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Option: Structural
AE Faculty Consultant: Kevin Parfitt
Building: The Duncan Center
Location: Dover, DE
October 10, 2007

TECHNICAL ASSIGNMENT 1

EXECUTIVE SUMMARY

In this report, the existing structure has been analyzed by describing the structure and its components based on the contract documents. From this analysis of the structure, the loading of the building was determined.

Spot checks of the gravity and lateral systems were also performed. It was found that the gravity system appeared to be over designed slightly, which is most likely due to value engineering, selecting a deeper member of the same weight to gain capacity with the same weight of steel.

As for the lateral system, the results were very different which can be attributed to mainly a difference in the Model Codes for determining lateral loads. Also, the 9^{th} edition ASD Manual was most likely used in design versus 13^{th} edition LRFD steel manual, which could have an effect upon the available strength of the material. It was found that the columns, based on the calculations in the spot check, would fail under the combined axial and bending load, although the columns were designed to hold an extra floor that was later eliminated from the design. However this was not enough to counteract the effect of the higher lateral forces. The beams in the lateral system were also below the required capacity due to the same reasoning of Model Code difference and the difference in ASD vs. LRFD.

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I. Introduction

The Duncan Center is a premium office building located in Dover, DE. The building has a total of six floors reaching an overall height of 93'-0". The first four floors are open flex office spaces, the fifth floor is a reception and banquet hall, and the sixth floor penthouse holds building management offices and small electrical and mechanical rooms, the larger of which are located in the basement along with storage space. The fourth and fifth floors are augmented with sizable balconies and the overall structure is crowned with the arched penthouse. See additional photographs on the picture page in the Appendix.

The purpose of this report is to examine the existing structure by performing a simplified analysis in order to provide a background for more in depth future studies. The structure of the Duncan Center is predominantly moment-framed steel with 5" thick composite metal deck slabs in typical bays of 24'-5" x 27'-8". The steel frame is supported by a concrete 40' deep auger-cast pile and deep grade beam system. The veneer of the building is non-loading bearing brick and glass panel, backed with cold-formed steel studs, which is ultimately supported by the steel frame. The roof , including the arched penthouse roof, is comprised of 24" o.c. cold formed steel roof trusses. Additional calculations in support of the material presented in this report are available upon request.

II. STRUCTURAL SYSTEMS

Foundation System

The foundation system begins with auger cast concrete piles as per the recommendation of the geotechnical engineer, John D. Hynes & Associates, Inc. The structural engineer was presented with the choice of several different diameters and depths of piles and a 16" dia., 40' long pile reinforced with a cage in the top 10" of the pile of 6-#6 and #3 ties at 12" o.c. was selected, with a bearing capacity of 85 tons.

On top of these piles rest the pile caps of variant cross section with a depth of 3'-1" each, see Figure 1: Foundation Plan in Sketches. Upon the pile caps rest the 24"x24" concrete piers with 8-#8 vertical bars with #3 ties at 12" o.c. The piers are enclosed by 1' wide by 2' deep grade beams with 4-#6 bars top and bottom with #3 ties at 12" o.c., which support the 12" CMU foundation walls with 4-#4 horizontal and 4-#4 vertical reinforcement The piers are finally topped off with 18"x18" steel baseplates ranging in thickness from 1" to 2-1/4" with 4-1" dia. A325N bolts, see Figure 6: Typical Exterior Foundation Section in Sketches.

The basement slab on-grade is a 4" cast-in place concrete slab reinforced with 6x6 W2.9xW2.9 welded wire fabric on 4" of porous fill.

Floor Systems

The floor system for the Duncan Center typical on all floors is 5" composite slab with 2" 20 gage composite metal deck reinforced with 6x6 W2.0xW2.0 welded wire fabric. The deck is welded to the structural steel girders beneath with 23-3/4" x 4" long shear studs, where as the beams have 14-3/4" x 4" long shear studs. Giving the overall floor system a fire rating of 2 hours and forming a flexible diaphragm.

The typical floor bay has spans of 27'-8"x24'-5" with the beams running in the long direction, W16x31 interior and W18x35 between columns. The interior beams rest upon W24x55 girders which transfer the load to the columns which will be discussed in the Lateral Load Resisting System, see Figure 2: Second Floor Framing Plan, Figure 3: Typical Floor Framing Bay, and Figure 7: Typical Moment Connection Detail in Sketches.

Lateral Load Resisting System

The Lateral Load Resisting System is singularly comprised of the moment connected frame as each beam between columns and each girder are moment connected by double angle connections and full penetration welds to the columns, see the typical connection detail in typical section sketches. Columns range from W12x45 to W12x120 and are spliced at the third and the fifth floor, see Figure 5: Typical Moment Frame Elevation in Sketches.

Building Façade

The brick façade is supported by L5x5x3/8 typical shelf angles, supporting a total of 8' height of brick. The shelf angle is connected then to a C6x13 angle at 4' o.c. which runs parallel with the bricks and then transfers it load to a stiffener plate which is shear connected to the exterior girders and beams and an additional stiffener plate is connected to the opposite web face of the beam, see Figure 8: Typical Brick Support Section in Sketches. Besides at the support condition, the façade is backed by 6" 22 gage cold-formed steel studs at 16" o.c. which are supported L4x4x5/16 angles which transfer their load to W12x14 outriggers at 48" o.c. which are shear connected to the exterior girders and beams, see Figure 9: Typical Cold-Formed Steel Stud Support Section in Sketches.

Roof Framing

The roof framing formed of cold-formed steel trusses at 24" o.c. for both the lower flat fifth floor roof and the arched sixth floor penthouse with which rest on exterior structural steel girders, W16x26 typical at the fifth floor roof and W16x31 at the penthouse roof. Attached to the roof trusses is 20 gage galvanized Type B roof deck, see Figure 4: Roof Framing Plan in Sketches.

Special Conditions

As with any structure special conditions exist at certain locations, particularly at elevator shafts and stair towers, where special detailing. These conditions will not be analyzed in this report, but are mentioned to bring them to the attention of the reader. A few of these conditions are the entrance way framing, balcony framing, penthouse mechanical floor pads, basement mechanical floor pads, and reception and banquet hall large clear span glass outlook and post-up columns. Also, the structure must meet the criterion of exterior façade deflection not greater than L/600 of the span from attachment to steel and the vibrational requirements of the reception and banquet hall.

