University of North Carolina's

Imaging Research Building

Technical Report 1

Daniel R. Hesington, LEED AP

Structural Option

Consultant- Dr. Thomas Boothby

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Table of Contents

Executive Summary

The structural concepts and existing conditions report describes the structural system of the University of North Carolina's Imaging Research Building. This 10-story laboratory, office, and imaging research space is located on the UNC's Chapel Hill campus and makes use of a combinations of spread and mat foundations, concrete one-way slabs, ordinary reinforced shear walls, and a variety of columns and beams.

Gravity and lateral loads were calculated using ASCE 7-05 and compared to Mulkey Engineers and Consultants' design loads. The controlling wind lateral load was found to be in the North/South direction with a base shear of V=1536k compared to Mulkey's base shear 1050k. The seismic analysis also had a notable discrepancy between Mulkey's design values with a base shear of 1822 kips calculated for this report compared to a base shear of 1125 kips. Possible reasons for this discrepancy are noted in the body of the report.

Spot checks were also conducted on an interior column and an interior beam. These checks supported that the determination and accumulation of the gravity loads on this structure were comparable to those calculated by Mulkey. Both components of the structural system were adequately designed based on gravity load calculations alone. Although, when the moment was accounted for, the typical interior column falls outside the interaction diagram and subsequently fails. This is only briefly covered and not examined in detail as it was not required for this technical report.

Introduction

The structural concepts and existing conditions report contains a description of the structural system of the University of North Carolina's Imaging Research Building. The architecture of the project is briefly inspected to relate its impact on the structural design of the building. Also, an overview of the foundations, framing, slabs, and lateral force resisting systems is given to show how the components of the structural system work together. Not only that, but loads are calculated per ASCE 7-05 and compared to the loads found by the structural engineer on the project, Mulkey Engineers and Consultants. A combination of the project drawings, specification, and geotechnical report were used to obtain the information needed to examine the existing conditions. Finally, spot checks of typical floor framing are included to verify if the required loading was calculated and considered correctly.

The Imaging Research Building, also known as IRB, is located on the University of North Carolina's Chapel Hill campus on Mason Farm road. It has an "L" shaped floor plan containing a re-entrant corner, with the long face dimensions of 282'-4" by 247'-3". It has an overall height of 180'-0" from Basement 2 (second floor subgrade) to the roof, with a 20' setback at the mechanical mezzanine level. The building's usage will be a combination of research space, laboratories, and office space for UNC.

Architectural Design Concepts

The Imaging Research Building at UNC Chapel Hill was designed by the architecture firm Perkins + Will. Its primary usage is the driving force behind many of the structural decisions for the project. Once it is open, it will contain the most advanced imaging equipment in any one spot in the world. First, the two subgrade floors house several heavy pieces of imaging research equipment that have large Gaussian fields. Because of this, foundations, walls, and slabs were made thicker than usual, which will result in the use of mass concrete pouring techniques to be required when constructed. For example, the foundation where a 1.5GHZ NMR machine will sit required a 6' thick mat footing.

Above grade you will find typical bays sizes of 21'-4" by 21'-4", and 21'-4" by 31'-4" driven by the laboratory space requirements on every floor. A bridge also connects the new imaging research facility to existing Lineberger Cancer Center on the second floor. At the eighth floor, a large area houses all of the mechanical equipment with a partial mezzanine at the floor above, which services all of the imaging and laboratory equipment below. These architectural and usage restraints have a generous effect on the structural system as noted below, and hopefully in future technical reports.

Structural System

Foundation

The geotechnical engineering study was performed by Tai and Associates on November 12, 2008. The study indicates that the subsurface materials on the site consist of pavement and topsoil, fill, residual soil, weathered rock, and rock and boulders. Based on this composition, Tai and Associates were confident in giving Mulkey a net allowable bearing pressure of 6000 pounds per square foot to use in their foundation calculations.

