Faculty Advisor – Professor Kevin Parfitt
Renaissance Schaumburg Hotel and Convention Center
Schaumburg, Illinois

Structural Concepts/Structural Existing Conditions Report
Due: October 5th, 2005

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Executive Summary

This report details structural considerations for the design of the Renaissance Schaumburg Hotel and Convention Center located in Schaumburg, Illinois. Specifically, this report will focus on the main hotel's structural composition and performance.

The 17-stories of the Renaissance Schaumburg Hotel create quite an impressive display of engineering technology in the open landscape surrounding the city's newest and largest hotel. The 465,885 square feet of the hotel are supported by columns of reinforced concrete and steel. Most of the steel on this project is reserved for the convention center but in the first 3 floors of the hotel steel columns are used to transfer gravity loads from concrete supports above. The hotel has many unique features including a swimming pool located on the ground floor, a health center, a grand ballroom, secondary ballrooms, a restaurant, a large open atrium area, and 500 guest rooms. The scheduled completion date for the building is slated for July 2005, when the first visitors to the area can receive four-star service in one of the finest hotels that the Chicago area has to offer.

This report is limited to analysis based on the most current design documents made available for the Renaissance Schaumburg Hotel and Convention Center. Its function is to provide a detailed description of the structural systems in use, including those found in the foundation, slab construction, framing, and lateral force resisting systems. Simplified sketches have been included to further explain shear wall systems, framing plans, and lateral load distribution. Please see the appendix for other figures. Gravity, wind, and seismic loading are explored in this report, a summary of which can also be found in the attached appendix. This report will further detail the current existing conditions and structural concepts used in the design of the Renaissance Schaumburg Hotel and Convention Center.

Building Description

Introduction

The Renaissance Schaumburg Hotel and Convention Center (RSHCC) is a 17 story, 4-star hotel with an accompanying convention center attached to its east wing. This report will explore the structural concepts used in the design of the hotel structure which is primarily constructed out of reinforced concrete. The hotel sits in the heart of Schaumburg Illinois, a growing sub-urban area which is only a short drive north from downtown Chicago. The new hotel and convention center in the city of Schaumburg has been a project that was first discussed in the mid-1980s and is a welcomed edition to the area as it will serve as an economic and cultural center for many years to come.

Structural System

The RSHCC is an excellent example of the use of reinforced concrete for a high-rise hotel structure. The structural system, in addition to supporting the 17 stories with over 500 rooms, the superstructure also supports multiple ballrooms, a restaurant, conference rooms, a cooling tower, a club lounge. The 465,885 square feet of the hotel are supported by columns of reinforced concrete and steel. Most of the structural steel on this project is reserved for the convention center but in the first 3 floors of the hotel steel columns are used to transfer gravity loads from the concrete supports above. For the primary support from the foundation to the third through sixth floors large 42" diameter columns are used, which are then blocked-out to rectangular columns with sizes ranging from 12"x24" to 18"x28" to support the upper levels of the hotel structure. Shear walls are used in three main locations throughout a typical floor plan and, as a diaphragm element, post-tensioned concrete slab can be found on almost ever floor of the structure which helps to reduce the amount of concrete typically necessary to carry loads.

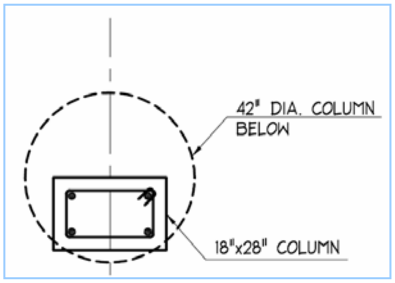


Figure 1- Block Out

Foundation

The RSHCC employs the use of drive steel piles for foundation support, grade beams which vary from 24" to 60" deep are then supported by the piles. All driven steel piles and concrete pile caps must develop a 100 ton capacity with a minimum safety factor of 2. These structural steel piles transfer loads from the foundation into the earth. All perimeter wall and column foundations must all bear a minimum of 3'6" below the finished grade. The grade beams then span over the pile caps and support the slab on grade, which is typically 6" throughout the ground floor plan of the building.

Framing

The frame skeleton of the RSHCC is rather unique. The architect called for large atrium spaces and designed the floor systems above the main lobby area to appear as though they almost float. To accomplish this, a typical 42" diameter concrete column spans the first 3 to 6 levels of the hotel, which supports the slab. Typical slab thickness is 7.5" and on most floors uses a post-tension slab system which helps to reduce the amount of concrete needed. Steel is also utilized on lower floors (usually as a gravity load transfer from upper levels of concrete columns) which typical include beam and girder sizes of W16x26 and W24x55 respectively. The column grid for the main hotel structure is laid out in the east-west direction to 27' on center for 5 spans. However, there is a rather non-regular spacing of north-south column lines which also have 5 spans totaling 117 feet. Each of the two stair cases on the front exterior of the building are constructed out of steel and use moment resisting connections.