III. Sketches

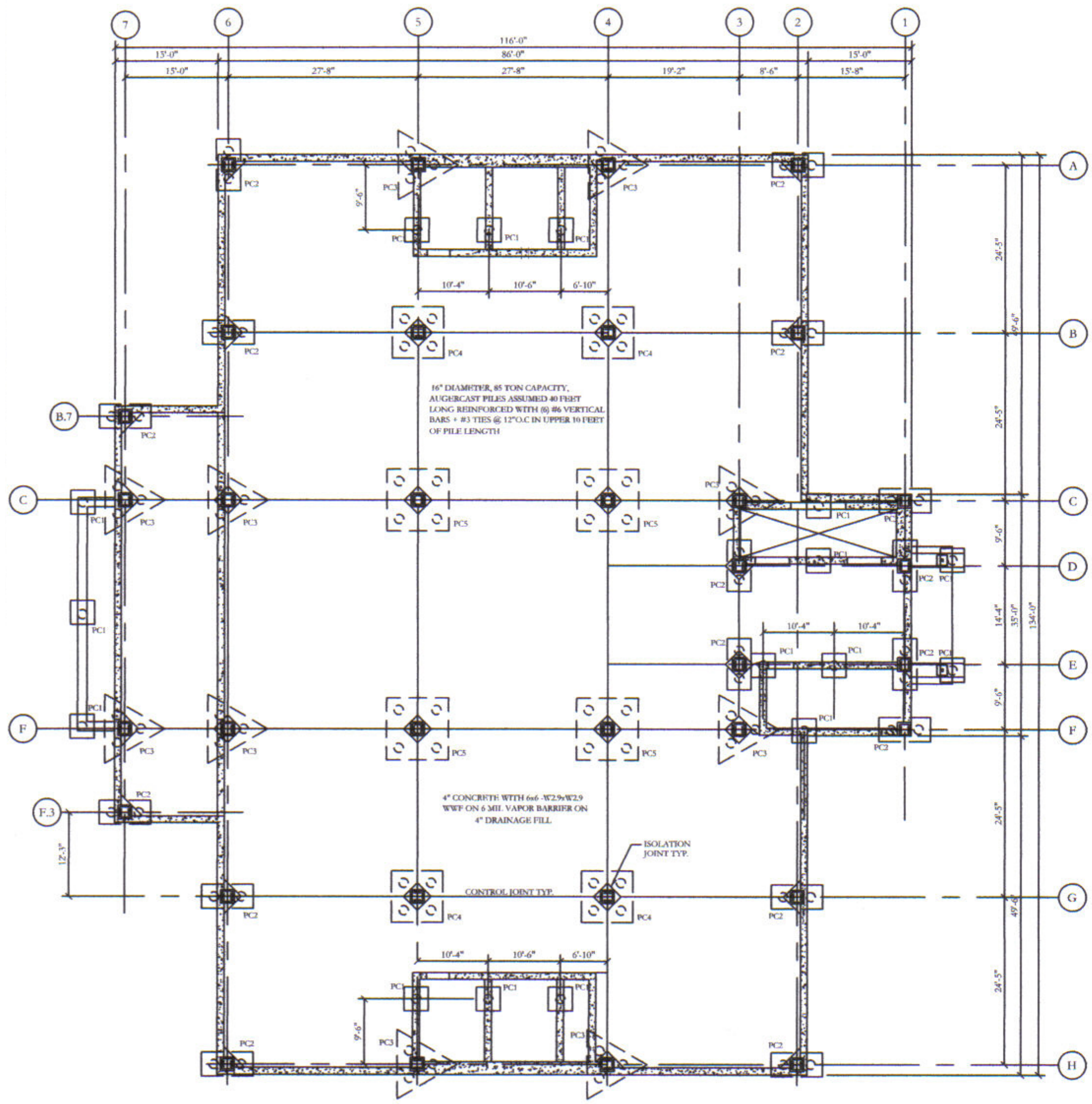


Figure 1: Foundation Plan

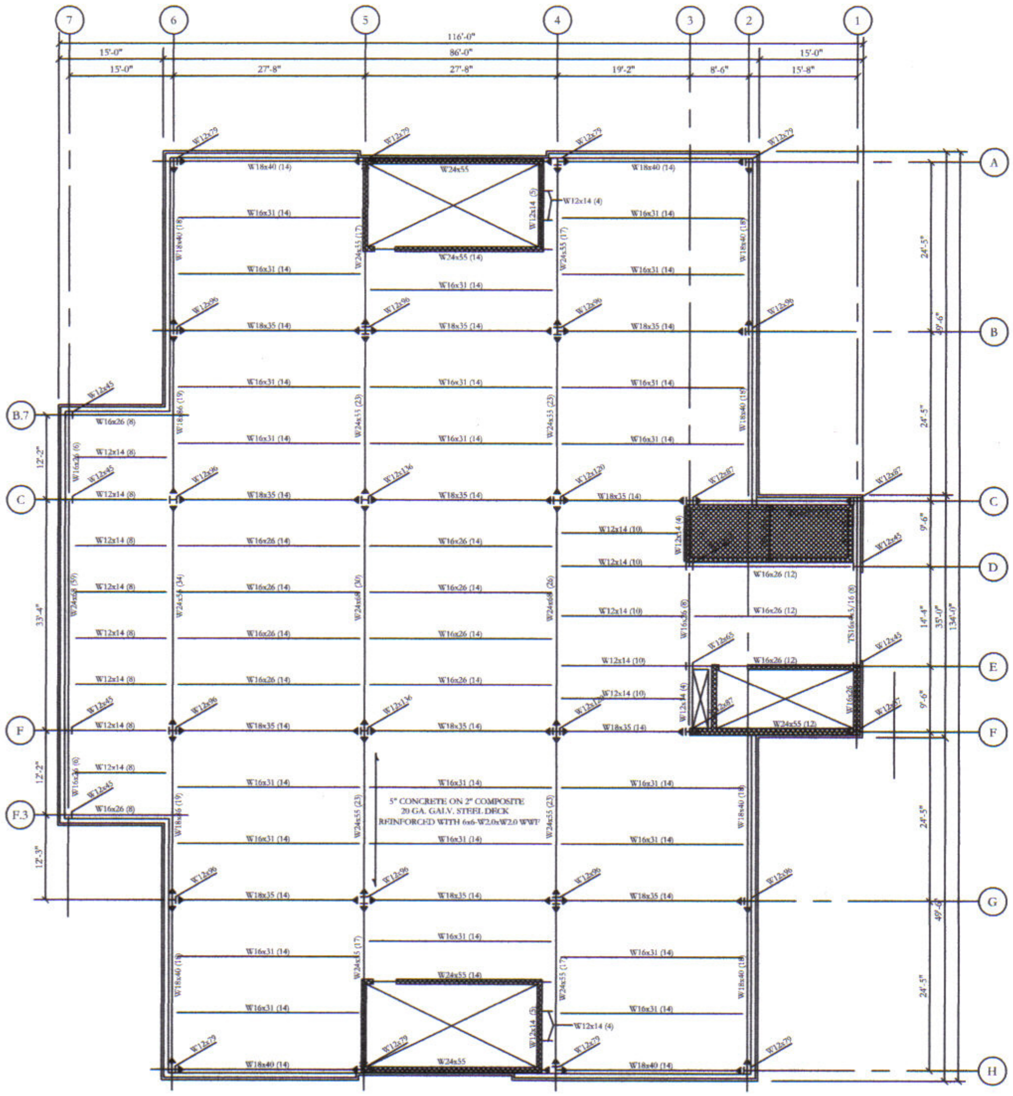


Figure 2: Second Floor Framing Plan

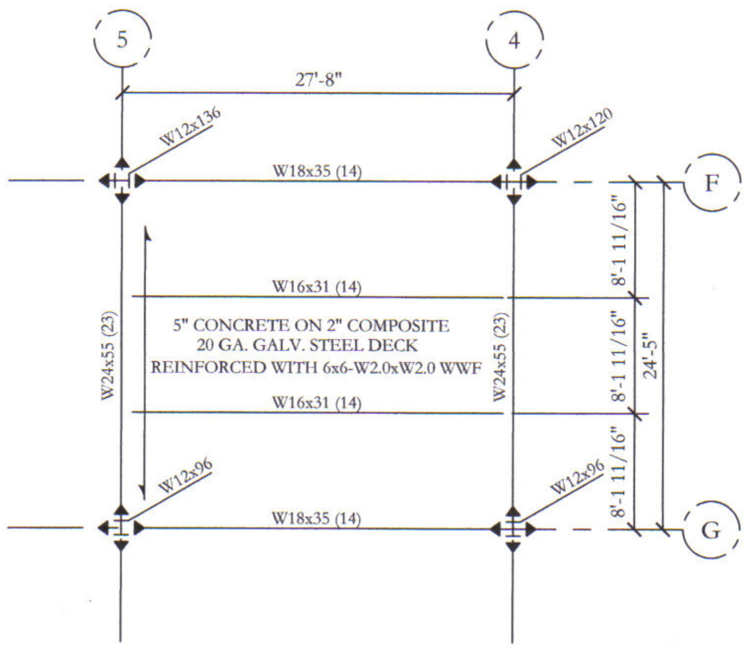


Figure 3: Typical Floor Framing Bay

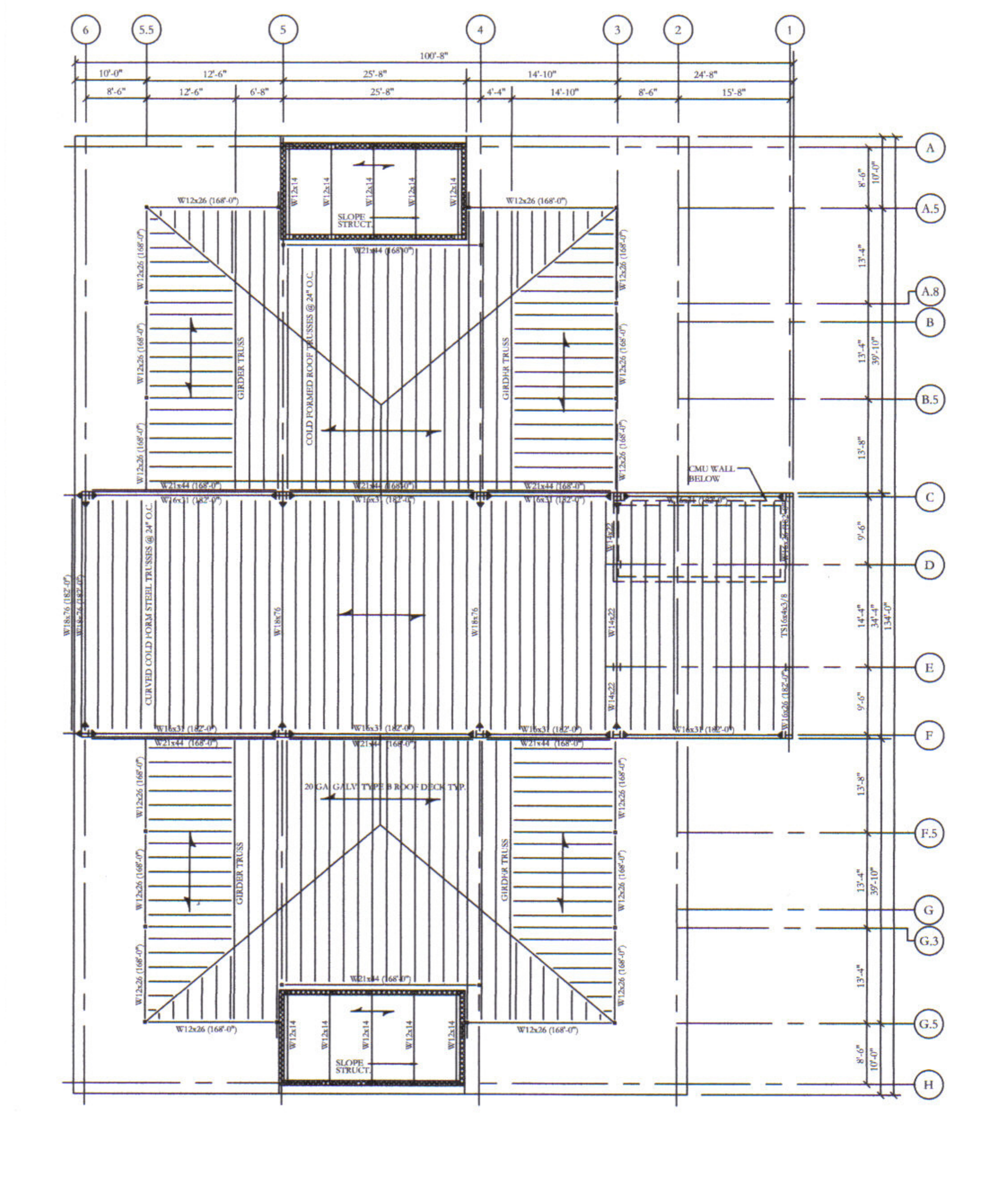


Figure 4: Roof Framing Plan

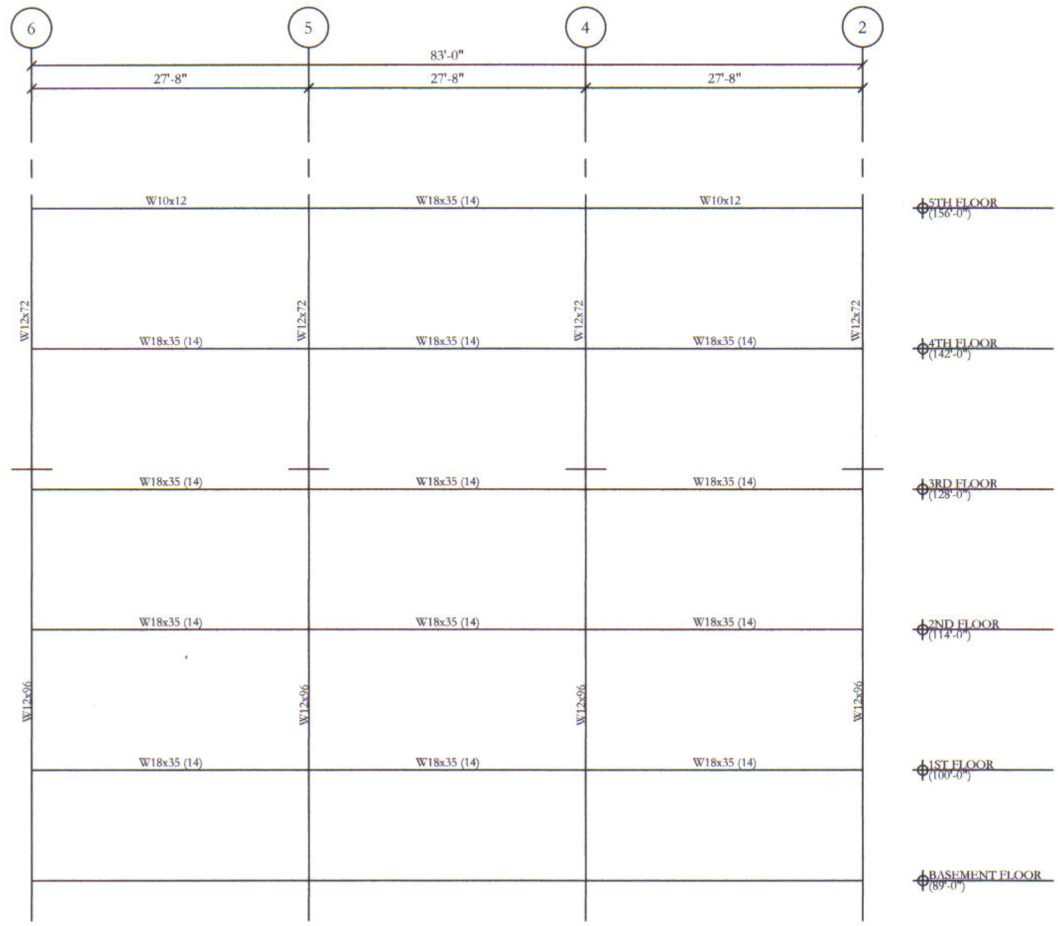


Figure 5: Typical Moment Frame Gridline G Elevation

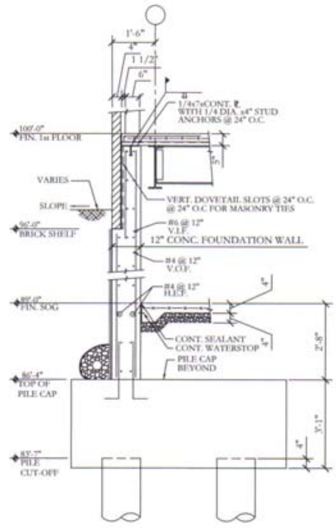


Figure 6: Typical Exterior Foundation Section

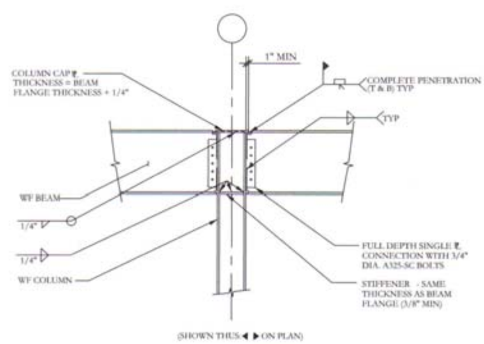


Figure 7: Typical Moment Connection Detail

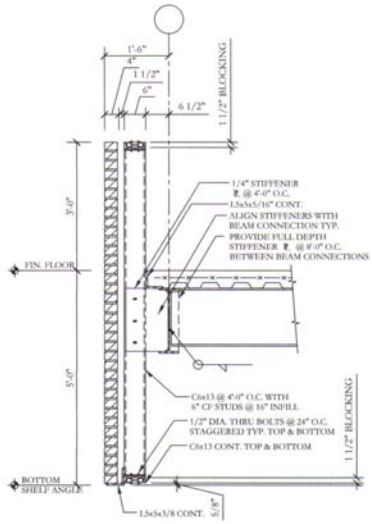


Figure 8: Typical Brick Support Section

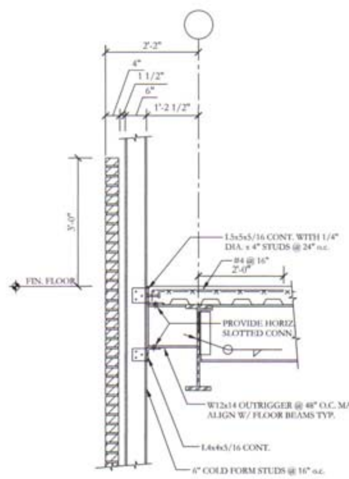


Figure 9: Typical Cold-Formed Steel Stud Support Section

IV. Codes and Materials

Design Codes and Reference Standards

National Building Code: Building Officials and Code Administrators (BOCA) 1999
"National Building Code"
Design Loads: American Society of Civil Engineers (ASCE) 7-98
"Minimum Design Loads for Buildings and Other Structures"
Steel Reference Standard: American Institute of Steel Construction (AISC) ASD 9th Edition
"Allowable Stress Design and Plastic Design Specification for Structural Steel Buildings"
Concrete Reference Standard: American Concrete Institute (ACI) 318-99
"Building Code Requirements for Structural Concrete"
Masonry Reference Standard: American Concrete Institute (ACI) 530.1-99
"Building Code Requirements & Specifications for Masonry Structures"
Reinforcement Reference Standard: American Concrete Institute (ACI) 315-99
"Details and Detailing of Concrete Reinforcement"
Metal Deck Reference Standard: Steel Deck Institute (SDI) 1999
"Design Manual for Composite Decks, Form Decks, and Roof Decks"
Cold Formed Steel Reference Standard: American Iron and Steel Institute (AISI) 1996
"Specification for the Design of Cold-Formed Steel Structural Members"

Note: Many of the versions here have been assumed based on the dates of design and construction as the specifications reads "the latest edition" for all reference standards.