Because of this allowable bearing pressure, Mulkey had to be creative with their foundation design. The result is a mixture of spread footings under the columns, and a combination of spread and mat footings under the large imaging research equipment and shear walls. The walls below grade range from 18” to 36” in thickness¸ and in one location a 36” wall spans both subgrade floors to the first floor unbraced. An example of a typical mat footing can be seen in Figure 1.1. As with the other mat footings, this one is combined and sits under two pieces of large imaging equipment. It is 6’-0” thick and also services a shear wall that steps 6’ in elevation. Another area of note in the foundation design is a 6’-0” thick concrete footing which will service a cyclotron, another heavy piece of imaging equipment.

Superstructure

The first floor and the floors above to the eighth floor is a 6” one-way slab (NWC) with a compressive strength (f’c) of 5 ksi. The beams on these levels are mostly 18”x20” T-Beams, which change directions at the re-entrant corner where the building changes directions, while the girders vary, but are typically 28”x30”.

The interior and perimeter column dimensions vary but most are 20”x20” square columns with #3 ties above the first floor, and 24”x24” below grade, and all have a compressive strength of 7 ksi. The typical frame consists of four bays with three of them being approximately twenty feet in width and the other being thirty feet in width to accommodate the laboratories that occupy these spaces on almost every floor of the building.

The perimeter of the building consists of spandrel beams which tie the curtain wall system into the rest of the building at each floor level.

