24, CCR
- 2001 California Building Code
  - Part 2, Title 24, CCR
  - (1997 UBC and 2001 CA Amendments)
- 2004 California Electrical Code
  - Part 3, Title 24, CCR
  - (2002 NEC and 2004 CA Amendments)
- 2001 California Fire Code
  - Part 4, Title 24, CCR
  - (2000 UMC and 2001 Amendments)

Design Codes Used in Thesis Analysis

- American Society of Civil Engineers (ASCE)
  - ASCE7-05, Minimum Design Loads for Buildings and Other Structures
- International Building Code, 2006 Edition
- American Institute of Steel Construction (AISC)
  - Steel Construction Manual, Thirteenth Edition (LRFD)

Design Loads

Gravity Loads

Live Load (psf)		
Live Load	As Designed	Per ASCE 7
Treatment Rooms	80*+20(partitions)	60
Patient Room	80*+20(partitions)	40
Other Rooms (offices)	80*+20(partitions)	50
Storage Areas		
Fixed Racks	125	125
Mobile Racks	250	250
Corridors	100	80
Mechanical Rooms	125	-
Roof (Mech)	125	100
Roof (Other)	20*	20

The designed live loads were found to be larger than the minimum live loads specified by ASCE7-05. It is likely that these values were higher based on the more stringent requirements of OSHPD as well as the experience of the designers.

Floor Dead Loads	
Material	(psf)
6 1/4" Concrete Deck	50
Finishes	1
MEP and Misc.	20
Total	71

Partition Wall Dead Loads (psf)	
Per ASCE7-05 12.7.2	10

Exterior Wall Dead Loads	
Material	(psf)
5" Concrete Panels	50
6" Metals Studs and Wallboard	0.38
6" Batt Insulation	0.9
Total	51.28

Roof Dead Loads	
Material	(psf)
80 Mil. TPO Roof Membrane	5.5
5/8" Dens Deck	2.5
6 1/4" Concrete Deck	60.4
Total	68.4

Dead load values were determined from a combination of sources including but not limited to ASCE7-05, design aids, and manufacturer specifications.

Snow Loads

Due to the facilities location in San Francisco, CA; snow loads were not found to be a contributing gravity load to the structure.

Wind Loads

Wind loads were calculated as prescribed by ASCE7-05 Chapter 6. Although the New Acute Care Facility is an addition to an existing structure, it was modeled as an independent structure for the purpose of this analysis. This simplification was appropriate in that it allows for the possibility of the existing Chinese Hospital structure being demolished at a later date.

Microsoft Excel was used extensively in both the analysis and determination of net wind pressures, story forces, and overturning moments. The net wind pressures comprised of pressure of the windward, leeward, side, and internal area of the building. A detailed summary of the analysis can be found in Appendix A. Once the net wind pressures were determined, the net wind loads were found. Wind loads were the largest in the NS direction resulting in a base shear of 199 kips and an overturning moment of 34,880 ft-kips (See Figure 4).

Wind Loads - NS Direction							
Floor	Height Above Ground (ft)	Story Height (ft)	Wind Pressure (psf)	Internal Pressure (psf)		Net Pressure (psf)	
				(+)(Gc_{pi})	(-)(Gc_{pi})	(+)(Gc_{pi})	(-)(Gc_{pi})
Ground	0	12.5	6.91	4.15	-4.15	2.76	11.06
1	12.5	13.5	6.91	4.15	-4.15	2.76	11.06
2	26	13.5	8.18	4.15	-4.15	4.04	12.33
3	39.5	13.5	9.34	4.15	-4.15	5.19	13.49
4	53	13.5	10.19	4.15	-4.15	6.04	14.33
5	66.5	15	10.89	4.15	-4.15	6.74	15.04
6	81.5	15	11.65	4.15	-4.15	7.50	15.80
PH	96.5	18.5	12.15	4.15	-4.15	8.00	16.30
Parapet	101.5	5	12.29	4.15	-4.15	8.14	16.44
PH Roof	115	-	12.73	4.15	-4.15	8.58	16.88
Leeward	All	-	-12.79	4.15	-4.15	-16.94	-8.65
Side	All	-	-18.65	4.15	-4.15	-22.80	-14.50
Roof	0 to 52.875'	-	-22.71	4.15	-4.15	-26.86	-18.57
	52.875' to 105.75'	-	-17.24	4.15	-4.15	-21.39	-13.10
	105.75' to 134.83'	-	-14.29	4.15	-4.15	-18.44	-10.14

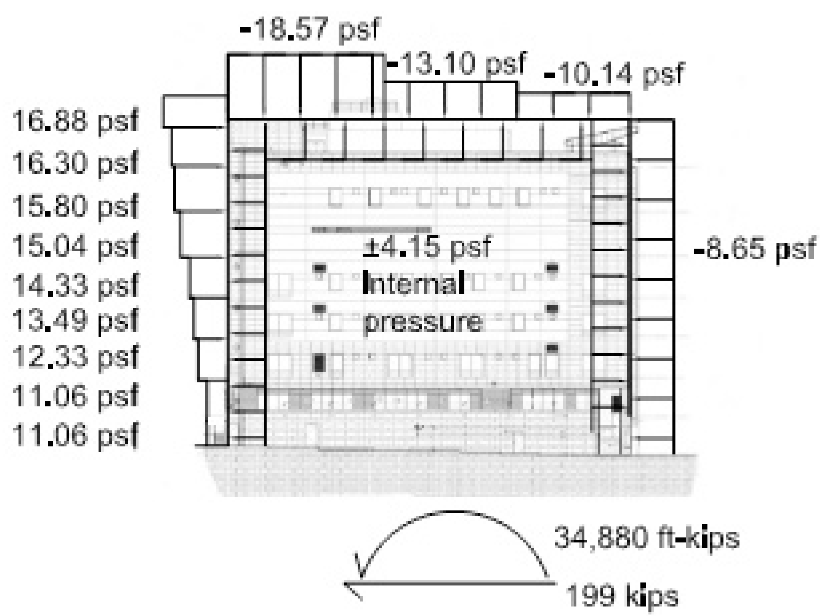

Figure 6: NS Wind Loads Diagram

Wind Loads - NS Direction					
Floor Level	Floor Height (ft)	Elevation (ft)	Story Force (kips)	Total Story Shear (kips)	Overturning Moment (ft-k)
Ground	6.25	0	9.32	199.43	0
1	13	12.5	19.38	190.11	2376.43
2	13.5	26	22.44	170.74	4439.11
3	13.5	39.5	24.55	148.29	5857.46
4	13.5	53	26.09	123.74	6558.12
5	14.25	66.5	28.89	97.65	6493.54
6	15	81.5	31.95	68.76	5603.72
PH	16.75	96.5	36.81	36.81	3551.96
		Total Overturning Moment (ft-kips)			34880.34
		Total Shear (kips)			199.43

Wind Loads - EW Direction							
Floor	Height Above Ground (ft)	Story Height (ft)	Wind Pressure (psf)	Internal Pressure (psf)		Net Pressure (psf)	
				(+)(GC_{pi})	(-)(GC_{pi})	(+)(GC_{pi})	(-)(GC_{pi})
Ground	0	12.5	6.71	4.15	-4.15	2.57	10.86
1	12.5	13.5	6.71	4.15	-4.15	2.57	10.86
2	26	13.5	7.97	4.15	-4.15	3.82	12.11
3	39.5	13.5	9.10	4.15	-4.15	4.96	13.25
4	53	13.5	9.93	4.15	-4.15	5.79	14.08
5	66.5	15	10.62	4.15	-4.15	6.48	14.77
6	81.5	15	11.37	4.15	-4.15	7.22	15.52
PH	96.5	18.5	11.86	4.15	-4.15	7.72	16.01
Parapet	101.5	5	12.00	4.15	-4.15	7.85	16.15
PH Roof	115	-	12.43	4.15	-4.15	8.29	16.58
Leeward	All	-	-14.32	4.15	-4.15	-18.47	-10.18
Side	All	-	-4.15	4.15	-4.15	-8.29	0.00
Roof	0 to 52.875'	-	-25.69	4.15	-4.15	-29.84	-21.55
	52.875' to 95.395'	-	-15.75	4.15	-4.15	-19.90	-11.60
		-	-	4.15	-4.15	-	-