Analysis Codes and Reference Standards

National Building Code: International Code Council (ICC) 2006
"International Building Code (IBC)"
Design Loads: American Society of Civil Engineers (ASCE) 7-05
"Minimum Design Loads for Buildings and Other Structures"
Steel Reference Standard: American Institute of Steel Construction (AISC) 13th Edition
"Specification for Structural Steel Buildings" (LRFD)
Concrete Reference Standard: American Concrete Institute (ACI) 318-05
"Building Code Requirements for Structural Concrete"
Masonry Reference Standard: American Concrete Institute (ACI) 530.1-05
"Building Code Requirements & Specifications for Masonry Structures"
Reinforcement Reference Standard: American Concrete Institute (ACI) 315-05
"Details and Detailing of Concrete Reinforcement"
Metal Deck Reference Standard: United Steel Deck (USD) 2006
"Steel Decks for Floors and Roofs"
Cold Formed Steel Reference Standard: American Iron and Steel Institute (AISI) 2002
"Specification for the Design of Cold-Formed Steel Structural Members"

Note: The building was designed and built under older and now out of date codes, listed above under Design Codes, however the building will be analyzed using the revised codes listed above under Analysis Codes.

Material Properties

Steel:	
W and S Shapes:	A572 Grade 50
Square or Rectangular Tubes:	A500 Grade B
Round Pipes:	A501
Miscellaneous Steel:	A36
Bolts:	A325N
Steel Studs:	A108
Weld Strength:	E70XX
Concrete:	
Foundations:	Normalweight, F_y=4000 psi, 6 +/- 1% Air Entrainment
Floor Slabs:	Normalweight, F_y=4000 psi, 6 +/- 1% Air Entrainment
Reinforcement:	A615 Grade 60
Welded Wire Fabric:	A185
Masonry:	
Concrete Masonry Units (CMU):	C90 Grade N Type I, F_m=1500 psi
Grout:	F_y=3000 psi
Reinforcement:	A615 Grade 60
Cold-Formed Steel:	
Metal Deck:	A525 Grade 60
Generic Cold-Formed Steel:	A446 Grade D (12-16 gage) A446 Grade A (18 gage and lower)
Miscellaneous Steel:	A525 Grade 60

Note: Material strengths are based on American Society for Testing and Materials (ASTM) Standard ratings.

V. Building Loads

Dead Loads

Roof	22	PSF
Balcony	78	PSF
Floor	70	PSF
Exterior Wall	55	PSF
Partition Wall	15	PSF

See Appendix pg.18 for further breakdown per loading condition.

Note: Building dead loads do not include supporting structural member self-weights.

Live Loads

Space	Load	
Roof	20	PSF
Balcony	100	PSF
Stairs and Exits	100	PSF
Corridor-First Floor	100	PSF
Corridor-Other Floors	80	PSF
Lobby	100	PSF
Dance Halls and Ballrooms	100	PSF
Office Space	50	PSF

Snow Loads

Flat Roof Snow Load

p_f=22 psf

Lower Roof Snow Drift Load

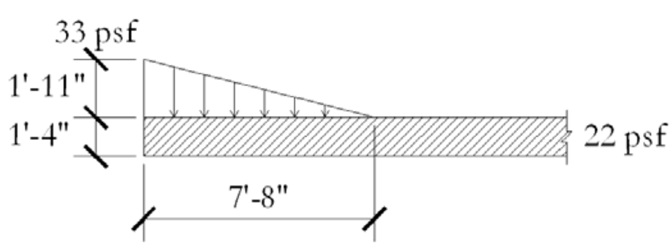
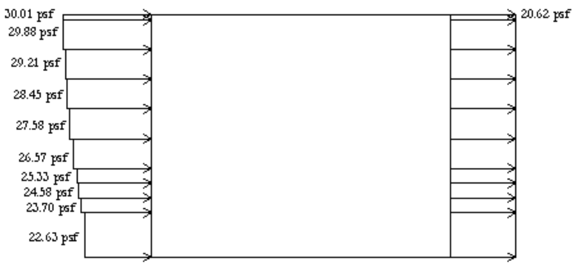


Figure 10: Snow Drift Loading Diagram

See Appendix pg.18 for calculations.

Wind Loads



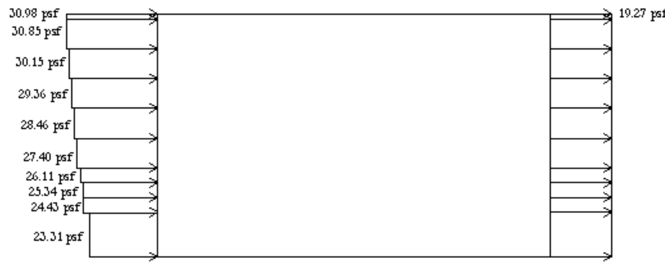
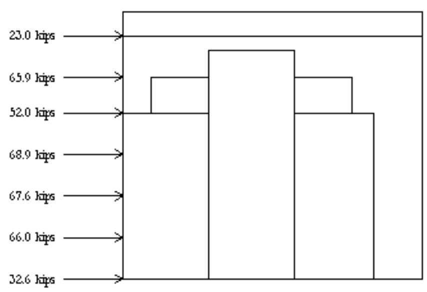


Figure 11: North-South Direction Wind Load Figure 12: East-West Direction Wind Load



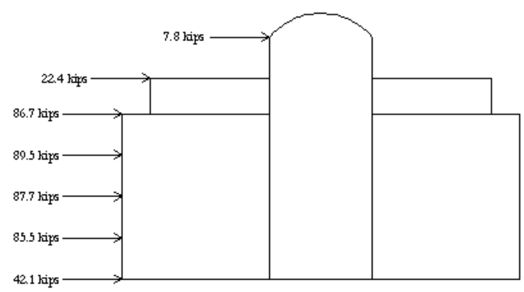


Figure 13: North-South Direction Story Shear Figure 14: East-West Direction Story Shear
See Appendix pg.19 for calculations.

Seismic Loads

Equivalent Lateral Force

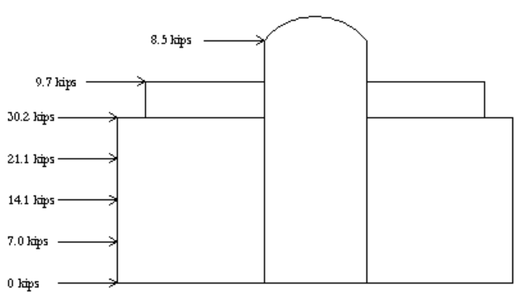


Figure 15: Story Shear
See Appendix pg. 23 for calculations.

VI. SPOT CHECKS

Gravity Floor Bay

Gravity Floor Bay Spot Check			
Member	Existing	Spot Check	Reason for Difference of Results
Joist	W16x31 (14)	W14x30 (12)	Value Engineering
Beam	W18x35 (14)	W12x16 (14)	Lateral Member
Girder	W24x55 (23)	W14x26 (24)	Lateral Member
Column	W12x96	Passed	NA

See Appendix pg. 27 for calculations.

Lateral Frame

Lateral Frame Spot Check			
Member	Existing	Spot Check	Reason for Difference of Results
Beam	W18x35 (14)	W16x45 (26)	Code Variance
Column	W12x96	Failed	Code Variance

See Appendix pg. 30 for calculations.