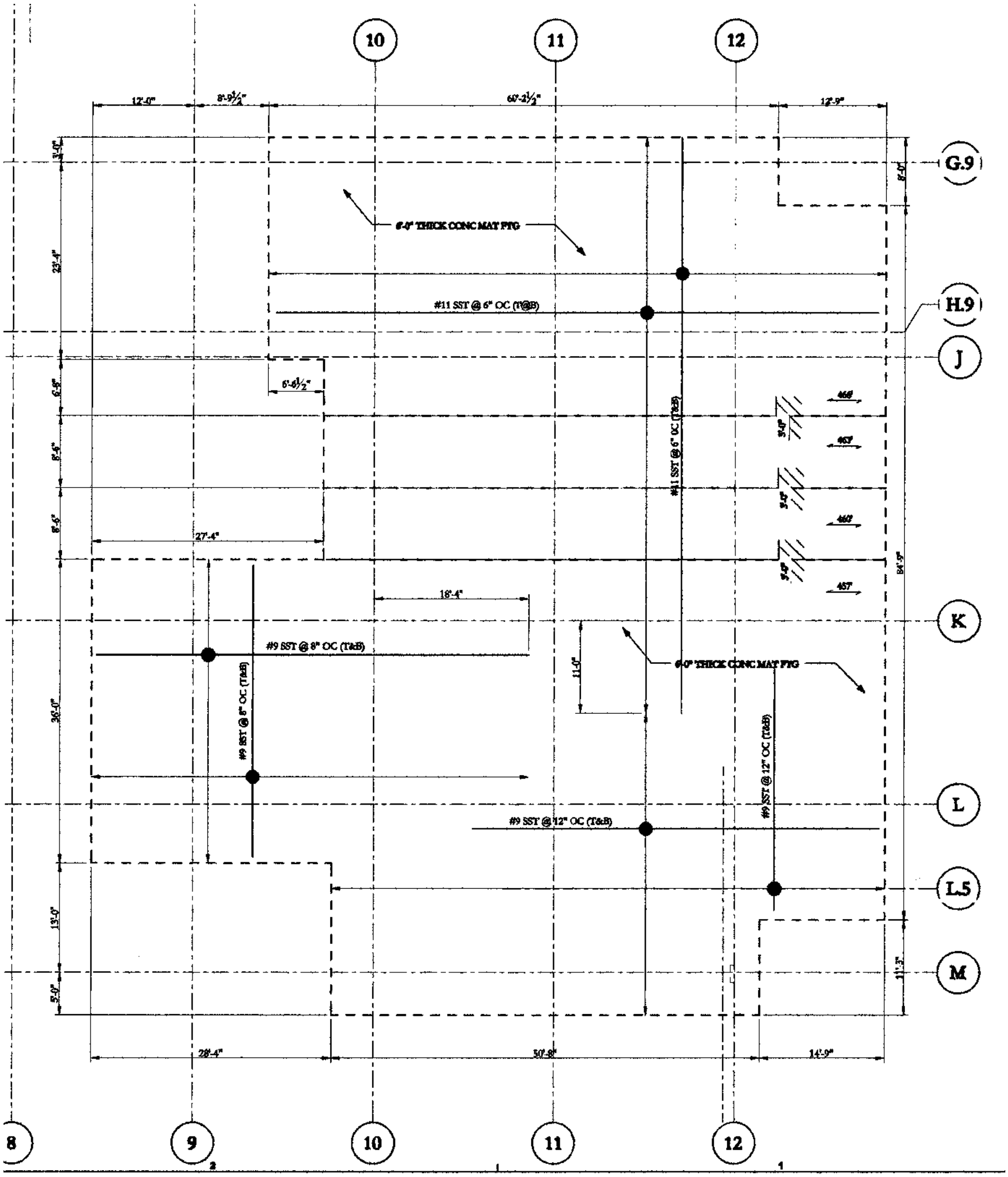

Figure 1.1 – Typical Mat Foundation Under Imaging Equipment

Lateral System

Shearwalls are used as the lateral force resisting system in the UNC Imaging Research Building. The largest shearwalls are wrapped around the main elevator and stairwell cores where the other ones encase mechanical closets. The shearwalls exist from the mechanical mezzanine to the foundation with others picking up in between. There are forty-one shearwalls either 12” or 16” thick, and several of them have openings for doors, which can be seen in the typical shear wall elevation. Figure 1.2 shows an example of the shearwalls around the main stair and elevator core, while Figure 1.3 is an example of a shear wall elevation.