Slab Systems

Multiple types of concrete slab systems are used in this project including one-way, two-way (with drop-panels), and post-tensioned slabs. Stud-rails are also used near column supports in order to minimize punching failure, eliminate excess drop-panels, and allow for the possibility of smaller column sizes. These stud-rails are typically used on column lines K, L and M, or the south-east side of the building, this is most likely due to the column line's adjacency to a change in slab elevations*.

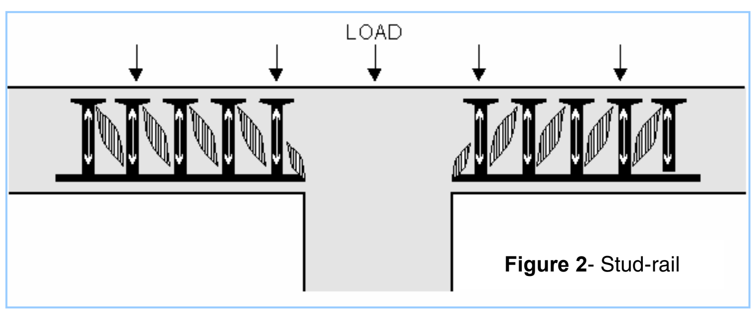


Figure 2- Stud-rail

The post-tensioned concrete slab is the most prevalent type of floor system used through the 17 stories of the building. Typical effective stresses in the post-tensioned tendons are typically around 20 kips per foot. This type of slab is useful due to its efficient use of concrete. In some systems, it results in a 30% savings of concrete when compared to typically reinforced concrete slabs.

* Figure 2 – Stud-rail image courtesy www.studrail.com

Shear Wall

As Stated above, shear walls are used to help distribute story shear forces due to wind and low possible seismic loads. The shear wall schedule calls for 8,000 psi concrete to be used on the bottom 6 floors while 6,000 psi concrete is reserved for the upper levels at which the thickness also decreases from 18" to 12". Typical ASTM A615 Grade 60 steel is also called for in reinforcement of the shear walls with sizing and placement varying from #8's at 8" to a minimum of #4's at 12". The shear wall layout is basically three C-shaped concrete walls which are placed through the floor plan as shown below.

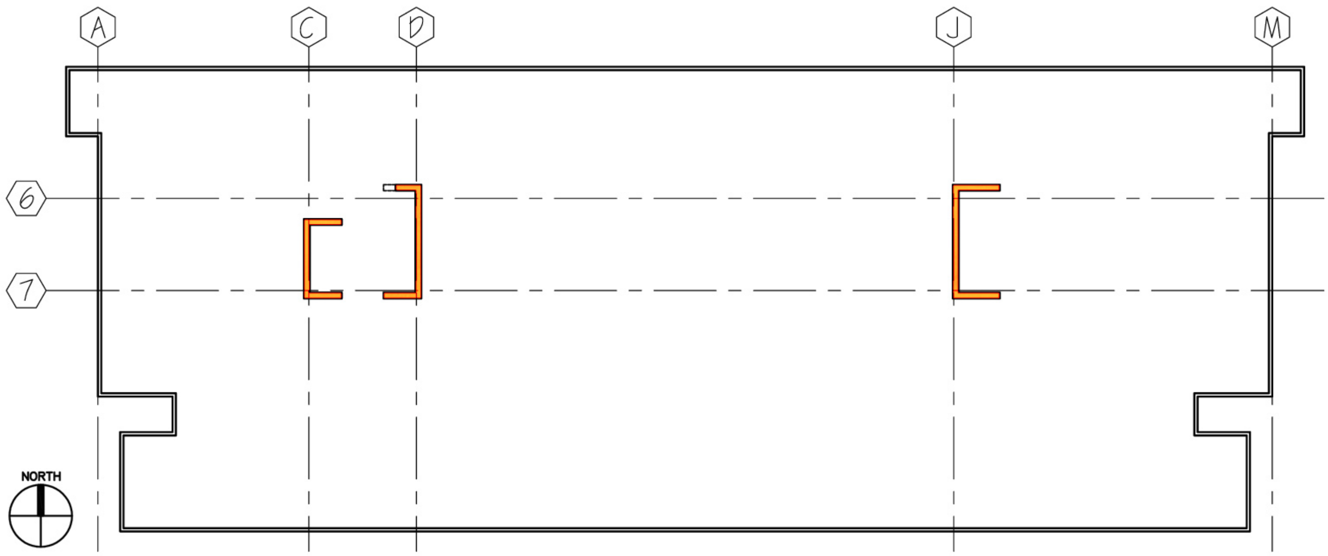


Figure 3 - Shear Walls

Materials

The RSHCC utilizes structural concrete throughout the hotel structure. The structural notes from the lead structural engineering firm state that all the concrete is normal weight and varies from the typical 4,000psi to as much as 8,000psi in columns, shear walls, and link beams.