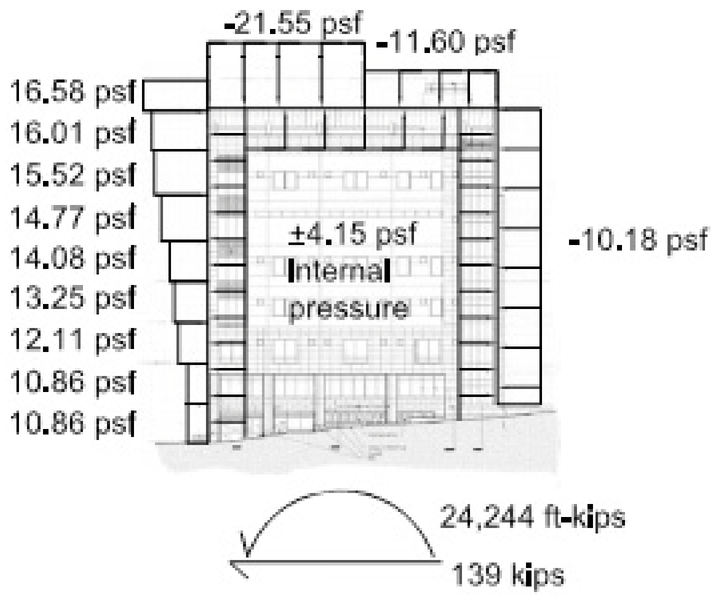

Figure 7: EW Wind Load Diagram

Wind Loads - EW Direction					
Floor Level	Floor Height (ft)	Elevation (ft)	Story Force (kips)	Total Story Shear (kips)	Overturning Moment (ft-k)
Ground	6.25	0	6.48	138.62	0
1	13	12.5	13.47	132.14	1651.76
2	13.5	26	15.60	118.67	3085.45
3	13.5	39.5	17.07	103.07	4071.29
4	13.5	53	18.13	86.01	4558.29
5	14.25	66.5	20.08	67.87	4513.40
6	15	81.5	22.21	47.79	3894.92
PH	16.75	96.5	25.58	25.58	2468.82
		Total Overturning Moment (ft-kips)			24243.92
		Total Shear (kips)			138.62

Seismic Loads

Seismic loads were determined using the Equivalent Lateral Force Method as described in ASCE7-05. In addition to this, the USGS Earthquake Ground Motion Parameter Application was used to confirm the seismic response coefficients for San Francisco's latitude and longitude (37°N, 122°W). Like the wind loads, Microsoft Excel was used extensively in the process of determining seismic loads. A detailed description of the process used can be found in Appendix B.

Building weight was determined by summing the weight of all the steel members on each floor, then adding the weight of the dead loads, 25% storage area live loads, and a partition weight of 10 psf as prescribed by ASCE7-05 §12.7.2. Since the lateral load resisting system consisted of special moment frames in both the NS and the EW direction, one analysis was performed to cover both directions. The results of the analysis can be found in the table below and in Figure 6.

Seismic Loads

Level	Story Weight (kips)	Story Height (ft) h_x	Modified h_x^k	$w_x h_x^k$	C_{vx}	Story Force (kips) $F_x=C_{vx}V$	Story Shear (kips) $V_x=\Sigma F_i$	Moment Contribution (ft-kips) M_x
Penthouse	1779.45	115	157.93	281023.70	0.22	330.85	0.00	38047.38
Roof	1896.83	96.5	132.52	251372.15	0.19	295.94	330.85	28558.04
6	1967.70	81.5	111.92	220230.77	0.17	259.28	626.79	21130.98
5	1977.88	66.5	91.32	180626.71	0.14	212.65	886.06	14141.24
4	1978.37	53	72.78	143993.48	0.11	169.52	1098.71	8984.68
3	1993.64	39.5	54.24	108144.21	0.08	127.32	1268.23	5029.03
2	2034.90	26	35.71	72656.99	0.06	85.54	1395.55	2224.00
1	2009.43	12.5	17.17	34494.03	0.03	40.61	1481.09	507.62
Ground	2007.41	0	0.00	0.00	0.00	0.00	1521.70	0
				Effective Seismic Weight W (kips)				17645.60
				Base Shear $V=C_sW$ (kips)				1521.70
				Overturning Moment $M=\Sigma M_x$ (ft-kips)				118622.98

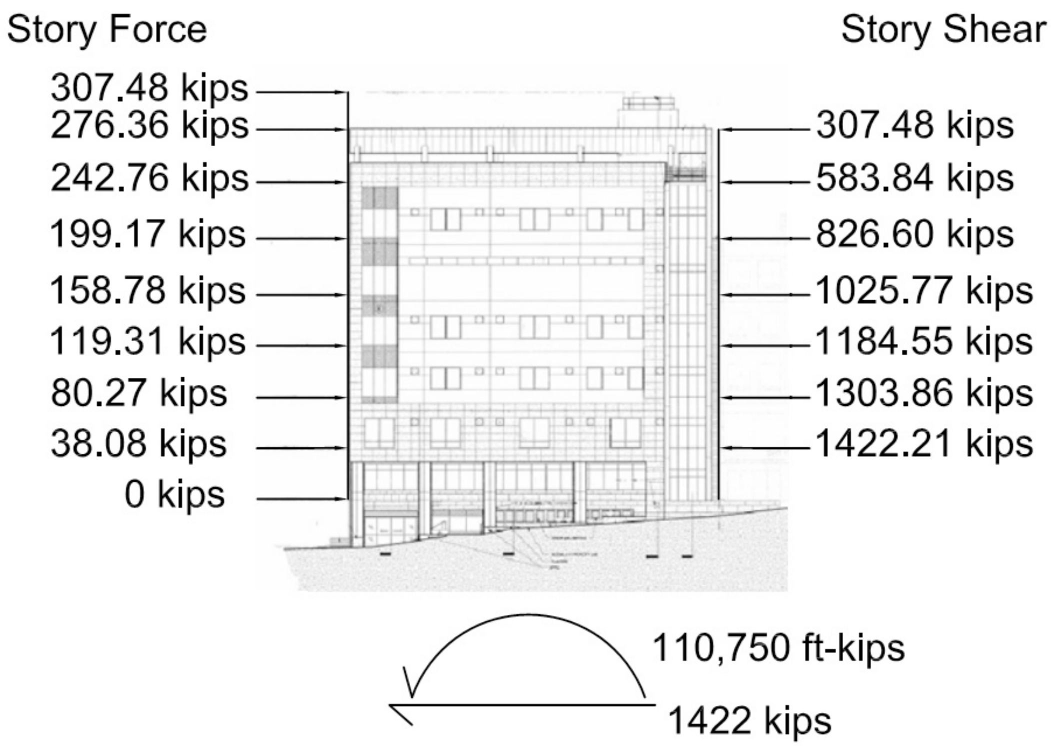

Figure 8: Seismic Load Diagram

The seismic loads used by ARUP, the structural engineers on the project, were not available at the time of this report. However, since seismic loads are the controlling lateral force for this structure, the values calculated in this report will be confirmed prior to an in-depth analysis of the lateral system.

Spot Checks

A series of spot checks were performed in order to determine the accuracy of the gravity loads determined in this report. A detailed set of these calculations can be found in Appendix C.

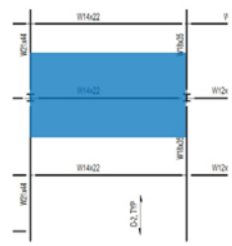

Figure 9: Interior Beam Spot Check

The first spot check performed was on an interior beam located on the third floor along grid line 3 and between grids B and C. This beam, a W14x22, can be considered representative of an interior beam located in the central portion of the building throughout the structure.

The analysis performed revealed that the designed beam can carry the required load once composite action is in effect. However, the beam failed to carry the required loads that would be in place during construction before the steel and concrete are effectively working together. A W14x26 would have to be used to carry the load calculated in this study. Since the live loads selected from ASCE7-05 were generally lower than those used in the design, it can be concluded that the dead loads used in this analysis were too large by a small margin.