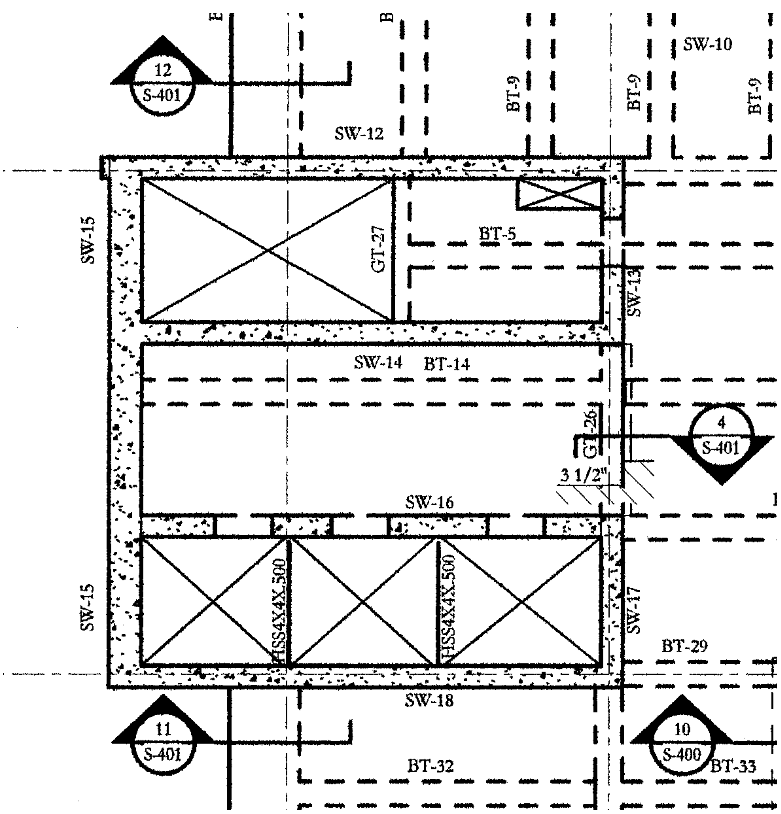

Figure 2.2 - Shearwalls around Elevator Core

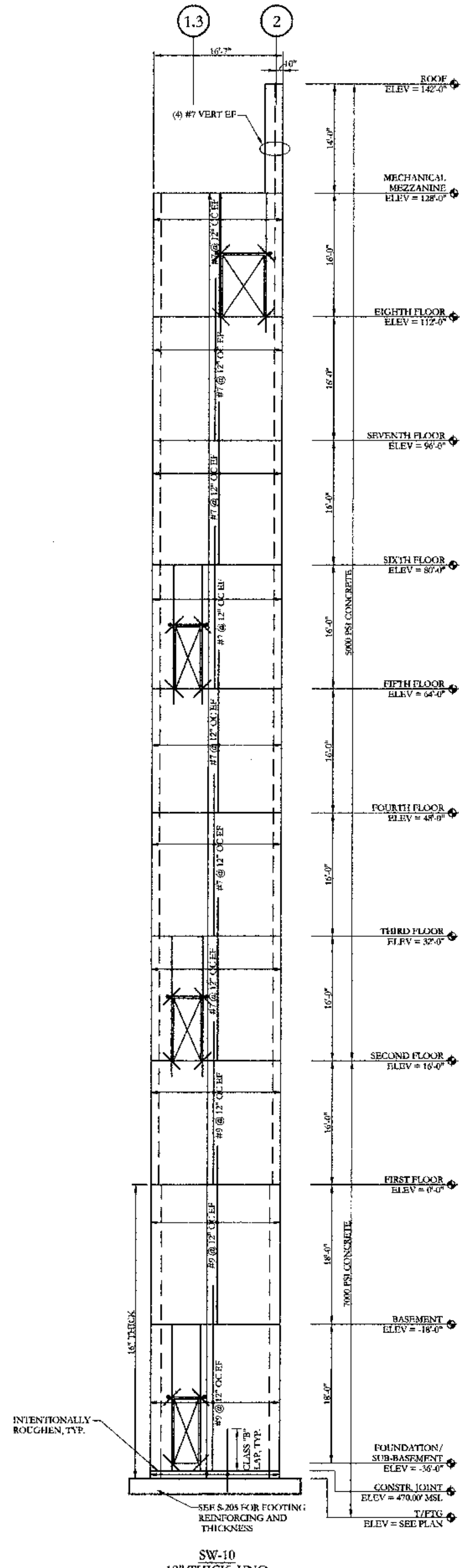

Figure 1.3 - Typical Shearwall Elevation

Codes & Design Standards

Applied to original design:

2009 North Carolina State Building Code (2006 International Building Code with revisions)

American Concrete Institute (ACI 318-05), Building Code Requirements for Structural Concrete

Substituted for thesis analysis:

American Society for Civil Engineers (ASCE 7-05), Minimum Design Loads for Buildings and Other Structures, 2005

American Concrete Institute (ACI 318-08), Building Code Requirements for Structural Concrete

Material Strength Requirement Summary:

Concrete/Reinforcing Steel (28 day compressive strength)

- Elevated Slabs on Metal Deck: 3500 psi
- Elevated Slabs and Beams: 5000 psi
- Columns, Shear Walls: 7000 psi
- Basement Walls, Site Walls: 7000 psi
- Slab on Grade, Footings, Grade Beams: 4000 psi
- Reinforcement: 60 ksi

Loads

Gravity Loads

The determination of gravity loads by Mulkey Engineers and Consultants was done using the 2009 North Carolina State Building Code (2006 International Building Code with Revisions), which adopts ASCE 7-05 for its minimum design loads for buildings. This report also uses ASCE 7-05 as the main reference in accordance with the requirements of AE Senior Thesis. In several places, Mulkey chose to use higher design loads than what was stipulated by the building code. These differences along with the rest of the design loads are noted in the Mulkey column of Table 1, while the code loads are in the ASCE 7-05 column. Calculations of the snow load are provided in Appendix A.

Table 1 -Gravity Loads

Description	Mulkey	ASCE 7-05
DEAD (DL)		
Reinforced Normal Weight Concrete	150 pcf	150 pcf
LIVE (LL)		
Roof	30 psf	20 psf
Offices	50 psf	50 psf
Public Areas, Lobbies	100 psf	100 psf
Laboratories	100 psf	60 psf
Corridors, 2nd & Above	100 psf	100 psf
Corridors Ground	100 psf	100 psf
Stairs	100 psf	100 psf
Catwalk	40 psf	40 psf
Storage	125 psf	125 psf
Heavy File Storage	200 psf	250 psf
Mechanical Rooms	150 psf	150 psf
Level B1	150 psf	N/A
SNOW (S)		
Snow	16.5 psf	16.5 psf
SUPERIMPOSED (SDL)		
Finishes, MEP, Partitions	20-25 psf	20-25 psf
Bathroom Terrazo	40 psf	N/A
Lobby Terrazo	60 psf	N/A
Mechanical Courtyard	300 psf	N/A
3T MRI Room	250 psf	N/A
7T Sheilding	75 psf	N/A
Hot Cells	350 psf	N/A
Water Tank	350 psf	N/A