Structural steel grades also vary through the structure, wide flanges and built-up plate girders use ASTM A992 and A572 respectively, which both supply a yielding stress of 50 ksi, while column base plates, angles, channels, and other miscellaneous plates use A36.

Composite steel decks are to have the minimum yield strength of 40 ksi and be no less than 20 gauge.

Design Codes and Standards

Building Codes

The design of the Renaissance Schaumburg Hotel and Convention Center is based off of the IBC 2003 code. For this project, since the IBC 2003 was adopted (with some minor changes and deletions) provisions to ASCE 7 have also been incorporated. For the load development procedures demonstrated throughout the rest of this report will be based on those outlined in ASCE 7-02.

Specifications and Standards

ACI 315, the "Manual of Standard Practice for Detailing Reinforced Concrete Structures" and ***ACI 318***, the latest edition of the "Building Code Requirements for Structural Concrete" are to be considered for all procedures involving the placement techniques of concrete and concrete reinforcement.

The latest editions of the ***AISC LRFD Specification and Codes*** are to be conformed to for all detailing, fabrication, and erection of structural steel (3rd Edition Manual of Steel Construction).

All welding work is to also conform to ***AWS D1.1*** "Structural Welding Code".

Loading Calculations

The following snow, gravity, and lateral load developments are based on the provisions of ASCE 7-02. Appropriate load factors and applicable live load reductions were as used as per ASCE 7-02.

Snow Load

Flat roof snow loads (for area of 311 feet by 117 feet):

$$p_f = 0.7 C_e C_t I p_g$$

$$C_e = 0.9 \qquad C_t = 1.2 \qquad I = 1.0 \qquad p_g = 25$$

$$p_f = 0.7(0.9)(1.2)(1.0)(25) = 18.9 < I(20) = 20\,psf$$

Gravity Loads

Live Loads (psf):

o Typical Floors (Hotels refer to residential)	(40psf + 20psf for partitions) 60 psf
o Public rooms and Corridors	100 psf
o Roof	30 psf
o Mechanical Areas	150 psf
o Stairs	100 psf
o Elevator Lobbies	100 psf
o Ballrooms	100 psf
o Bridge	100 psf

Dead Loads (psf):

o Mechanical/Ceiling	7 psf
o Finishes	60 psf
o Carpet/Miscellaneous	5 psf
	Total = 72 psf

Lateral Loads

For the lateral load computation due to wind and seismic forces Microsoft Excel was used to develop an interactive spreadsheet to calculate story shear forces. These files are available for download at the links posted in each appendix.

A summary of story forces and the results of both wind and seismic can be found below, with a detailed look at the procedure in the Appendix A for wind and Appendix B for seismic.

Wind

o ASCE 7-02, Chapter 6 – See page 10 for loading diagram

Velocity Pressure Envelope

	Windward		Leeward		Max
Z(ft)	N-S	E-W	N-S	E-W	(psf)
0-15	15.08	15.08	-17.90	-12.00	32.97
20	15.96	15.96	-17.90	-12.00	33.86
25	16.67	16.67	-17.90	-12.00	34.57
30	17.38	17.38	-17.90	-12.00	35.28
40	18.45	18.45	-17.90	-12.00	36.34
50	19.33	19.33	-17.90	-12.00	37.23
60	20.04	20.04	-17.90	-12.00	37.94
70	20.75	20.75	-17.90	-12.00	38.65
80	21.46	21.46	-17.90	-12.00	39.36
90	21.99	21.99	-17.90	-12.00	39.89
100	22.35	22.35	-17.90	-12.00	40.24
120	23.24	23.24	-17.90	-12.00	41.13
140	24.12	24.12	-17.90	-12.00	42.02
160	24.65	24.65	-17.90	-12.00	42.55
180	25.36	25.36	-17.90	-12.00	43.26
200	25.90	25.90	-17.90	-12.00	43.79
184.67	25.49	25.49	-17.90	-12.00	43.38

Distribution & Summary

Level	h/floor (ft)	Z (ft)	V (k)	M (ft-k)
18	8	184.67	54.48	10,060.20
17	8	176.67	107.74	19,034.35
16	11.67	168.67	132.30	22,314.06
15	9.67	157	128.33	20,147.19
14	9.67	147.33	127.92	18,847.11
12	9.67	137.67	126.74	17,447.26
11	9.67	128	126.32	16,169.20
10	9.67	118.33	124.53	14,735.94
9	9.67	108.67	123.66	13,437.25
8	9.67	99	122.05	12,082.63
7	10.21	89.33	123.74	11,054.34
6	9.79	79.13	123.10	9,740.55
5	9.67	69.33	117.87	8,172.65
4	9.67	59.67	115.05	6,864.67
3	9.67	50	112.99	5,649.55
2	24.33	40.33	196.82	7,938.47
1	16	16	212.66	3,402.49
0	0	0	82.04	
Σ	Base		2,258.33	217,097.92