The next spot check I performed was on a W12x72 interior column on the 2nd floor located at grid C-3. For the purposes of analyzing this column, the load was taken to be the dead load, including self weights of the beams and framing into the column and the column self weight, and live loads. Lateral loads were not taken into account at this time, therefore beam-column effects were not considered.

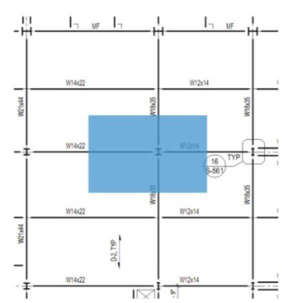

Figure 10: Column Spot Check

The analysis of the column used in the design revealed that the axial loads could be carried by a large margin. The main reason for is that the column had additional requirements based on the lateral loads on the structure. This is particularly true since the New Acute Care hospital lies in a region of large seismic activity. Another possible reason for this difference could be that 2nd order effects were ignored in the initial column analysis since all the beam/girder connections are pinned. However, in any real structure, there is some element of fixity, which would result in higher loads on the column.

The final spot check performed was on a W18x35 girder located on the 3rd floor along grid line C between grid lines 3 and 4. The analysis showed that a larger steel section is required. A W18x46 was found to be the next size that would resist the required loads. In addition to this, an additional 10 shear studs would be required to obtain composite action.

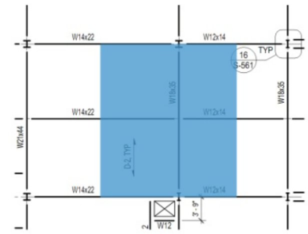

Figure 11: Girder Spot Check

This confirms the assertion that gravity loads, most likely dead loads, are larger than those used by the designer. Although the gravity loads has an effect on seismic loads on the structure, this discrepancy will more than likely prove to be negligible over the entirety of the structure. However, the variations will be checked with the engineer of record prior to the next report.

APPENDIX

Appendix A: Wind Analysis

JACOBS

Subject: WIND ANALYSIS Project: ______
Sheet No. 1 Of 3
Authored by ______ Date ______ Checked by ______ Date ______

USE METHOD 2 SINCE BUILDING IS NOT A LOW-RISE AND MEETS REQ OF 6.5.1 & 6.5.2

- BASIC WIND SPEED. => USING TABLE 6-1, $V = 85$ MPH
- WIND DIRECTIONALITY FACTOR => $K_d = 0.85$ => USING TABLE 6-4 (BUILDINGS)
- OCCUPANCY CATAGORY = III (HEALTH CARE FACILITY) IBC 1604
  ↳ IMPORTANCE FACTOR $I = 1.15$ (ASCE TABLE 6-1)
- USE EXPOSURE C (CONFIRMED IN PLANS)
- TOPOGRAPHIC FACTOR

  NOB HILL ELEV = 315' ← ASSUME BOTTOM OF HILL IS SEA LEVEL

  BUILDING ELEV. = 110'

  $\frac{110}{315} = 0.349$ ∴ NOT TOP $\frac{1}{2}$ OF HILL

  ∴ USE $K_{zt} = 1.0$

- INTERPOLATE TABLE 6-3 TO FIND K_h & K_z AT BUILDING ELEVATIONS: SEE EXCEL SPREADSHEET
- VELOCITY PRESSURES

  $q_z = 0.00256\, K_z K_{zt} K_d V^2 I$

  $q_h = 0.00256\, K_h K_{zt} K_d V^2 I$

  > SEE EXCEL FOR GRAPHS

GUST EFFECT FACTORS

$n_1 = \frac{75}{H} = \frac{75}{115} = 0.652$ (LOWER BOUND ASCE 7-05 C6-17)

$n_1 = \frac{100}{H} = \frac{100}{115} = 0.869$ (AVG FOR n_1 ASCE 7-05 C6-18) ✱ USE FOR CALCS

IN EITHER CASE $n_1 \leq 1$ Hz ∴ STRUCTURE IS FLEXIBLE.

JACOBS

Subject: WIND ANALYSIS Project: ______
Sheet No. 2 Of 3
Authored by ______ Date ______ Checked by ______ Date ______

- $g_Q = g_V = 3.4$
- $g_R = \sqrt{2\ln(3600 n_1)} + \dfrac{0.577}{\sqrt{2\ln(3600 n_1)}}$
- $\bar{z} = 0.6h \geq z_{min}$
- $I_{\bar{z}} = c\left(\dfrac{33}{\bar{z}}\right)^{1/6}$
- $L_{\bar{z}} = \ell\left(\dfrac{\bar{z}}{33}\right)^{\bar{\epsilon}}$
- $Q = \sqrt{\dfrac{1}{1+0.63\left(\dfrac{B+h}{L_{\bar{z}}}\right)^{0.63}}}$
- $\bar{V}_{\bar{z}} = \bar{b}\left(\dfrac{\bar{z}}{33}\right)^{\bar{\alpha}} V\left(\dfrac{88}{60}\right)$
- $N_1 = \dfrac{n_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}}$
- $R_n = \dfrac{7.47 N_1}{(1+10.3N_1)^{5/3}}$
- $R_h = \dfrac{1}{\eta} - \dfrac{1}{2\eta^2}\left(1-e^{-2\eta}\right)$
- $\eta = 4.6 n_1 h/\bar{V}_{\bar{z}}$
- $R_B = \dfrac{1}{\eta} - \dfrac{1}{2\eta^2}\left(1-e^{-2\eta}\right)$
- $\eta = 4.6 n_1 B/\bar{V}_{\bar{z}}$
- $R_L = \dfrac{1}{\eta} - \dfrac{1}{2\eta^2}\left(1-e^{-2\eta}\right)$
- $\eta = 15.4 n_1 L/\bar{V}_{\bar{z}}$
- $R = \sqrt{\dfrac{1}{\beta} R_n R_h R_B (0.53+0.47R_L)}$

– SEE EXCEL SPREADSHEET FOR CALCS
– SELECT VALUES CAN BE FOUND IN ASCE7-05 TABLE 6-2.

APPROPRIATE VALUES WERE CALCULATED IN NS & EW DIRECTIONS.

$G_F = 0.925\left(\dfrac{1+1.7I_{\bar{z}}\sqrt{g_Q^2Q^2+g_R^2R^2}}{1+1.7g_V I_{\bar{z}}}\right)$

JACOBS

Subject: WIND ANALYSIS | Project:
Sheet No. 3 Of 3
Authored by ___ Date ___ Checked by ___ Date ___

- Building is Fully Enclosed.
- Building Has A Parapet
  - Parapet Pressures

$P_p = q_p G C_{pn}$ → Determined using K_z & q_z spreadsheet

- Find External Pressure Coefficients C_p ⟹ See Excel Spreadsheet
  - Walls (use ASCE Fig. 6-6)
    - Interpolate L/B values to obtain C_p
  - Roof

0 → 0
$h/2 \to 52.875'$
$h = 105.75'$
↑ $2h = 211.50'$
MEAN ROOF HEIGHT

- Use Fig. 6-6
- Ridge = 0°
- Interpolate between 1st C_p values for each distance using h/L so for $0 \to h/2$

$$\frac{h}{L} \quad \frac{(0.78-0.5)}{(1-.5)} = \frac{(x - -.9)}{(-1.3 - -.9)}$$

- Use appropriate values for NS & EW directions.
- Use Reduction Factors as appropriate values interpolate

- Internal Pressure
  ±0.18 for enclosed structures. (Fig 6-5)

Designed Wind Pressures

Windward Walls $P_z = q_z G_f C_p - q_h(GC_{pi})$

Leeward Walls, Side Walls, Roofs } $P_h = q_h G_f C_p - q_h(GC_{pi})$

Wind Load Design Criteria	
Design Wind Speed	85 mph
Directionality Factor K_d	0.85
Importance Factor (I_w)	1.15
Exposure	C
Topographic Factor (k_{zt})	1
Mean Roof Height (h)	105.75 ft
K_h	1.27
q_h	23.04