VII. Appendix

Dead Loads

Roof			Balcony		
Metal Roof Sheathing	1	PSF	Concrete Pavers	12	PSF
4" Rigid Insulation	6	PSF	4" Rigid Insulation	6	PSF
Steel Deck	2	PSF	Concrete	48	PSF
Light-gage Steel Trusses	3	PSF	Deck	2	PSF
HVAC	3	PSF	HVAC	3	PSF
Acoustical Ceiling Tile	2	PSF	Acoustical Ceiling Tile	2	PSF
Miscellaneous	5	PSF	Miscellaneous	5	PSF
Total	22	PSF	Total	78	PSF
Floor			Exterior Wall		
Quarry Tile Flooring	10	PSF	4" Brick Façade	40	PSF
Concrete	48	PSF	5/8" Gypsum Board	3	PSF
Steel Deck	2	PSF	6" Batt Insulation	6	PSF
HVAC	3	PSF	5/8" Gypsum Board	3	PSF
Acoustical Ceiling Tile	2	PSF	Miscellaneous	3	PSF
Miscellaneous	5	PSF			
Total	70	PSF	Total	55	PSF
Partition Wall					
5/8" Gypsum Board	3	PSF			
6" Batt Insulation	6	PSF			
5/8" Gypsum Board	3	PSF			
Miscellaneous	3	PSF			
Total	15	PSF			

Snow Loads

Flat Roof Snow Load
Terrain Category C
$C_e=0.9$
$C_t=1.0$
$I=1.1$
$p_g=25$ psf
p_f equals the larger of:
$p_f=0.7\ C_e\ C_t\ I\ p_g$

$=(0.7)(0.9)(1.0)(1.1)(25\ psf)$
$=18\ psf$
$p_f=20I$
$=20(1.1)$
$=22\ psf$
$p_f=22\ psf>LL=20\ psf$ Roof Snow Load Controls

Lower Roof Snow Drift Load
$\gamma=0.13\ p_g+14$
$=(0.13)(25\ psf)+14$
$=17.3\ pcf<30\ pcf$ OK
$h_b= p_f/\gamma$
$=22\ psf/17.3\ pcf$
$=1.27\ ft$
$h_c=14\ ft-1.27\ ft$
$=12.7\ ft$
$h_c/ h_b=12.7\ ft/1.27\ ft$
$=10>0.2$ Snow drift required.
h_d equals larger of:
higher roof, $l_u=34.67\ ft$
$h_d=0.43(l_u^{1/3})((p_g+10)^{1/4})-1.5$
$=0.43(34.67\ ft^{\ 1/3})((25\ psf+10)^{1/4})-1.5$
$=1.91\ ft$
lower roof, $l_u=49.67\ ft$
$h_d=0.75[0.43(l_u^{1/3})((p_g+10)^{1/4})-1.5$
$=0.43(49.67\ ft^{\ 1/3})((25\ psf+10)^{1/4})-1.5]$
$=1.78\ ft$
$h_d=1.91\ ft< h_c=12.7\ ft$
$w=4\ h_d$
$=4(1.91\ ft)$
$=7.64\ ft<8\ h_c=8(12.7\ ft)=101.6\ ft$ OK
$p_d= h_d\ \gamma$
$=(1.91\ ft)(17.3\ pcf)$
$=33\ psf$

Wind Loads

Main Wind Force Resisting System
$V=100\ mph$
$K_d=0.85$
Occupancy Category III
$I=1.15$
Exposure Category C
$15\ ft<z=82\ ft< z_g=900\ ft$
$\alpha=9.5$
$K_z=2.01(z/z_g)^{2/\alpha}$ (see table below)

K_{zt}=1.0
C_t=0.028
h_n=82 ft
x=0.8
$T_a=C_t h_n^x$
=(0.028)(82 ft)$^{0.8}$
=0.951 s
T≤ C_u T_a=(1.7)(0.951 s)=1.62 s
f=1/T
=1/1.57 s
=0.637 H_z<1.0 H_z Flexible Building
North-South Direction
c=0.20
z=0.6h
=0.6(82 ft)
=49.2 ft>z_{min}=15 ft OK
$I_z=c(33/z)^{1/6}$
=(0.20)(33/49.2 ft) $^{1/6}$
=0.187
g_Q=3.4
B=132.67 ft
h=82 ft
ℓ=500
ε=1/5.0
$L_z= \ell(33/z)^{\varepsilon}$
=500(33/49.2 ft) $^{(1/5.0)}$
=462 ft
Q=(1/(1+0.63((B+h)/ L_z) $^{0.63}$) $^{1/2}$
=(1/1+0.63((132.67 ft+82 ft)/462) $^{0.63}$)$^{1/2}$
=0.849
n_1=f
=0.637 H_z
g_R=(2ln(3600n_1)$^{1/2}$+(0.577/(2ln(3600n_1) $^{1/2}$)
=(2ln(3600(0.637)) $^{1/2}$+(0.577/(2ln(3600(0.637)) $^{1/2}$)
=3.94
Assuming β=0.02
b=0.65
α=1/6.5
$V_z=b(z/33)^{\alpha}V(88/60)$
=(0.65)(49.2 ft/33)$^{(1/6.5)}$(100 mph)(88/60)
=101 mph
$N_1=n_1V_z/L_z$
=(0.637)(101 mph)/462 ft
=0.139
$R_n=7.47N_1/(1+10.3N_1)^{5/3}$
=7.47(0.139)/(1+10.3(0.139)) $^{5/3}$=

$=0.236$

$R_h=(1/(4.6n_1h/V_z))-((1/2(4.6n_1h/V_z)^2)(1-e^{-2(4.6n1h/Vz)}))$

$= (1/(4.6(0.637)(82\ ft)/101\ mph))$

$-((1/2(4.6(0.637)(82\ ft)/101\ mph))^2)(1-e^{-2(4.6(0.637)(82\ ft)/(101\ mph))}))$

$=0.333$

$R_B=(1/(4.6n_1B/V_z))-((1/2(4.6n_1B/V_z)^2)(1-e^{-2(4.6n1B/Vz)}))$

$= (1/(4.6(0.637)(132.67\ ft)/101\ mph))$

$-((1/2(4.6(0.637)(132.67\ ft)/101\ mph)^2)(1-e^{-2(4.6(0.637)(132.67\ ft)/101\ mph)}))$

$=0.226$

L=101.25 ft

$R_L=(1/(15.4n_1L/V_z))-((1/2(15.4n_1L/V_z)^2)(1-e^{-2(15.4n1L/Vz)}))$

$= (1/(15.4(0.637)(101.25\ ft)/101\ mph))$

$-((1/2(15.4(0.637)(101.25\ ft)/101\ mph)^2)(1-e^{-2(15.4(0.637)(101.25\ ft)/101\ mph)}))$

$=0.097$

$R=((1/\beta)R_nR_hR_B(0.53+0.47R_L))^{1/2}$

$=((1/0.02)(0.236)(0.333)(0.226)(0.53+0.47(0.097))^{1/2}$

$=0.715$

$g_V=3.4$

$G=0.925((1+1.7I_z(g_Q{}^2Q^2+g_R{}^2R^2)^{1/2})/(1+1.7g_VI_z))$

$=0.925((1+1.7(0.187)((3.4)^2(0.849)^2+(3.94)^2(0.715)^2)^{1/2})/(1+1.7\ (3.4)(0.187)))$

$=1.01$

East-West Direction (see North-South Direction for other values)

B=101.25 ft

$Q=(1/(1+0.63((B+h)/\ L_z)^{0.63})^{1/2}$

$=(1/1+0.63((101.25\ ft+82\ ft)/462)^{0.63})^{1/2}$

$=0.860$

$R_B=(1/(4.6n_1B/V_z))-((1/2(4.6n_1B/V_z)^2)(1-e^{-2(4.6n1B/Vz)}))$

$= (1/(4.6(0.637)(101.25\ ft)/101\ mph))$

$-((1/2(4.6(0.637)(101.25\ ft)/101\ mph)^2)(1-e^{-2(4.6(0.637)(101.25\ ft)/101\ mph)}))$

$=0.283$

L=132.67 ft

$R_L=(1/(15.4n_1L/V_z))-((1/2(15.4n_1L/V_z)^2)(1-e^{-2(15.4n1L/Vz)}))$

$= (1/(15.4(0.637)(132.67\ ft)/101\ mph))$

$-((1/2(15.4(0.637)(132.67\ ft)/101\ mph)^2)(1-e^{-2(15.4(0.637)(132.67\ ft)/101\ mph)}))$

$=0.075$

$R=((1/\beta)R_nR_hR_B(0.53+0.47R_L))^{1/2}$

$=((1/0.02)(0.236)(0.333)(0.283)(0.53+0.47(0.075))^{1/2}$

$=0.793$

$G=0.925((1+1.7I_z(g_Q{}^2Q^2+g_R{}^2R^2)^{1/2})/(1+1.7g_VI_z))$

$=0.925((1+1.7(0.187)((3.4)^2(0.849)^2+(3.94)^2(0.793)^2)^{1/2})/(1+1.7\ (3.4)(0.187)))$