Seismic
o ASCE 7-02, Chapter 9

Seismic Summary						
Level	w_x	h_x	$w_x h_x^k$	C_{vx}	F_x (kips)	M_x (ft-kips)
18	1,335.61	184.67	6,435,194	0.06814	89.6	16,548
17	2,649.81	176.67	11,880,671	0.12581	165.4	29,227
16	2,649.81	168.67	11,018,876	0.11668	153.4	25,879
15	2,696.89	157.00	9,981,600	0.10570	139.0	21,821
14	2,671.21	147.33	8,916,557	0.09442	124.2	18,293
12	2,671.21	137.67	7,985,550	0.08456	111.2	15,308
11	2,671.21	128.00	7,094,540	0.07512	98.8	12,645
10	2,671.21	118.33	6,244,635	0.06613	87.0	10,290
9	2,671.21	108.67	5,437,066	0.05757	75.7	8,227
8	2,671.21	99.00	4,673,209	0.04949	65.1	6,442
7	2,671.21	89.33	3,954,623	0.04188	55.1	4,919
6	2,678.17	79.13	3,255,346	0.03447	45.3	3,587
5	2,672.82	69.33	2,621,198	0.02776	36.5	2,531
4	2,671.21	59.67	2,052,448	0.02173	28.6	1,705
3	2,671.21	50.00	1,540,048	0.01631	21.4	1,072
2	2,671.21	40.33	1,086,206	0.01150	15.1	610
1	2,859.53	16.00	258,815	0.00274	3.6	58
Σ	44,254.73		94,436,581	1	1,315.0	179,162

Summary

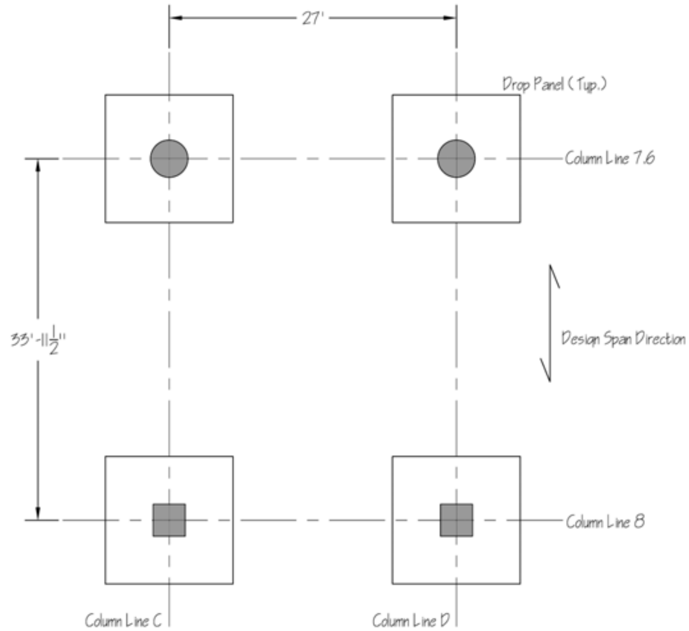
Load Analysis Summary			
	Wind	Seismic	Total
Shear @ 18	54.48	84.97	139.45
Shear @ 17	107.74	170.47	278.21
Shear @ 16	132.30	158.11	290.40
Shear @ 15	128.33	143.16	271.48
Shear @ 14	127.92	127.91	255.84
Shear @ 12	126.74	114.56	241.29
Shear @ 11	126.32	101.78	228.10
Shear @ 10	124.53	89.58	214.11
Shear @ 9	123.66	78.00	201.65
Shear @ 8	122.05	67.04	189.09
Shear @ 7	123.74	56.73	180.47
Shear @ 6	123.10	46.70	169.80
Shear @ 5	117.87	37.60	155.48
Shear @ 4	115.05	29.44	144.49
Shear @ 3	112.99	22.09	135.08
Shear @ 2	196.82	15.58	212.40
Shear @ 1	212.66	3.71	216.36
Shear @ Ground	82.04		82.04
Base Shear	2,258.33	1,347.43	3,605.76
Overturning Moment	217,097.92	183,228.15	400,326.07
*Shear (kips) **Moment (ft-kips)			

Spot Checks

Gravity

Slab

Scope: Check 2-way Drop Panel Slab Design on floor 2 between columns C-7.6 and D-8. (see image)