Velocity Pressure Coefficents K_z and Velocity Pressure q_z			
Floor Level	Height	K_z	q_z
Ground	0	0.850	15.368
1	12.5	0.850	15.368
2	26	0.948	17.140
3	39.5	1.037	18.749
4	53	1.102	19.924
5	66.5	1.156	20.900
6	81.5	1.215	21.958
Roof	96.5	1.253	22.654
Parapet	101.5	1.264	22.848
Penthouse	115	1.298	23.459

Building Dimensions		
	N-S Wind	EW Wind
B	95.395	134.83
L	134.83	95.395
h	105.75	105.75

B=normal to wind direction

L=parallel to wind direction

h=mean roof height

Gust Effect Factors G and G_f		
Term	NS Wind	EW Wind
n_1	0.86	
g_Q	3.40	
g_v	3.40	
g_R	4.15	
z_{MEAN}	63.45	
c	0.2	
I_{ZMEAN}	0.179	
L_{ZMEAN}	569.841	
Q	0.858	0.844
V_{zMEAN}	89.607	
N_1	5.469	
R_n	0.048	
η_h	4.669	
R_h	0.191	
η_B	4.212	5.953
R_B	0.209	0.154
η_L	19.928	14.099
R_L	0.049	0.068
β	0.010	
R	0.326	0.282
G_f	0.899	0.883

Combined Net Design Pressure on Parapet (lbs/ft^2)		
	windward	leeward
GC_{pn}	1.5	-1.0
p_p	34.2725967	-22.8483978

External Pressure Coefficents		
Wind Direction	NS	EW
L/B	1.413386446	0.707521
C_p (walls) windward	0.8	
C_p (walls) leeward	-0.417322711	-0.5
C_p (walls) sidewall	-0.7	
h/L	0.784320997	1.108549
C_p (roof)		
0-h/2	-1.12	-1.3
h/2-h	-0.79	-0.7
h-2h	-0.612	-
>2h	-	-
Reduction Factor	0.8	0.8

Appendix B: Seismic Analysis

JACOBS

Subject: SEISMIC CALCS Project: ____
Sheet No. 1 Of 3
Authored by TMA Date ____ Checked by ____ Date ____

SEISMIC GROUND MOTION VALUES

SITE CLASS => C

S_S => 0.2 SEC SPECTRAL RESPONSE ACCELERATION

- 150% g @ 37°N 122°W

S_1 => 1 SEC SPECTRAL RESPONSE ACCELERATIONS

- 0.620% g

DETERMINE USING USGS WEBSITE.

$S_S \geq 0.15$, $S_1 \geq 0.6$

MAT FOUNDATION USED FOR SEISMIC DAMPING.

∴ SEE GEOTEC REPORT (PG. 30) FOR SEISMIC HAZARD ANALYSIS
↳ PROBABILISTIC SEISMIC HAZARD ANAL (PSHA)

SEISMIC DESIGN CATEGORY

- OCCUPANCY CATEGORY => IV FROM IBC TABLE 1604.5
- CONDITIONS OF SIMPLIFIED DESIGN PROCEDURE (ASCE 12.14) ARE NOT M
- CONDITIONS OF 11.6

1) $T_a = C_t h_n^x$ STEEL MOMENT FRAMES ARE USED

$= (0.028)(115)^{0.8} = 1.25$

$0.8T_S = 0.8\left(\frac{S_{D1}}{S_{DS}}\right) = 0.46$

$T_a > 0.8T_S$ ∴ ORIGINAL S_{DS} & S_{D1} DETERMINATIONS HOLD.

USING TABLE 11.6-1 $S_{DS} = 1 > 0.5$
∴ SDC = D

JACOBS

Subject: SEISMIC CALCS Project: ____
Sheet No. 2 Of 3
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PERMITTED ANALYTICAL PROCEDURE

- SDC = D
- USE T_a AS APPROXIMATION OF T $T_a = 1.25 < 3.5 T_s = 2$

$T_a < C_u = 1.4$ GOOD.

STRUCTURE IS REGULAR ∴ EQUIVALENT LATERAL FORCE PROCEDURE MAY BE USED.

EQUIVALENT LATERAL FORCE PROCEDURE

- SPECIAL STEEL MOMENT FRAMES USED AROUND PERIMETER OF BUILDING.
- USING TABLE 12.2-1
  - $R = 8.0$
  - $\Omega_o = 3$
  - $C_d = 5.5$
- USING TABLE 11.5-1
  - $I = 1.5$
- USING FIGURE 22-15 $T_L = 12$ SEC FOR SAN FRANCISCO

$T = 1.25 < T_L = 12$ ∴ USE

$$C_s = \frac{S_{D1}}{T\left(\frac{R}{I}\right)} = 0.086 < \frac{S_{DS}}{\left(\frac{R}{I}\right)} = 0.1875$$

↑ USE THIS VALUE

$$\frac{0.5 S_1}{\left(\frac{R}{I}\right)} = 0.058 \implies C_s > 0.058$$

JACOBS

Subject SEISMIC CALCS Project

Sheet No. 3 Of 3

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TOTAL BUILDING WEIGHT =
INCLUDES
- TOTAL DEAD LOAD → USED 250 psf (HEAV
- MIN 25% LIVE LOAD IN STORAGE AREAS.
- PARTITION WT OR 10 psf
- WT OF PERMANENT EQUIP (ASSUMED 100 psf IN ROOMS
- SNOW (IF OVER 30 PSF)

- SEE EXCEL SPREAD SHEET FOR CALCS.

$W = 17,569.82^{\text{KIPS}} \approx \boxed{17,570^{\text{KIPS}}}$

- $V = C_S W = 0.086(17,570) = 1515$

- SINCE $2.5\text{SEC} > T_a = 1.25 < 0.5\text{SEC}$

INTERPOLATE TO FIND k
$k = 0.75 + 0.5T$
$= 0.75 + 0.5(1.25)$
$k = 1.37\text{ SEC}$

- LATERAL SEISMIC FORCE DETERMINED BASED

$$F_x = \frac{w_x h_x^k}{\sum w_i h_i^k}$$

- $V_x = \sum F_i$ ∴ SEISMIC DESIGN STORY SHEAR DECREASES TO ZERO @ ROOF LEVEL.

- SINCE BUILDING USES STEEL FOR MF'S IN STRUCTURE IT CAN BE IDEALIZED AS HAVING A FLEXIBLE DIAPHRAGM. (ASCE 7-05, 12.3.1.1)

Ground Floor				
Beam	# of Beams	Length (ft)	Unit Weight (lbs/ft)	Weight (lbs)
W12x14	1	9	14	126
W14x22	1	9	22	198
W14x22	1	10.583	22	232.826
W14x22	1	13.25	22	291.5
W24x176	1	23.8333	176	4194.661
W24x176	1	17.0833	176	3006.661
W24x176	1	18	176	3168
W18x35	1	22.625	35	791.875
W12x14	7	18.042	14	1768.116
W14x22	1	18.042	22	396.924
W12x14	2	8.71	14	243.88
W12x14	1	4.583	14	64.162
W14x22	1	10.1667	22	223.6674
W18x35	1	23.833	35	834.155
W24x176	1	23.8333	176	4194.661
W14x22	1	17.0833	22	375.8326
W14x30	1	17.8333	30	534.999
W14x22	1	11.0833	22	243.8326
W14x44	1	21.5833	44	949.6652
W18x35	2	21.5833	35	1510.831
W24x55	4	10.1667	55	2236.674
W12x14	9	10.1667	14	1281.004
W18x50	1	10.1667	50	508.335
W14x22	1	10.1667	22	223.6674
W18x50	5	23.8333	50	5958.325
W12x14	5	23.8333	14	1668.331
W12x16	7	23.8333	16	2669.33
W12x14	9	17.5833	14	2215.496
W14x22	1	17.8333	22	392.3326
W12x14	7	17.8333	14	1747.663
W12x19	2	17.8333	19	677.6654
W18x35	4	24	35	3360
W21x44	2	24	44	2112
W18x35	2	21.5833	35	1510.831
W21x44	1	21.5833	44	949.6652
W12x14	2	11.25	14	315
W12x14	3	8.708	14	365.736
W14x22	7	18.0417	22	2778.422
W12x14	1	5.625	14	78.75
W14x22	1	5.625	22	123.75