Wind Loads

Wind loads were determined using ASCE 7-05 Section 6.5 which describes Method 2 – Analytical Procedure. The variables used in this analysis are located in Table 2a and the calculations that support these values can be found in Appendix B.

Table 2a - Wind Variables			ASCE 7-05 References
Basic Wind Speed	V	95 mph	(Fig. 6-1)
Directionality Factor	k_d	0.85	(Table 6-4)
Importance Factor	I	1.15	(Table 6-1)
Exposure Category		B	(Sec. 6.5.6.3)
Topographic Factor	K_{zt}	1	(Sec. 6.5.7.1)
Velocity Pressure Exposure Coefficient evaluated at Height z	K_z	Varies	(Table 6-3)
Velocity Pressure at Height z	q_z	Varies	(Eq. 6-15)
Velocity Pressure at Mean Roof Height (North/South)	q_h	25.29 psf	(Eq. 6-15)
Velocity Pressure at Mean Roof Height (East/West)	q_h	24.62 psf	(Eq. 6-15)
Equivalent Height of Structure	>	94.6'	(Table 6-2)
Intensity of Turbulence	$I_{>}$	0.252	(Eq. 6-5)
Integral Length Scale of Turbulence	$L_{>}$	454.6'	(Eq. 6-7)
Background Response Factor (East/West)	Q	0.794	(Eq. 6-6)
Background Response Factor (North/South)	Q	0.786	(Eq. 6-6)
Gust Effect Factor (East/West)	G	0.878	(Eq. 6-4)
Gust Effect Factor (North/South)	G	0.873	(Eq. 6-4)
External Pressure Coefficient (Windward)	C_p	0.8	(Fig. 6-6)
External Pressure Coefficient (E/W Leeward)	C_p	-0.47	(Fig. 6-6)
External Pressure Coefficient (N/S Leeward)	C_p	-0.5	(Fig. 6-6)

Table 2b was developed to determine the wind pressures in the north/south direction. Because the building is exposed at the first basement level in these directions the wind pressure is higher than in the east and west direction. As a result the wind base shear is also higher in the north/south direction with a total of 1536 kips. This value is roughly 400 kips than value used by Mulkey. In comparing their wind pressures to the values obtained for this report, the qz values were virtually the same, while the overall wind pressures varied with mine being almost 10 psf higher than Mulkey's at the roof. While it is hard to ascertain why, the reason behind this could be due to the fact that a constant leeward pressure is used in my calculations.

Table 2a-Wind Loads (North/South) B=282'-4" L-247'-3"

Floor	Height Above Ground- z (ft)	Story Height (ft)	Kz	Qz	Wind Pressure (psf)		Total Pressure (psf)	Force (k) of Windward only	Force (k) of Total Pressure	Story Shear Windward (k)	Story Shear Total (k)
					Windward	Leeward					
Roof	162	14.33	1.13	25.52	22.38	-15.59	37.97	73.00	123.86	36.50	61.93
Mech Mez.	148.66	18.66	1.11	25.07	22.06	-15.59	37.65	98.11	167.44	122.05	207.58
8	130	16	1.07	24.17	21.43	-15.59	37.02	96.80	167.22	219.50	374.91
7	114	16	1.03	23.26	20.80	-15.59	36.39	93.95	164.37	314.88	540.71
6	98	16	0.98	22.13	20.01	-15.59	35.60	90.39	160.81	407.05	703.30
5	82	16	0.94	21.23	19.38	-15.59	34.97	87.54	157.96	496.01	862.69
4	66	16	0.87	19.65	18.27	-15.59	33.86	82.55	152.97	581.05	1018.15
3	50	16	0.81	18.29	17.33	-15.59	32.92	78.28	148.70	661.47	1168.99
2	34	16	0.72	16.26	15.91	-15.59	31.50	71.86	142.29	736.53	1314.49
1	18	18	0.6	13.55	14.02	-15.59	29.61	71.23	150.45	808.08	1460.86
B1	0	0	0	0.00	0.00	0	0.00	0.00	0.00	843.69	1536.08
∑Story Shear (Windward) = 843.69 k					**∑Story Shear (Total) =**		**1536.08 k**				