Summary:

ACI 318-05 design:

Location	Strip	Use	Totals
Interiror Support (-)	C.S	#6's@8"	21-#6's
	M.S	#4's@12"	14-#4's
Midspan (+)	C.S	#6's@8"	21-#6's
	M.S	#4's@12"	14-#4's

10" slab
12" drop-panel

As-built Design:

Location	Strip	Totals	ϕ M_n
Interiror Support (-)	C.S	17-#7's	732.08
	M.S	15-#5's	339.35
Midspan (+)	C.S	10#5's	227.28
	M.S	16-#5's	153.37

10" slab
20" drop-panel

The after comparing the moment capacity of each system, you can see that by following the ACI code similar values were obtained to those actually used in the building. The reason for the as-built design to have a larger capacity may be due to un-conservative load estimates (mostly with sustained dead load and live load conditions) when following the ACI code.

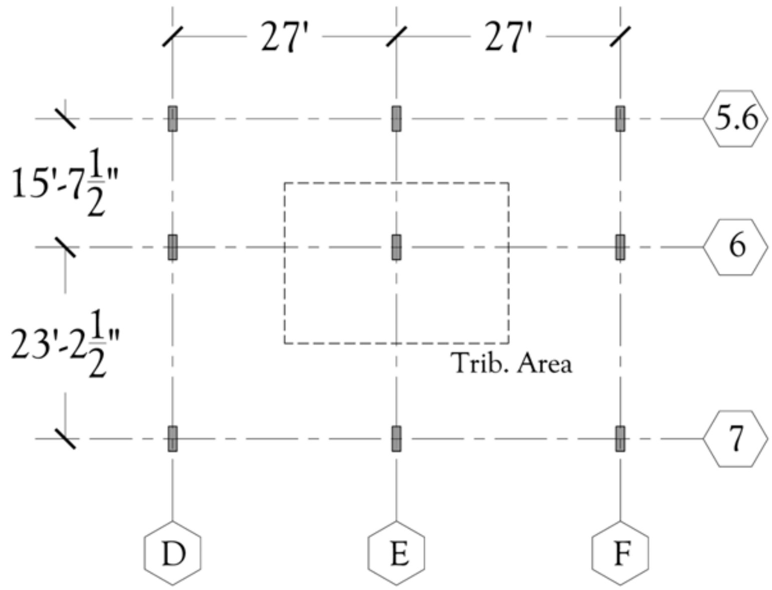
See Appendix C for detailed gravity spot check calculations which include ACI design ϕM_n values

Column

Design a column on column line E6, floor 8.

For simplification, the column design guide used consider f'_c=4ksi and not the 6ksi used in the actual building. When calculating A_{smin} 6ksi was considered for the computation of K_n and R_n, which induces error into the final calculation. Both designs looked at a 1' by 3' column at column line E6. The design presented below in Appendix C.2 compare in the follow chart

Design	As
Design Check	12.96 in^2
As-built	8 in^2



Additional Considerations

Some other additional considerations of the structural design should also note the interesting architectural feature on the north face of the building. A rather large concrete disc is suspended as a covered driveway for valet service. This disc also has a running water feature that pours over the disc and lands in the reflecting pond in the front of the building. This feature, due to its size and mass may warrant special consideration due to wind loadings, since the building site is in the C-class of wind exposure unusually high wind loads may need to be designed for. Also the possibility of freezing and increased loads to snow and drift should be considered for winter months.

Conclusion

A lot of different and interesting structural systems were used in the design of the Renaissance Schaumburg Hotel and Convention Center. The main hotel superstructure is a cast-in-place concrete with a wide variety of slab types. The hotel appears that it will be a good structure to further explore, and upon completing a STAAD/Ram model a better understanding lateral forces and system performance will be attained. The next step, upon a more detailed analysis, is to closely examine the performance of the shear walls in there attempt to resist lateral forces. At this time this concludes the preliminary observation of the structural concepts and existing conditions employed in the design of the Renaissance Schaumburg Hotel and Convention Center.[†]

[†] End of Report – Continue on with Appendices

Appendix A

Wind Load Analysis

Building Properties	
B (ft)	117
L (ft)	311
h (ft)	184.67
K_{zt}	1
K_d	0.85
V (mph)	90
Importance	III
I_w	1.15
Exposure	C
α	9.5
z_g	900
z_{min}	15
c	0.2
$\in$	0.2
l	500
~~b~~	0.154
~~α~~	0.65
a	0.105
b	1

Period Parameters	
Struct. Type	Concrete
C_t	0.016
x	0.9
(check eq) T	1.753354
Natural f	0.570335
Rigidity	Flex