W12x14	1	9.2083	14	128.9162
W14x22	1	9.2083	22	202.5826
W14x22	1	7.944	22	174.768
W14x22	1	21	22	462
W14x22	1	25.75	22	566.5
W14x22	1	23	22	506
W14x22	1	10.333	22	227.326
W36x150	2	23.8333	150	7149.99
W24x207	1	17.0833	207	3536.243
W24x207	1	16.875	207	3493.125
W24x207	1	23.8333	207	4933.493
W24x207	1	17.0833	207	3536.243
W24x207	1	17.833	207	3691.431
W24x192	1	29.125	192	5592
W30x148	1	21	148	3108
W24x192	1	25.75	192	4944
W30x148	1	23	148	3404
W24x192	1	28.3747	192	5447.942
Total Beam Weight (lbs)				105627.8
Floor weight from beams (psf)				8.57

Ground Floor Column Weight (lbs)				
Column Size	# of Columns	Floor Height (ft)	Unit Weight (lbs/ft)	Weight (lbs)
W14x445	10	12.5	445	55625
W14x426	12	12.5	426	63900
W14x398	2	12.5	398	9950
W12x120	5	12.5	120	7500
W12x106	2	12.5	106	2650
W12x96	2	12.5	96	2400
W12x79	1	12.5	79	987.5
Total Column Weight (lbs)				143012.5

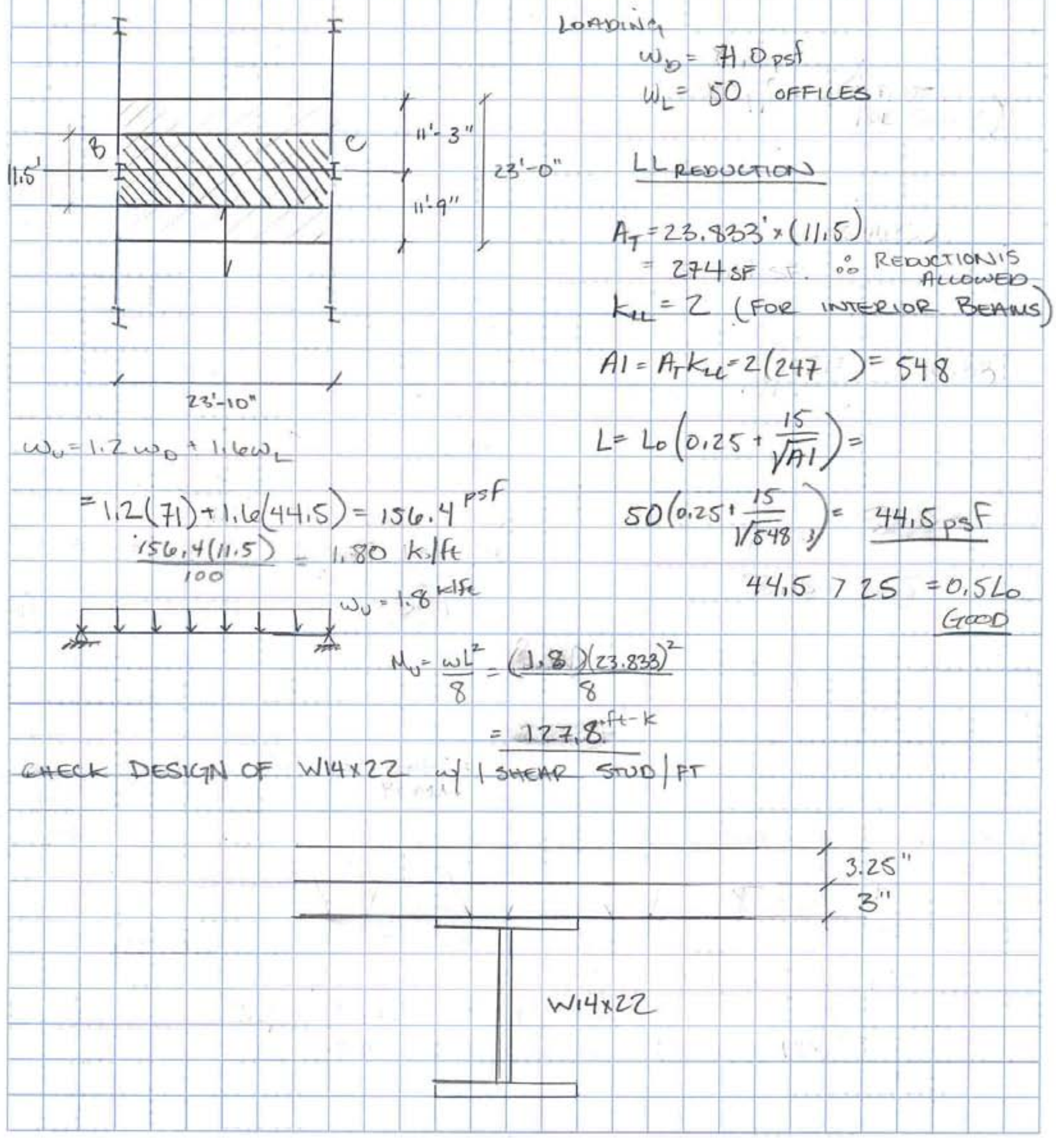
Ground Floor Story Weight		
	(psf)	Weight (lbs)
Dead Loads		
Floor	71	875075
Exterior Wall	50.38	23197
Partition Wall	10	123250
Live Load		
25% in Storage Areas	250	14062.5
Weight of Permanent Equip.	100	595000
Beam Weight		105627.7861
Column Weight		143012.5
Total Story Weight (kips)		1879.23

Seismic Design Criteria		
	By Design	By ASCE 07
Z-Factor	0.4	0.4
Importance Factor (I)	1.5	1.5
R (SMRF System)	8.5	8
R (Basement Shear Wall System)	5.5	5.5
Ω_0 (SMRF System)	2.8	3
Ω_0 (Basement Shear Wall System)	2.8	2.8
Near Field Factors N_a	1	
Near Field Factors N_v	1.09	1.09
C_S		0.086
Seismic Coefficent C_a	0.4	0.4
Seismic Coefficent C_v	0.61	0.61
Soil Type	S_c	S_c
S_d	-	5.5

Seismic Ground Motion Values	
S_s .2 Sec Spectral Response Accel.	1.5
S_1 1 Sec Spectral Response Accel.	0.62
F_a	1
F_v	1.3
S_{MS}	1.5
S_{M1}	0.860
$S_{DS} = 2S_{MS}/3$	1
$S_{D1} = 2S_{M1}/3$	0.573

Appendix C: Spot Checks

Composite Beam Spot Check



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Subject: TECH. 1 — COMPOSITE BEAM SPOT CHECK   Project: ____   Sheet No. 1 Of 3

Authored by: TMA   Date: ____   Checked by: ____   Date: ____

SPOT CHECK #1: COMPOSITE STEEL BEAM

LOCATION: 3RD FLOOR

GRID LINE 3 BETWEEN B & C

LOADING

$w_D = 71.0$ psf

$w_L = 50$ OFFICES

LL REDUCTION

$A_T = 23.833' \times (11.5)$

$= 274$ SF ∴ REDUCTION IS ALLOWED

$K_{LL} = 2$ (FOR INTERIOR BEAMS)

$AI = A_T K_{LL} = 2(247) = 548$

$L = L_o\left(0.25 + \frac{15}{\sqrt{AI}}\right) =$

$50\left(0.25 + \frac{15}{\sqrt{548}}\right) = 44.5$ psf

$44.5 > 25 = 0.5 L_o$

GOOD

$w_u = 1.2 w_D + 1.6 w_L$

$= 1.2(71) + 1.6(44.5) = 156.4$ psf

$\frac{156.4(11.5)}{100} = 1.80$ k/ft

$M_u = \frac{wL^2}{8} = \frac{(1.8)(23.833)^2}{8}$

$= 127.8$ ft-k

CHECK DESIGN OF W14x22 w/ 1 SHEAR STUD / FT

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Subject: TECH 1 — Project: — COMPOSITE BEAM SPOT CHECK — Sheet No. 2 of 3

Authored by ______ Date ______ Checked by ______ Date ______

DETERMINE EFFECTIVE FLANGE WIDTH

$b_{eff} \leq \frac{SPAN}{4} = \frac{(23.833')(12''/')}{4} = 71.5''$ ⇐ CONTROLS

OR

$\leq$ DIST. TO ADJ BEAM $= (11.25')(12''/') = 135''$

CHECK REQUIRED STRENGTH OF BARE STEEL UNDER CONST. DEAD + LIVE LOAD.