$=1.05$

Enclosed Structure

$GC_{pi}=\pm 0.18$

Windward

Cp=0.8

Leeward, North-South Direction

L=101.25 ft

B=132.67 ft
L/B=101.25 ft/132.67 ft
=0.763
C_p=-0.5
Leeward, East-West Direction
L=132.67 ft
B=101.25 ft
L/B=132.67 ft/101.25 ft
=1.310
C_p=-0.438
$q_z=0.00256\ K_z\ K_{zt}\ K_d V^2 I$ (see table below)
$q=q_z$ windward
$=q_h$ leeward
$q_i=q_h$
$P=qG\ C_p-q_i(GC_{pi})$ (see table below)

			P (psf)			
			North-South Direction		East-West Direction	
z (ft)	Kz	qz (psf)	Windward	Leeward	Windward	Leeward
82	1.21	30.4	30.01	-20.62	30.98	-19.27
80	1.21	30.2	29.88	-20.62	30.85	-19.27
70	1.17	29.4	29.21	-20.62	30.15	-19.27
60	1.14	28.4	28.45	-20.62	29.36	-19.27
50	1.09	27.4	27.58	-20.62	28.46	-19.27
40	1.04	26.1	26.57	-20.62	27.40	-19.27
30	0.98	24.6	25.33	-20.62	26.11	-19.27
25	0.95	23.7	24.58	-20.62	25.34	-19.27
20	0.90	22.6	23.70	-20.62	24.43	-19.27
15	0.85	21.2	22.63	-20.62	23.31	-19.27
0	0.00	0.0	5.47	-20.62	5.47	-19.27

Story	Height (ft)	Tributary Height Above (ft)	Tributary Height Below (ft)	Tributary Height (ft)
Roof	82	2	2.5	4.5
6	73	4.5	8.5	13.0
5	56	8.5	7.0	15.5
4	42	7.0	7.0	14.0
3	28	7.0	7.0	14.0
2	14	7.0	7.0	14.0
1	0	7.0	NA	7.0

Story Width		Story Shear (kips)	
North-South Direction	East-West Direction	North-South Direction	East-West Direction
101.25	34.67	23.0	7.8
101.25	34.67	65.9	22.4
67.75	114.00	52.0	86.7
101.25	132.67	68.9	89.5
101.25	132.67	67.6	87.7
101.25	132.67	66.0	85.5
101.25	132.67	32.6	42.1

Cumulative Shear (kips)		Overturning Moment (kip*ft)	
North-South Direction	East-West Direction	North-South Direction	East-West Direction
23.0	7.8	1889.1	641.9
88.9	30.2	6492.1	2205.4
140.9	116.9	7890.4	6546.4
209.8	206.4	8813.5	8668.1
277.5	294.1	7768.8	8233.5
343.5	379.5	4808.6	5313.6
376.0	421.7	0.0	0.0

Seismic Loads

Latitude: 39.17° N
Longitude: -75.54 ° W
From USGS Java Ground Motion Parameter Calculator
S_s=0.172
S_1=0.079
Assuming Site Class D (Not reported in Geotechnical Engineer's Report)
F_a=1.6
F_v=2.4

$S_{MS}=F_aS_s$
$=(1.6)(0.172)$
$=0.275$
$S_{M1}=F_vS_1$
$=(2.4)(0.079)$
$=0.190$
$S_{DS}=2/3\ S_{MS}$
$=(2/3)(0.275)$
$=0.183$
$S_{D1}=2/3\ S_{M1}$
$=(2/3)(0.190)$
$=0.127$

T_L=6 s
C_u=1.65
C_t=0.028
h_n=82 ft
x=0.8
$T_a=C_t h_n^x$
$=(0.028)(82\ ft)^{0.8}$
=0.951 s
$T \leq C_u T_a$
=(1.65)(0.951 s)
=1.57 s
Occupancy Category III
I=1.25
Seismic Design Category B
Special Steel Moment Frames
R=8
C_s equals the smallest of:
$C_s = S_{DS}/(R/I)$
=(0.183)/(8/1.25)
=0.029
$T=1.62\ s<T_L=6\ s$
$C_s = S_{D1}/(T(R/I))$
=(0.127)/(1.62(8/1.25))
=0.012
$S_1=0.079>0.6$
$C_s = S_1/(R/I)$
=(0.079)/(8/1.25)
=0.012
C_s=0.012>0.01   OK
$V= C_s W$
=(0.012)(7557 kips)
=90.7 kips
k=1.56
$C_{vx}=w_x h_x^k/\Sigma w_i h_i^k$
$F_x=C_{vx}V$

Story	Floor Area (sf)	Floor Dead Load (psf)	Floor Self-Weight (psf)
Roof	8138	22	5
6	2179	70	10
Balcony	4772	78	10
5	8138	70	10
4	12910	70	10
3	12910	70	10
2	12910	70	10
1	12910	70	10

Story	Tributary Wall Height (ft)	Wall Perimeter (ft)	Wall Dead Load (psf)
Roof	4.5	271.84	55
6	13.0	271.84	55
Balcony	5.0	467.84	48
5	15.5	363.50	55
4	14.0	467.84	55
3	14.0	467.84	55
2	14.0	467.84	55
1	7.0	467.84	55

Story	Total Floor Weight (kips)
Roof	287
6	369
Balcony	532
5	961
4	1393
3	1393
2	1393
1	1213
Total Building Weight	7541

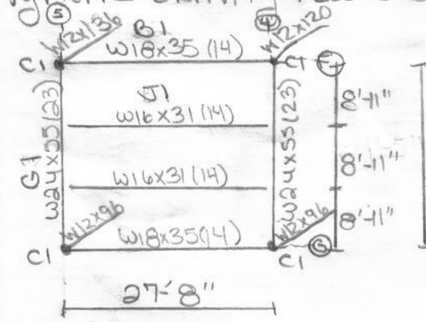
Story Shear							
Story	wx (kips)	hx (ft)	k	wxhx^k	Cvx	V (kips)	Fx (kips)
Roof	287	82	1.56	36714	0.09373	90.7	8.5
6	369	73	1.56	41986	0.10719	90.7	9.7
5	1493	56	1.56	130441	0.33303	90.7	30.2
4	1393	42	1.56	91272	0.23302	90.7	21.1
3	1393	28	1.56	60848	0.15535	90.7	14.1
2	1393	14	1.56	30424	0.07767	90.7	7.0
1	1213	0	1.56	0	0.00000	90.7	0.0
Totals	7541	NA	NA	391684	1	NA	90.7

Story	Overturning Moment (kip*ft)
Roof	697.1
6	709.7
Balcony	1691.5
5	887.7
4	394.5
3	98.6
2	0.0

Wind Base Shear=1.6(376 kips)=602 kips>Seismic Base Shear=90.7 kips (Wind Controls)

Gravity Floor Bay Spot Check

TYPICAL GRAVITY FLOOR BAY SPOT CHECK



A572 GRADE 50 STEEL, $f_y = 50$ KSI
A108 3/4" x 4" SHEAR STUDS, $Q_n = 14.6^k$ / 21.5^k
5" CONCRETE SLAB, $f'c = 4000$ PSI
2" METAL DECK

LOADS:

$DL = 70$ PSF

$LL = 50$ PSF

$LLc = (145\ PCF)(5")(1/12) = 60.4$ PSF

J1. BEAM DESIGN

ORIGINAL: W16x31 (14)

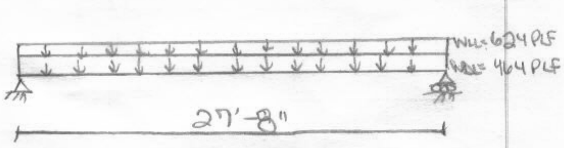
SPAN = 27'-8"

LOADS:

$W_{DL} = (70\ PSF)(8.92') = 624$ PLF

$W_{SW} = 31$ PLF

$W_{LL} = (50\ PSF)(8.92') = 446$ PLF



TOTAL $= 1.2(W_{DL}+W_{SW}) + 1.6W_{LL} = 1.2(624\ PLF + 31\ PLF) + 1.6(446\ PLF) = 1500$ PLF

$V = \frac{wl}{2} = \frac{(1500\ PLF)(27.67')}{2} = 20.8^k$

$M = \frac{wl^2}{8} = \frac{(1500\ PLF)(27.67')^2}{8} = 144^{k'}$

$b_{eff} = 8.92'$

$\frac{S}{4} = \frac{27.67'}{4} = 6.92'$ | MIN

$b_{eff} = 6.92'$

ASSUMING $a = 1"$

$Y2 = 5" - \frac{1"}{2} = 4.5"$

W12x16, P.N.A. BFL

$\phi M_n = 149^{k'} > 144^{k'}$

$\Sigma Q_n = 130^k$

$a = \frac{\Sigma Q_n}{0.85 f'c\, b_{eff}} = \frac{130^k}{0.85(4000 PSI)(6.92')(12)} = 0.460" < 1"$ ✓ OK