The values obtained for the East/West direction can be found in table 2c. Also in this direction there is a large gap between the values obtained in my analysis and the values used by Mulkey used in their design. The gap though is consistent for both directions. For both cases the pressure diagram for the building can be found with the rest of the wind calculations in Appendix B.

Table 2c-Wind Loads (East/West) B=247'-3" L=282'-4"											
Floor	Height Above Ground- z (ft)	Story Height (ft)	Kz	qz	Wind Pressure (psf)		Total Pressure (psf)	Force (k) of Windward only	Force (k) of Total Pressure	Story Shear Windward (k)	Story Shear Total (k)
					Windward	Leeward					
Roof	144	13.33	1.10	24.84	21.90	-14.59	36.49	46.52	77.50	23.26	38.75
Mech Mez.	130.66	18.66	1.06	23.94	21.27	-14.59	35.86	63.37	106.84	78.20	130.92
8	112	16	1.02	23.04	20.64	-14.59	35.23	81.65	139.37	150.72	254.03
7	96	16	0.98	22.13	20.01	-14.59	34.60	79.16	136.87	231.12	392.15
6	80	16	0.93	21.00	19.22	-14.59	33.81	76.04	133.76	308.72	527.47
5	64	16	0.87	19.65	18.27	-14.59	32.86	72.29	130.01	382.88	659.35
4	48	16	0.80	18.07	17.17	-14.59	31.76	67.93	125.64	452.99	787.18
3	32	16	0.71	16.03	15.75	-14.59	30.34	62.31	120.03	518.11	910.01
2	16	16	0.58	13.10	13.70	-14.59	28.29	54.20	111.92	576.36	1025.99
1	0	0	0.00	0.00	0.00	0	0.00	0.00	0.00	603.46	1081.94
∑Story Shear (Windward) = 603.46 k					**∑Story Shear (Total) =**		**1081.94 k**				

Seismic Loads

In order to calculate the seismic forces for the Imaging Research Building, chapters 11 and 12 were of ASCE 7-05 were referenced. Mulkey also calculated their seismic loads using ASCE 7-05. Unfortunately, again there was a large difference in the base shears between my analysis and Mulkey's design. The analysis performed for this report yielded a base shear of 1822 kips whereas Mulkey designed for a base shear of 1125 kips. The building weights for both my analysis and Mulkey's design were very close, so it seems the errors lie in the calculation of the lateral forces. One possible reason as that for this is that for this analysis the Equivalent Lateral Force Method was used, despite the fact that IRB contains type two horizontal irregularities. This method was chosen for this report as a base method to approximate the seismic forces because type two horizontal irregularities require a more advanced modal response analysis which utilizes computer software which was not required for this technical report.

These calculations along with a sample calculation of the building weight for one floor, and a diagram of the story shear and base shear as a result of the seismic loads, can be found in Appendix C. Table 3a provides a list of variables used where Table 3b shows the calculations of story shear and overturning moments via excel.