$M_U = 1.2(71) + 1.6(40) = 149.2^{ft\text{-}k}$ (40 = ASSUME)

$\phi M_p = 125^{ft\text{-}k} < 149.2^{ft\text{-}k}$

∴ A W14x22 WOULD NOT BE ABLE TO SUPPORT THIS LOAD W/O SOME COMPOSITE ACTION.

A W14x26 ($\phi M_p = 151^{ft\text{-}k}$) WOULD WORK

CHECK DEFLECTION UNDER CONSTRUCTION LOADS FOR W14x26

$\Delta_{D.CONST} = \frac{5 w_D L^4}{384 EI} = \frac{5(0.46)(23.833')(1728)}{384(29{,}000)(245)} = 0.47''$ (→ 40 psf (11.5')/1000)

$\Delta_{ALLOW} = L/360 = \frac{(23.833' \times 12''/')}{360} = 0.79''$

$0.47 < 0.79$ GOOD

NOW, CHECK DESIGNED BEAM SIZE UNDER FULL GRAVITY LOADS w/ COMPOSITE ACTION.

ASSUME $\Sigma Q_n = 81.2^{kips}$

$a = \frac{\Sigma Q_n}{0.85 f'_c b_{eff}} = \frac{81.2}{0.85(4)(71.5'')} = 0.33''$

→ TABLE 3-19 CONFIRMS THAT A COMPOSITE W14x22 CAN CARRY THE REQUIRED LOAD

$Y_2 = 3.25 - \frac{0.33}{2} = 3.085'' \approx 3''$

PNA #7 $\phi M_n = 174^{'k} > M_U = 127.8^{'k}$ GOOD

JACOBS

Subject TECH 1 Project
COMPOSITE BEAM SPOT CHECK Sheet No. 3 Of 3
Authored by Date Checked by Date

NOW CHECK THAT THE DESIGNED # OF SHEAR STUDS WORK.

1 $\frac{3}{4}$" ϕ SHEAR STUDS SPACED @ 1' INTERVALS.

LIGHTWEIGHT CONCRETE ON METAL DECK ⊥ TO BEAM.
↳ $f'_c = 4000^{psi}$

⇒ USING WEAK SHEAR STUD VALUES $Q_n = 17.2^{kip}$

$$\#\text{ OF STUDS REQ'D} = \frac{\Sigma Q_n}{Q_n}(2) = \frac{81.2(2)}{17.2} = 9.44$$

STUDS PROVIDED: 1 PER LF ∴ 23 STUDS > # REQUIRED
GOOD

THEREFORE, THE DESIGNED BEAM (A W14X22 w/ 1 SHEAR STUD/LF) WILL EASILY CARRY THE LOAD.

CHECK DEFLECTION

USING TABLE 3-20 $I_{LB} = 326\ in^4$

$$\Delta = \frac{5 w_L L^4}{384 EI} = \frac{5(0.575)(23.833)^4(1728)}{384(29{,}000)(326)} = 0.44\ in$$

(↳ $50\left(\frac{11.5}{1000}\right)$)

ACCORDING TO THE PROJECT REQ, $\Delta_{LL,max}$ FOR INTERIOR BEAMS IS 0.75^{in}

0.44 in < 0.75 in.

GOOD.

Column Spot Check

JACOBS

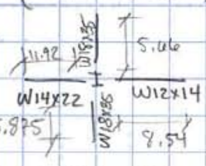
Subject: TECH 1 — Project:

COLUMN SPOT CHECK — Sheet No. 1 Of 1

Authored by ______ Date ______ Checked by ______ Date ______

TO CALCULATE LOADS ACTING ON COLUMNS, USE

- DEAD LOADS: 71 PSF
- LIVE LOADS: 60 PSF (USE LARGEST, MORE CONSERVATIVE)
- BEAM WEIGHT (ASSUME



$W_B = 785.5$ (PER FLOOR)

- COLUMN WEIGHT: TAKEN FROM EXCEL SPREAD FOR BUILDING WEIGHT

CHECK THE COLUMN STRENGTH OF A

TREAT COLUMN AS BEING PINNED ON EACH END.

COLUMN USED W12x79

TOTAL COLUMN HEIGHT = 27'

BRACED LENGTH ON FLOOR 2 = 13.5'

$A = 23.2$

$I_x = 662\ in^4$ — $I_y = 216\ in^4$

$r_x = 5.34$ — $r_y = 3.05$

K=1.0 CONSERVATIVE

$$\frac{KL}{r_x} = \frac{(13.5')(12'')}{5.34''} = 30.33 \qquad \frac{KL}{r_y} = \frac{(13.5\times 12)}{3.05} = 53.11$$

↑ CONTROLS

$$4.71\sqrt{\frac{E}{F_y}} = 4.71\sqrt{\frac{29000}{50}} = 113.4 > 53.11 \quad \therefore \text{INELASTIC BEHAVIOR}$$

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = \frac{\pi^2(29000)}{(53.11)^2} = 101.47\ ksi$$

$$F_{cr} = 0.658^{(F_y/F_e)}F_y = \left[0.658^{(50/101.47)}\right]50 = 40.68\ ksi$$

NOMINAL STRENGTH OF COLUMN $P_n = F_{cr}A = (40.68)(23.2) = 943.82^k$

$$\phi P_n = 0.9(943.82) = 849.438^k$$

$P_u = 330.23^k$ (SEE EXCEL SPREADSHEET FOR LOAD DETERMINATIC

$\phi P_n \gg P_u$ ∴ THE DESIGNED COLUMN HAS MORE THAN ENOUGH STRENGTH FOR THE AXIAL LOAD.

- PINNED CONNECTIONS TO BEAMS & GIRDERS ∴ NO 2ND ORDER EFFECTS

Composite Girder Spot Check

JACOBS

Subject: TECH 1 | Project:
GIRDER SPOT CHECK | Sheet No. 1 Of 3
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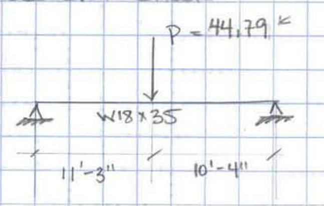
GIRDER SPOT CHECK



$M_{MAX} = 43.98^K (11.25')$

$= 494.8'^K$

$b_{eff} = \frac{21.588'(12''/')}{4} = 64.77''$ ← CONTROLS

OR

$17.08'(12''/') = 205''$

TABLE 3-19 SHOWS THAT NO W18x35 WILL CARRY THE REQUIRED LOAD.

$P = P_{W14x22} + P_{W12x14}$

FIND LOAD FROM W14x22

$w_U = 1.2(71) + 1.6(63.7) = 187.1 \left(\frac{11.5}{1000}\right) = 2.15\ ^{K/'}$

→ GIRDER LIES ON CORRIDOR

$LL = 80\ psf$

$A_T = 753.6\ SF > 400\ SF$ ∴ LL REDUCTION IS ALLOWABLE

$L = 80\left(0.25 + \frac{15}{\sqrt{753}}\right) = 63.7\ psf$

FOR W14x22

$P = \frac{(2.15 \times 23.833)}{2} = 25.6^K$

FOR W12x14

$P = \frac{(2.15 \times 17.08)}{2} = 18.36^K$

$P_{TOTAL} = 43.98^K$

TRY A W18x46 w/ $\frac{3}{4}''$ SHEAR STUDS SPACED @ 1'

ASSUME $\Sigma Q_n = 310^{KIPS}$

$a = \frac{\Sigma Q_n}{0.85 f'_c b} = \frac{310}{0.85(4)(64.77)} = 1.4$

$Y_2'' = 3.25 - \frac{1.4}{2} = 2.55$

@ PNA BFL, $\phi M_n = 508'^K > M_U$ GOOD.