Rigid	
g_Q=g_v	3.4
ž	110.8
$I_ž$	0.16344
$L_ž$	637.0524
Q	0.84716
G	0.856322

Flexible	
g_R	4.05
R_n	0.037
N_1	8.14
η_h	10.86
η_B	0.059
η_L	61.21
R_h	0.088
R_B	0.962
R_L	0.016
$V_ž$	44.62
β	0.05
R	0.18
G_f	0.8688

Windward	
Cp	0.8

Leeward		
	Ratio	C_p
N-S	0.376	-0.50
E-W	2.658	-0.27

Pressure Coefficients		
Internal		
Enc. Type	Enclosed	
Internal (GC_{pi})	0.18	+/-

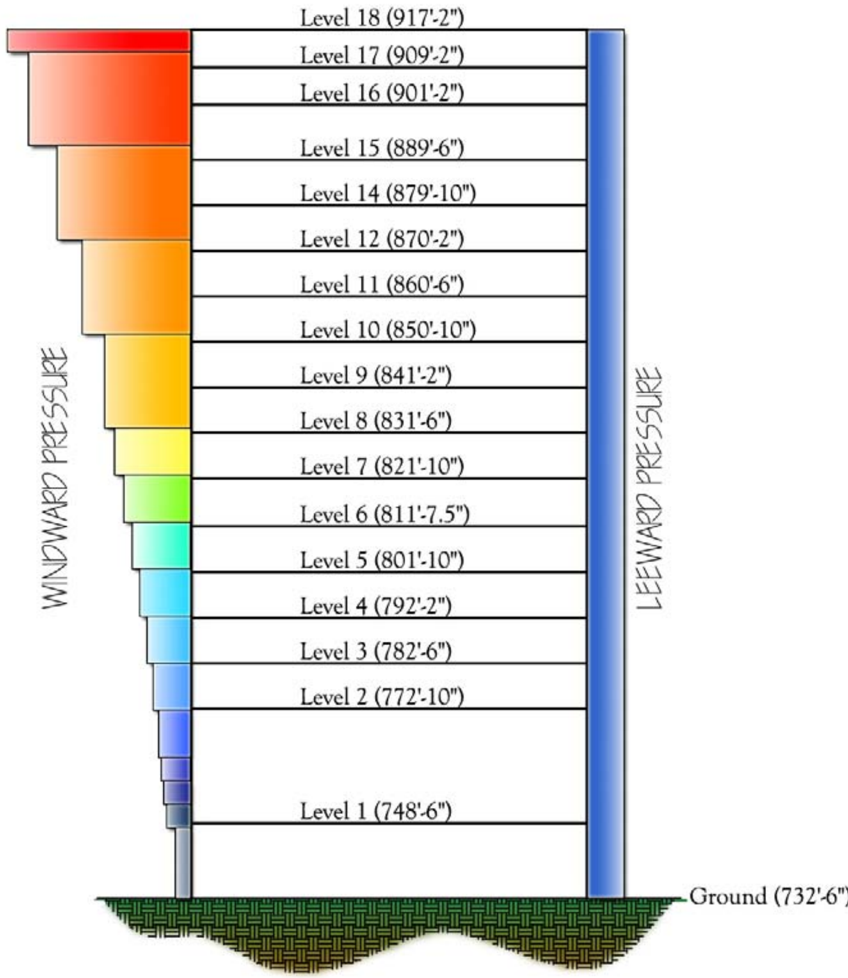
Pressures			
Windward	N-S	P_z	0.875
	E-W	P_z	0.875
Leeward	N-S	P_h	-0.614
	E-W	P_h	-0.412

This file is available at the following hyperlink:

http://www.arche.psu.edu/thesis/eportfolio/current/portfolios/ejy112/tech-assign.htm

Appendix A


311'
117'
N


Level 18 (917'-2")
Level 17 (909'-2")
Level 16 (901'-2")
Level 15 (889'-6")
Level 14 (879'-10")
Level 12 (870'-2")
Level 11 (860'-6")
Level 10 (850'-10")
Level 9 (841'-2")
Level 8 (831'-6")
Level 7 (821'-10")
Level 6 (811'-7.5")
Level 5 (801'-10")
Level 4 (792'-2")
Level 3 (782'-6")
Level 2 (772'-10")
Level 1 (748'-6")
Ground (732'-6")
WINDWARD PRESSURE
LEEWARD PRESSURE

Appendix B

Seismic Analysis

Building Properties	
B (ft)	117
L (ft)	311
h (ft)	184.67
# of Stories	17.00
ave. h/floor (ft)	10.86
Seismic Use group	I
Imp. (e)	1
Site Classification	D
S_s (%g)	0.2
S_1 (%g)	0.065
R	3
F_a	1.6
F_v	2.4
S_{DS}	0.32
S_{D1}	0.156