$Y2 = 5" - \frac{0.460"}{2} = 4.77"$

$\phi M_n = 152^{k'} > 144^{k'}$ ✓ OK

$\Delta L = \frac{5wl^4}{384EI_{LB}} = \frac{5(446\ PLF)(27.67')^4(1728)}{384(29000\ KSI) I_{LB}} \leq \frac{l}{360} = \frac{(27.67')(12)}{360} = 0.92"$

$I_{LB} = 220\ IN^4 \leq 298\ IN^4$ ✗ NO

$\Delta T = \frac{5wl^4}{384EI_{LB}} = \frac{5(624\ PLF + 31\ PLF + 446\ PLF)(27.67')^4(1728)}{384(29000\ KSI) I_{LB}} \leq \frac{l}{240} = \frac{(27.67')(12)}{240} = 1.38"$

$I_{LB} = 363\ IN^4 > 298\ IN^4$ ✗ NO

$W_{LLC} = (60.4\ PSF)(8.92') = 539\ PLF$

$\Delta_C = \frac{5wl^4}{384EI} = \frac{5(539\ PLF)(27.67')^4(1728)}{384(29000\ KSI)I} \leq \frac{l}{360} = \frac{(27.67')(12)}{360} = 0.92''$

$I = 266\ IN^4 > 103\ IN^4$ X NO

W14X30, P.N.A. 7

$\phi M_n = 257^{K'} > 144^{K'}$ ✓ OK

$\Sigma Q_n = 111^K$

$a = 0.393 < 0.460''$ ✓ OK

$Y_2 = 5'' - \frac{0.393''}{2} = 4.80''$

$\phi M_n = 257^{K'} > 144^{K'}$ ✓ OK

$I_{LB} = 535\ IN^4 > 363\ IN^4$ ✓ OK

$I = 291\ IN^4 > 266\ IN^4$ ✓ OK

STUDS $= \frac{\Sigma Q_n}{Q_n} = \frac{111^K}{21.5^K} = 5.16 \rightarrow 6 \rightarrow 12$

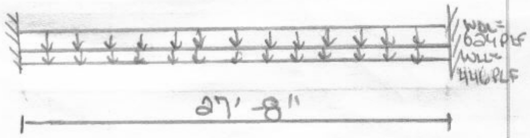
W14X30 (12)

B1. BEAM DESIGN

ORIGINAL: W18x35 (14)

SPAN: 27'-8"

LOADS:



$W_{DL} = (70\ PSF)(8.92') = 624\ PLF$

$W_{SW} = 35\ PLF$

$W_{LL} = (50\ PSF)(8.92') = 446\ PLF$

TOTAL $= 1.2(W_{DL} + W_{SW}) + 1.6\,W_{LL} = 1.2(624\ PLF + 35\ PLF) + 1.6(446\ PLF) = 1505\ PLF$

$V = \frac{wl}{2} = \frac{(1505\ PLF)(27.67')}{2} = 20.8^K$

$M = \frac{wl^2}{12} + 0.10\,wl^2 = \frac{(1505\ PLF)(27.67')^2}{12} + 0.10(1505\ PLF)(27.67')^2 = 211^{K'}$

$b_{eff} = 8.92'$

$\frac{S}{4} = \frac{27.67'}{4} = 6.92'$ | MIN

$b_{eff} = 6.92'$

ASSUMING $a = 1''$

$Y_2 = 5'' - \frac{1''}{2} = 4.5''$

W12X26 P.N.A. BFL

$\phi M_n = 218^{K'} > 211^{K'}$ ✓ OK

$\Sigma Q_n = 136^K$

$a = \frac{\Sigma Q_n}{0.85 f'_c b_{eff}} = \frac{136^K}{0.85(4000\ PSI)(6.92')(12)} = 0.482'' < 1''$ ✓ OK

$Y_2 = 5'' - \frac{0.482''}{2} = 4.76''$

$\phi M_n = 221^{K'} > 211^{K'}$ ✓ OK

$\Delta_L = \frac{1.2wl^4}{384EI_{LB}} = \frac{1.2(446\ PLF)(27.67')^4(1728)}{384(29000\ KSI)I_{LB}} \leq \frac{l}{360} = \frac{(27.67')(12)}{360} = 0.92''$

$I_{LB} = 52.9\ IN^4 < 441\ IN^4$ X NO

$\Delta_T = \frac{1.2wl^4}{384I_{LB}} = \frac{1.2(624\ PLF + 35\ PLF + 446\ PLF)(27.67')^4(1728)}{384(29000\ KSI)I_{LB}} \leq \frac{l}{240} = \frac{(27.67')(12)}{240} = 1.38''$

$I_{LB} = 87.4\ IN^4 < 441\ IN^4$ ✓ OK

$W_{LLC} = (60.4\ PSF)(8.92') = 539\ PLF$

$\Delta_C = \frac{1.2wl^4}{384EI} = \frac{1.2(539\ PLF)(27.67')^4(1728)}{384(29000\ KSI)I} \leq \frac{l}{360} = \frac{(27.67')(12)}{360} = 0.92''$

$I = 63.9\ IN^4 < 204\ IN^4$ ✓ OK

$\#STUDS = \frac{\Sigma Q_n}{Q_n} = \frac{136^k}{21.5^k} = 6.33 \rightarrow 7 \rightarrow 14$

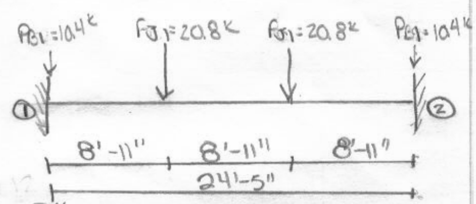
W12x16 (14)

G1. GIRDER DESIGN

ORIGINAL: W24x55 (23)

SPAN: 24'-5"

LOADS:



$P_{J1} = 20.8^k$

$P_{B1} = \frac{20.8^k}{2} = 10.4^k$

$V_1 = 1.07P_{J1} + P_{B1} = 1.07(20.8^k) + 10.4^k = 32.7^k$

$V_2 = 0.93P_{J1} + P_{B1} = 0.93(20.8^k) + 10.4^k = 29.7^k$

$M_1 = 0.222P_{J1}l + 0.281P_{J1}l = 0.222(20.8^k)(24.42') + 0.281(20.8^k)(24.42') = 255^{k'}$

$M_2 = 0.222P_{J1}l + 0.211P_{J1}l = 0.222(20.8^k)(24.42') + 0.211(20.8^k)(24.42') = 220^{k'}$

$b_{eff} = \frac{27.67'}{2} = 13.84'$

$\frac{S}{4} = \frac{24.42'}{4} = 6.11'$ MIN

$b_{eff} = 6.11'$

ASSUMING $a = 1''$

$Y2 = 5'' - \frac{1''}{2} = 4.5''$

W14x26, P.N.A. BFL

$\phi M_n = 256^{k'}$

$\Sigma Q_n = 174^k$

$a = \frac{\Sigma Q_n}{0.85f'_c b_{eff}} = \frac{174^k}{0.85(4000\ PSI)(6.11')(12)} = 0.698'' < 1''$ ✓ OK

$Y2 = 5'' - \frac{0.698''}{2} = 4.65''$

$\phi M_n = 258^{k'} > 255^{k'}$ ✓ OK

$\Delta_L = \frac{0.018Pl^3}{EI_{LB}} = \frac{0.018(6.17^k)(24.42')^3(1728)}{(29000\ KSI)I_{LB}} \leq \frac{l}{360} = \frac{(24.42')(12)}{360} = 0.81''$

$I_{LB} = 319\ IN^4 < 567\ IN^4$ ✓ OK

$\Delta_T = \frac{0.018Pl^3}{EI_{LB}} = \frac{0.018(15.2^k)(24.42')^3(1728)}{(29000\ KSI)I_{LB}} \leq \frac{l}{240} = \frac{(24.42')(12)}{240} = 1.22''$

$I_{LB} = 196\ IN^4 < 567\ IN^4$ ✓ OK

$W_C = 7.46^k$

$\Delta_C = \frac{0.018Pl^3}{EI} = \frac{0.018(7.46^k)(24.42')^3(1728)}{(29000\ KSI)I} \leq \frac{l}{360} = \frac{(24.42')(12)}{360} = 0.81''$

$I = 144\ IN^4 < 245\ IN^4$ ✓ OK

$\#STUDS = \frac{\Sigma Q_n}{Q_n} = \frac{174^k}{14.6^k} = 11.9 \rightarrow 12 \rightarrow 24$

W14x26 (24)

C1. COLUMN CHECK

ORIGINAL: W12x96, W12x120, W12x136

HEIGHT: 41'-0"