JACOBS

Subject: TECH 1 Project:
GIRDER SPOT CHECK Sheet No. 2 Of 3
Authored by Date Checked by Date

CHECK THAT 1 SHEAR STUD / LF WILL BE SUFFICENT)
- 3/4" ϕ SHEAR STUDS
- LIGHTWEIGHT CONCRETE $f'_c = 4$ ksi

USING TABLE 3-19 $Q_n = 18.3$

OF STUDS REQ'D $= \frac{\Sigma Q_n}{Q_n}(2) = \frac{310(2)}{18.3} = 33.4$ STUDS ✗

THEREFORE, THE NUMBER OF STUDS USED IN THE DESIGN (~22s) IS INSUFFICENT.

JACOBS

Subject: TECH 1 Project:
GIRDER SPOT CHECK Sheet No. 3 Of 3
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CHECK DEFLECTION

$I_{LB} = 1280 \text{ in}^4$

$\omega_L = 80\text{psf}\left(\frac{20.45'}{1000}\right) = 1.63$

$\Delta_{LL} = \frac{5 \omega_L L^4}{384 E I} = \frac{5(1.63)(21.833)^4(1728)}{384(29{,}000)(1280)}$

$= 0.22$

$0.22 < 0.75$ GOOD

$\Delta_{allow} = 0.75$

GOOD

Appendix D: Plans

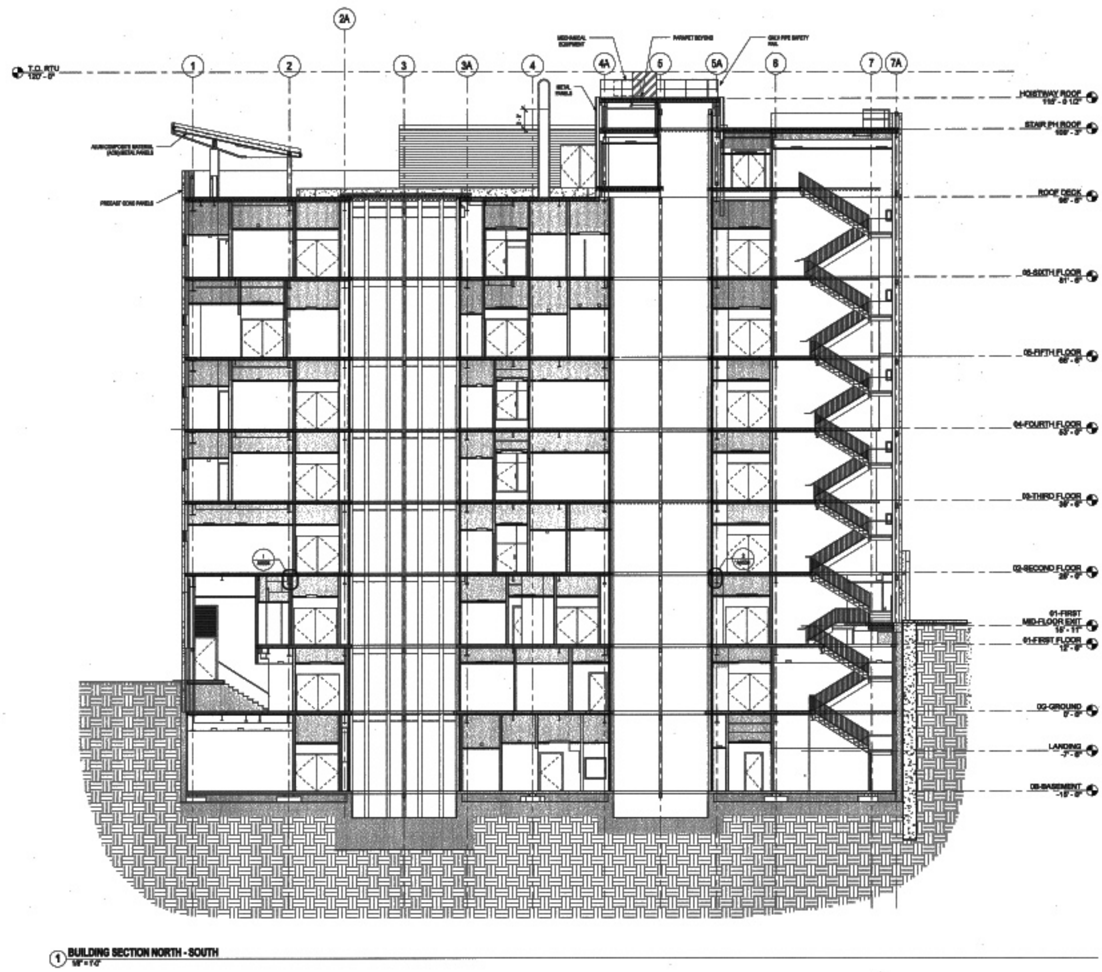

Figure 12: NS Buiding Section

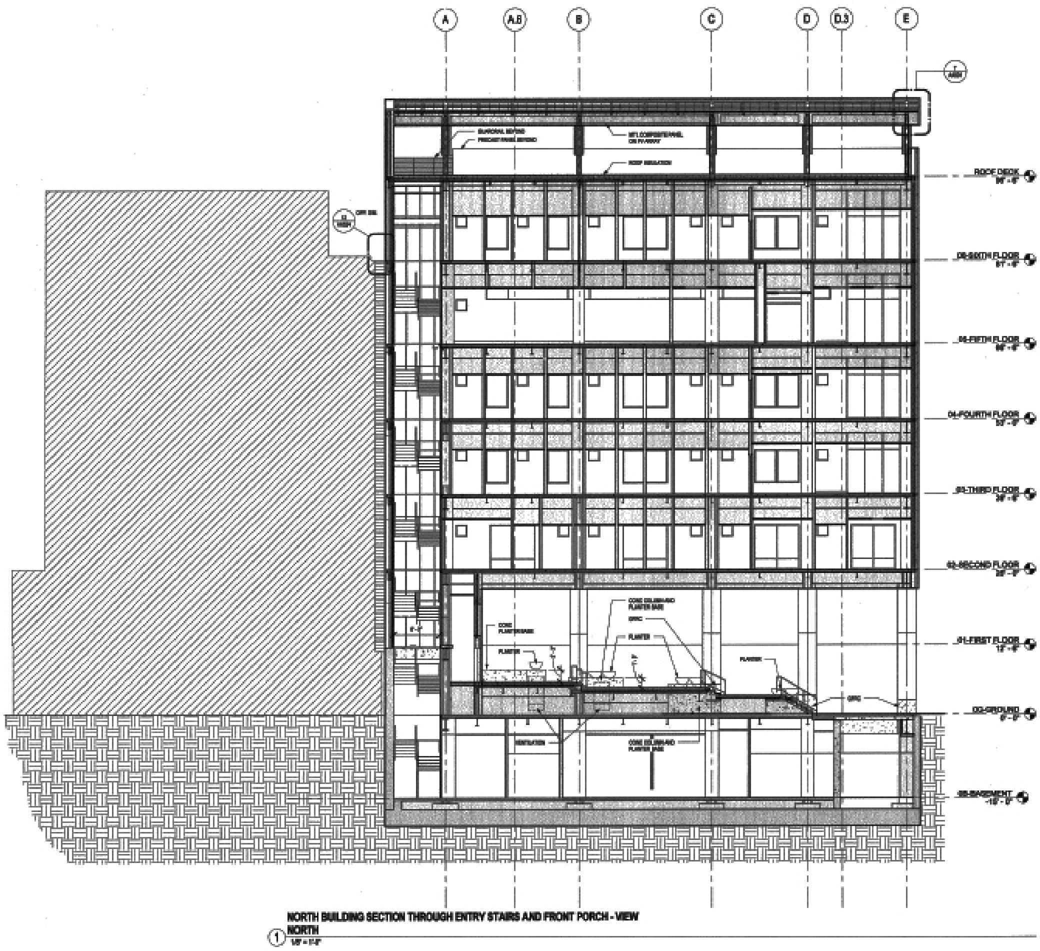

Figure 13: EW Building Section

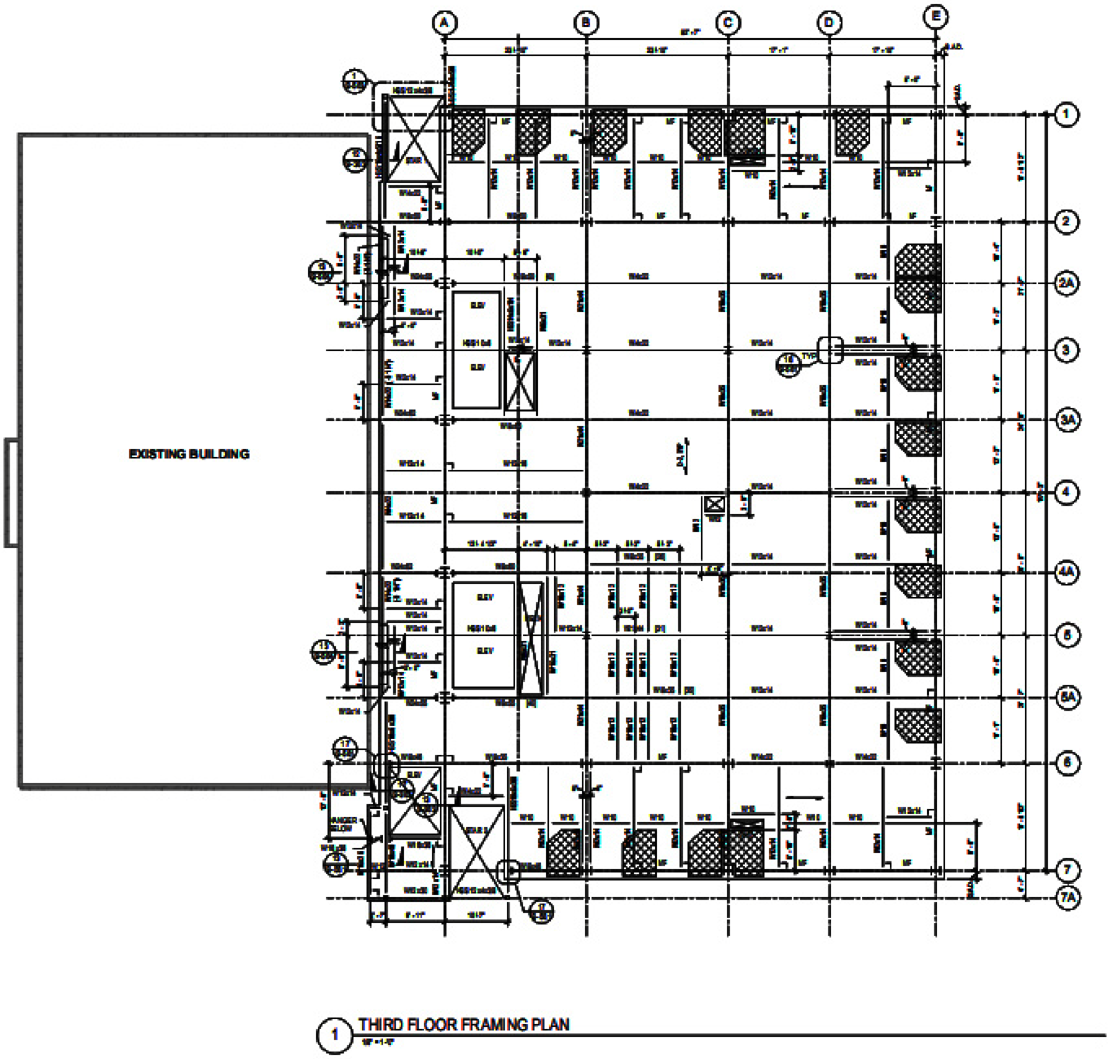

Figure 14: Typical Framing Plan

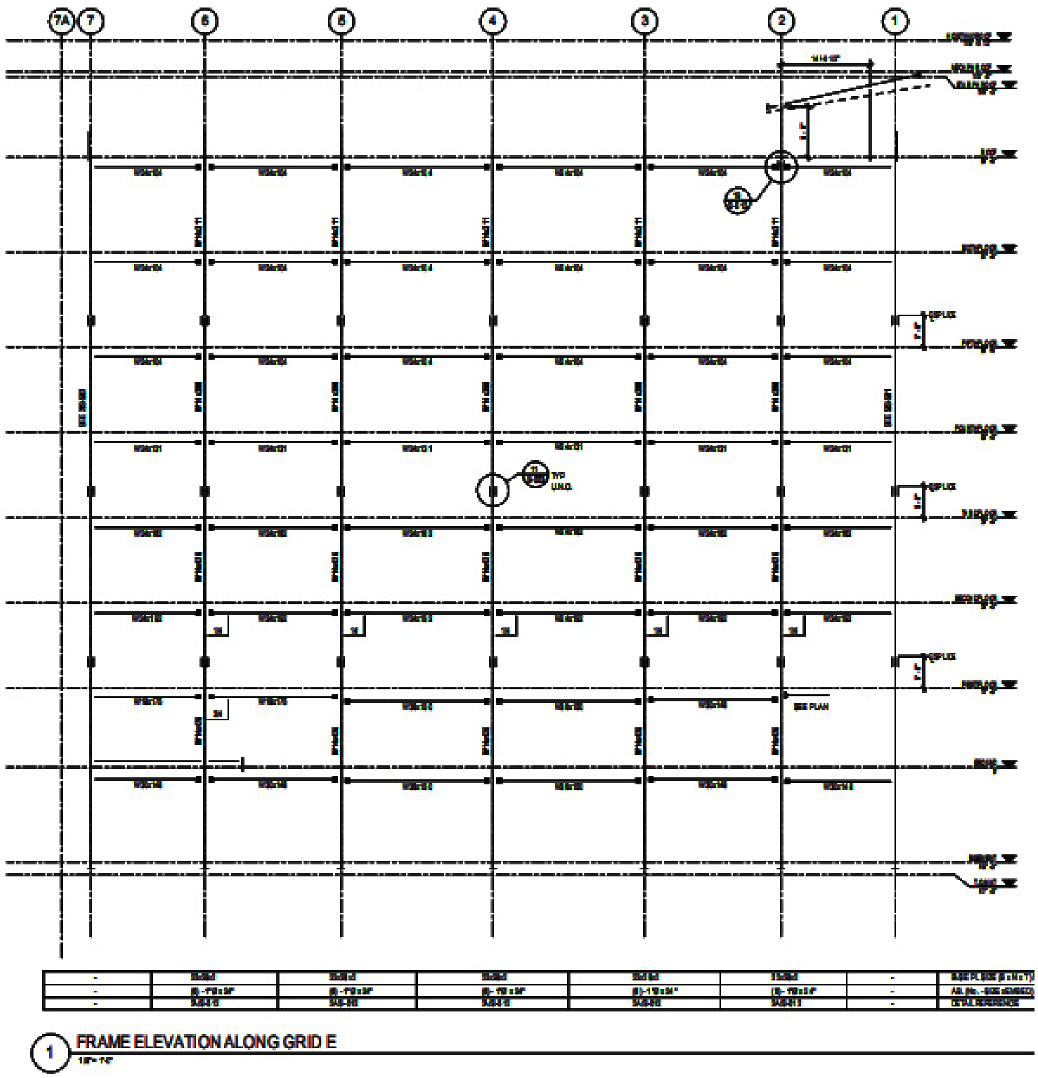

Figure 15: Typical Moment Frame Elevation