Response	
T	1.75
C_s	0.03

Load Summary (psf)	
Roof Dead	30
Snow	20
Floor Dead	72
Ex. Wall Dead	15
avg. w_{roof} (lbs)	1,306.9
avg. w_{floors} (lbs)	2,759.3
W_{total} (lbs)	45,346.3
V (lbs)	1,347.4

Distribution	
k	1.625

Seismic Summary

Level	w_x	h_x	$w_x h_x^k$	C_{vx}	F_x (kips)	M_x (ft-kips)
18	1,262.84	184.67	6,084,557	0.06306	85.0	15,692
17	2,722.58	176.67	12,206,960	0.12652	170.5	30,117
16	2,722.58	168.67	11,321,497	0.11734	158.1	26,667
15	2,769.66	157.00	10,250,948	0.10624	143.2	22,476
14	2,743.98	147.33	9,159,478	0.09493	127.9	18,846
12	2,743.98	137.67	8,203,107	0.08502	114.6	15,771
11	2,743.98	128.00	7,287,822	0.07553	101.8	13,027
10	2,743.98	118.33	6,414,763	0.06648	89.6	10,601
9	2,743.98	108.67	5,585,192	0.05789	78.0	8,476
8	2,743.98	99.00	4,800,525	0.04975	67.0	6,637
7	2,743.98	89.33	4,062,362	0.04210	56.7	5,068
6	2,750.94	79.13	3,343,804	0.03466	46.7	3,695
5	2,745.59	69.33	2,692,566	0.02791	37.6	2,607
4	2,743.98	59.67	2,108,365	0.02185	29.4	1,757
3	2,743.98	50.00	1,582,005	0.01640	22.1	1,105
2	2,743.98	40.33	1,115,798	0.01156	15.6	628
1	2,932.30	16.00	265,402	0.00275	3.7	59
Σ	45,346.34		96,485,151	1	1,347.4	183,228

This file is available at the following hyperlink:

http://www.arche.psu.edu/thesis/eportfolio/current/portfolios/ejy112/tech-assign.htm

Appendix C.1 – Gravity Spot Check – Slab Design

Column Strip:

$$C.S. = \frac{1}{4}27' = 6.75'$$

$$M.S. = 27 - 2\left(\frac{1}{4}27'\right) = 13.5'$$

Drop Panel:

$\frac{1}{6}(33'\text{-}11.5'') = 5.66'$ Say 6' per side =12'x12'

$\frac{1}{4}(t_s) = \frac{1}{4}(10'') = 2.5''$ Min panel thickness

Equivalent Square Section of a Circular Column: $0.89h = 0.89(42'') = 37.38''$

Minimum Thickness: $mnt = \frac{l_n}{36} = \frac{\left((33'\text{-}11.5'') - \left(\frac{37.38''}{2} + \frac{36''}{2}\right)\right)12''/1'}{36} = 10'' \therefore$ use 10" for thickness

Loading: DL=SW+DL$= 150\,pcf\left(\frac{10''}{12''/1'}\right) + 72\,\text{lb/ft}^2 = 197\,\text{lb/ft}^2$

LL=100 lb/ft^2

$w_u = 1.2(197) + 1.6(100) = 396.4\,\text{lb/ft}^2$ $\quad M_o = \frac{w_u l_2 l_n^2}{8} = \frac{0.3964(27)(30.9)^2}{8} = 1277.4\, ft - k/ft$

Location	Strip	Total M_u	Total Width	Moment/ft
Interiror Support (-) 65%	C.S (75%)	622.73	13.5	46.13
	M.S (25%)	207.58	13.5	15.38
Midspan (+) 35%	C.S (60%)	268.25	13.5	19.87
	M.S (40%)	178.84	13.5	13.25

Solve for each Moment/ft

Due to large M_o, A thicker drop panel will be assumed since larger support moments exist.

$A_s = \frac{M_n}{f_y(0.9d)}$ $\quad d = 10'' - \frac{3}{4}'' - \frac{3}{4}'' = 8.5''$ @t_s=10" $\quad a = \frac{A_s f_y}{0.85(f'_c)(b)}$ $\quad M_n = A_s f_y\left(d - \frac{a}{2}\right)$

Minimum Steel Reinforcing

$0.0018A_g = 0.0018(10)(12''/1') = 0.216 in^2/ft$ use #4@10"(A_s=0.24in^2)

$d = 10 - \frac{3}{4} - \frac{1}{2} - \frac{1}{4} = 8.5''$ $\quad a = \frac{A_s f_y}{0.85 f'_c b}$ $\quad a = \frac{0.24 \cdot 60}{0.85 \cdot 4 \cdot 12''} = 0.353''$

$M_n = A_s f_y\left(d - \frac{a}{2}\right)$ $\quad \phi M_n = (0.9)0.24(60)\left(8.5 - \frac{0.353}{2}\right) = 8.99\, ft - k$ (Will not suffice for any case below)