LOADS:

$P_r = 5\left(\frac{P_{B2}}{2} + V_{G11}\right) = 5\left(\frac{20.8^k}{2} + 32.7^k\right) = 216^k$

$M_{rx} = 5(M_{B2} + M_{B11}) = 211^{k'} + 255^{k'} = 466^{k'}$

TRY W12X96

$\frac{Kl}{r_y} = \frac{(1.0)(14')(12)}{3.09''} = 54.4''$ ← CONTROLS

$\frac{Kl}{r_x} = \frac{(1.0)(14')(12)}{5.44''} = 30.9''$

$Kl = (1.0)(14') = 14'$

$p \times 10^3 = 0.978$

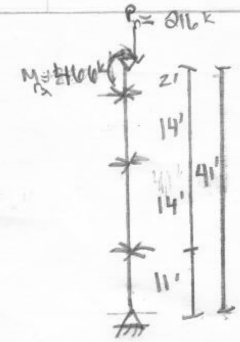
$b_x \times 10^3 = 1.67$

$\frac{P_r}{P_c} = \frac{pP_r}{10^3} = \frac{(0.978)(216^k)}{10^3} = 0.211 \geq 2.0$ ✓ OK

$\frac{P_r}{P_c} + \frac{8}{9}\left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right) \leq 1.0$

$\frac{pP_r}{10^3} + \frac{8}{9}\left(\frac{b_x M_{rx}}{10^3} + 0\right) \leq 1.0$

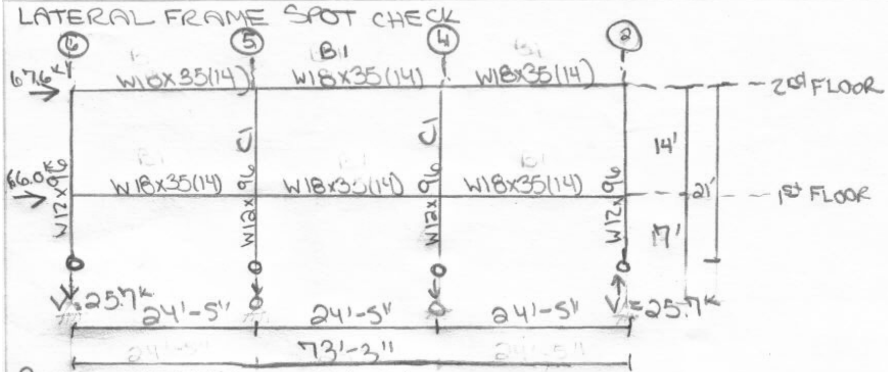
$\frac{(0.978)(216^k)}{10^3} + \frac{8}{9}\left(\frac{(1.67)(466^{k'})}{10^3} + 0\right) = 0.90 < 1.0$ ✓ OK



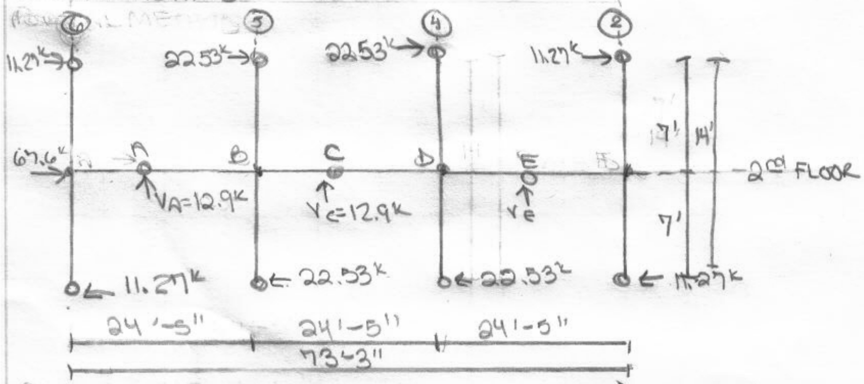
NOTE: DESIGN CONSTRAINTS OF NOT USING LIVE LOAD REDUCTIONS OR MEMBERS SMALLER THAN W12X14 WAS UTILIZED.

Lateral Frame Spot Check

LATERAL FRAME SPOT CHECK



PORTAL METHOD



$\circlearrowleft \Sigma M_A = -(11.27^k)(7') - (11.27^k)(7') + V(24.42'/2) = 0$

$V_A = 12.9^k$

$\circlearrowleft \Sigma M_B = -(22.53^k)(7') - (22.53^k)(7') + (12.9^k)(24.42'/2) + V_C(24.42'/2) = 0$

$V_C = 12.9^k$

$\circlearrowleft \Sigma M_D = -(22.53^k)(7') - (22.53^k)(7') + (12.9^k)(24.42'/2) + V_E(24.42'/2) = 0$

$V_E = 12.9^k$

$\circlearrowleft \Sigma M_{OT} = -(67.6^k)(21') - (66.0^k)(7') = -1882^{k'}$

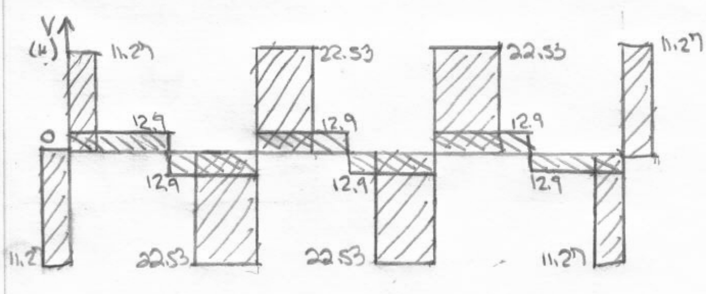
$\circlearrowleft \Sigma M_{RES} = -V(73.25')$

$\Sigma M_{OT} = \Sigma M_{RES}$

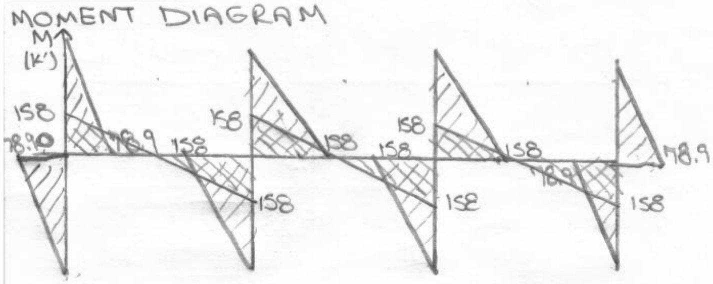
$-1882^{k'} = -V(73.25')$

$V = 25.7^k$

SHEAR DIAGRAM



MOMENT DIAGRAM



B1. BEAM DESIGN (SEE GRAVITY FLOOR BAY SPOT CHECK FOR MORE INFO)

ORIGINAL: W18 x 35 (14)

SPAN: 27'-8"

LOADS:

$V = 20.8^k + 1.6(12.9^k) = 41.4^k$

$M = 211^{k'} + 1.6(158^{k'}) = 464^{k'}$

$b_{eff} = 6.92$

ASSUMING $a = 1"$

$Y2 = 5" - \frac{1"}{2} = 4.5"$

W16x45 P.N.A. BFL

$\phi M_n = 482^{k'} > 464^{k'}$ ✓ OK

$\Sigma Q_n = 266^k$

$a = \frac{\Sigma Q_n}{0.85 f'_c b_{eff}} = \frac{266^k}{0.85(4000\,psi)(6.92')(12)} = 0.942" < 1"$ ✓ OK

$Y2 = 5" - \frac{0.942"}{2} = 4.53"$

$\phi M_n = 483^{k'} > 464^{k'}$ ✓ OK

$I_{LB} = 1192\ IN^4 > 87.4\ IN^4$ ✓ OK

$I = 586\ IN^4 > 63.9\ IN^4$ ✓ OK

$\#\ STUDS = \frac{\Sigma Q_n}{Q_n} = \frac{266^k}{21.5^k} = 12.4 \rightarrow 13 \rightarrow 26$

C1. COLUMN CHECK (SEE GRAVITY FLOOR BAY SPOT CHECK FOR MORE INFO)

ORIGINAL: W12x96, W12x120, W12x136

HEIGHT: 41'-0"

LOADS:

$P_r = 216^k + 5(12.9^k) = 281^k$

$M_{rx} = 466^{k'} + 158^{k'} = 624^{k'}$

TRY W12x96

$\frac{pP_r}{103} + \frac{8}{9}\left(\frac{b_x M_{rx}}{103} + 0\right) \leq 1.0$

$\frac{(0.978)(281^k)}{103} + \frac{8}{9}\left(\frac{(1.67)(624^{k'})}{103} + 0\right) = 1.20 > 1.0$ X NO