Use these equations to solve for table below

Location	Strip	Total M_u	Total Width	Moment/ft	A_s req	try	A_s/ft	a	ϕM_n
Interiror Support (-) 65%	C.S (75%)	622.73	13.5	46.13	0.50	#6's@8"	0.66	0.971	713.32
	M.S (25%)	207.58	13.5	15.38	0.17	#4's@12"	0.29	0.426	317.69
Midspan (+) 35%	C.S (60%)	268.25	13.5	19.87	0.52	#6's@8"	0.66	0.971	285.64
	M.S (40%)	178.84	13.5	13.25	0.35	#4's@8"	0.29	0.426	129.77

Note: d for ϕM_n at support is increased by 12" since it is measured from bottom of concrete to top steel section.

$d = 22'' - \frac{3}{4}'' - \frac{3}{4}'' = 20.5''$

Appendix C.2 – Gravity Spot Check – Column Design

Influence Area:

$A_t == ft^2$ (8 levels above floor 8) $A_t(8) = A_T = ft^2$

Loads:

LL: 100psf LL$_{reduction}$: $L_o\left(0.25+\frac{15}{\sqrt{A_I}}\right) \geq 0.4L_o$

DL: 72psf $A_t = 27\left(\frac{15'7.5''}{2}+\frac{23'2.5''}{2}\right) = 524.25ft^2$ (8 levels above floor 8)

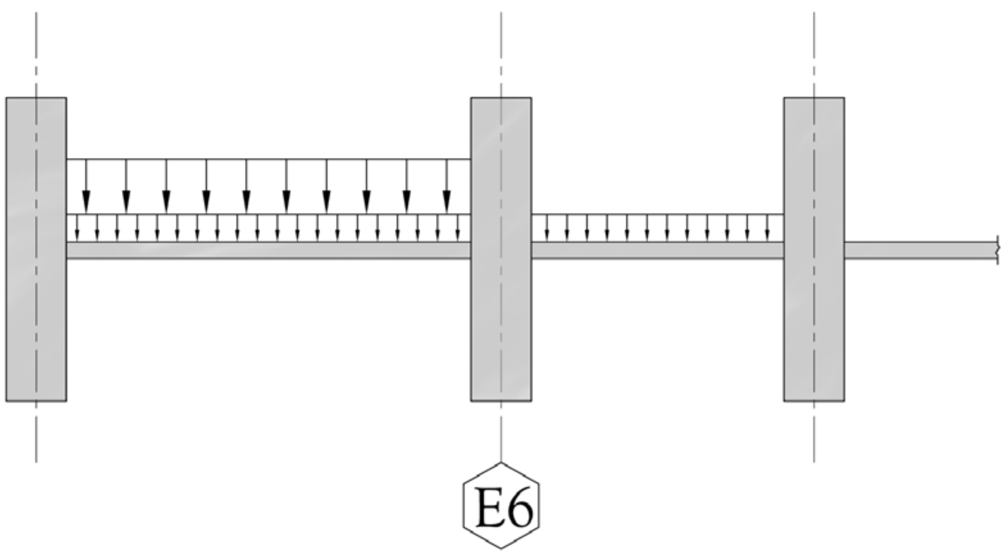
$A_t(8) = A_T = 4{,}194ft^2$ $A_I(4) = 16{,}776ft^2$

Reduction: $0.25+\frac{15}{\sqrt{16{,}776}} = 0.3658 \ngeq 0.4$ $\therefore$ use 0.4

Total Load: $1.2(DL)+1.6(LL) = 1.2(72)+1.6(40) = 150.4psf$

Solve for Axial Force:

$P = A_I w_u = 16{,}776ft^2(150.4psf) = 2{,}523.1kips$



Solve for Moment Reaction (see figure above):

$FEM_{left} = \frac{wl^2}{12} = \frac{4.061klf(23'2.5'')^2}{12} = 182.28ft-k$ $FEM_{lright} = \frac{wl^2}{12} = \frac{2.3328klf(15'7.5'')^2}{12} = 47.46ft-k$

$$M = \frac{182.28-47.46}{2} = 67.41ft-kip$$

Refer to Column Strength Interaction Diagrams (pg.758 of Design of Concrete Structures, Nilson)

Assume a 1' by 3' column is to be used.

$K_n = \frac{P_n}{f'_c A_g} = \frac{2{,}523.1}{6(432)} = 0.973$ $R_n = \frac{P_n e}{f'_c A_g h} = \frac{67.41}{6(432)12} = .026$ => $\rho_g = 0.03$

Required Steel Area: $A_c\rho_g = 0.03(432) = 12.96in^2$ compare to the 8in^2 used in the as-built design