Table 3a - Seismic Design Variables			ASCE 7-05 References
Site Class		C	(Table 20.3-1)
Occupancy		III	(Table 1-1)
Importance Factor		1.25	(Table 11.5-1)
Structural System		Building Frame Sytem: Ordinary Reinforced Concrete Shear Wall	(Table 12.2-1)
Spectral Response Acceleration, short	S_s	0.209 g	(USGS)
Spectral Response Acceleration, 1 s	S_1	0.081g	(USGS)
Site Coefficient	F_a	1.2	(Table 11.4-1)
Site Coefficient	F_v	1.7	(Table 11.4-2)
MCE Spectral Response Acceleration, short	S_{MS}	0.251	(Eq. 11.4-1)
MCE Spectral Response Acceleration, 1 s	S_{M1}	0.092	(Eq.11.4-2)
Design Spectral Acceleration, short	S_{DS}	0.167	(Eq. 11.4-3)
Design Spectral Acceleration, 1s	S_{D1}	0.092	(Eq. 11.4-4
Seismic Design Category	SDC	B	(Eq. 11.6-2)
Response Modification Coefficient	R	5	(Table 12.2-1)
Approximate Period Parameter	C_t	0.02	(Table 12.8-2)
Building Height (above grade)	h_n	162	
Approximate Period Parameter	x	0.75	(Table 12.8-2)
Calculated Period Upper Limit Coefficient	C_u	1.7	(Table 12.8-1)
Approximate Fundamental Period	T_a	0.92 s	(Eq. 12.8-7)
Fundamental Period Max	T_{max}	1.56	(Sec. 12.8.2)
Long Period Transition Period	T_L	8 g	(Fig. 22-15)
Seismic Response Coefficient	C_s	0.025	(Eq. 12.8-2)
Structural Period Exponent	k	1.21	(Sec. 12.8.3)

Table 3b- Seismic Loads								
Level	Story Weight W_x (k)	Height h_x (ft)	h_x^k	$w_x h_x^k$	C_{vx}	Lateral Force F_x (k)	Story Shear V_x (k)	Moments M_x (ft-k)
Roof	800	162	471.53	377227.33	0.03	55.45	0.00	8983.67
Mech Mez.	1200	148.66	424.97	509960.03	0.04	74.97	55.45	11144.64
8.00	7625	130	361.30	2754934.67	0.22	404.99	130.42	52649.02
7.00	7625	114	308.22	2350145.32	0.19	345.49	535.41	39385.39
6.00	7625	98	256.67	1957146.79	0.16	287.71	880.90	28195.84
5.00	7625	82	206.88	1577446.39	0.13	231.89	1168.61	19015.34
4.00	7625	66	159.09	1213076.00	0.10	178.33	1400.51	11769.76
3.00	7625	50	113.70	866949.01	0.07	127.45	1578.84	6372.34
2.00	7490.35	34	71.30	534061.91	0.04	78.51	1706.28	2669.35
1.00	7865.25	18	33.03	259771.95	0.02	38.19	1784.79	687.39
B1	9805.68	0	0.00	0.00	0.00	0.00	1822.98	0.00
$\sum w_i h_i^k$= 12400719.41		****$\sum F_x = V_x$= 1822.98 k**			**$\sum$Moments M_x = 180872.73 ft-k**			
Total Building Weight (Above Grade) = 72911.28 k								

Spot Checks

Spot checks were performed on the typical interior column and the typical interior beam. For the column, a gravity load take down based on ASCE 7-05 and Mulkey's loads was done to determine Pu at that level in question, which can be seen below. Also, even though not required for this analysis, moment on the column due to seismic load was calculated. Unfortunately, it seems that the combination of Pu and Mu results in column failure, which is illustrated in Figure 4.1. The lateral load analysis and its resulting moments will be examined further in future reports.

The typical interior beam calculations checked out better. Max moments and shears were determined from continuous beam coefficients from section 8.3 in ACI 318-08. These calculations can also be found in Appendix D.

Typical Interior Column Load (H.9-7) (ASCE 7-05)					
Level	DL (psf)	LL (psf)	1.2D + 1.6L (psf)	Area (sf)	Total Weight (k)
3	157.5	60	285	579.55	165.17
4	157.5	60	285	579.55	165.17
5	157.5	60	285	579.55	165.17
6	157.5	60	285	579.55	165.17
7	157.5	60	285	579.55	165.17
8	307.5	150	609	579.55	352.95
Mech Mezz	150	150	420	547.5	229.95
Roof	0	0	0	0	0.00
				Total =	1408.75

Typical Interior Column Load (H.9-7) (Mulkey)					
Level	DL (psf)	LL (psf)	1.2D + 1.6L (psf)	Area (sf)	Total Weight (k)
3	157.5	100	349	579.55	202.26
4	157.5	100	349	579.55	202.26
5	157.5	100	349	579.55	202.26
6	157.5	100	349	579.55	202.26
7	157.5	100	349	579.55	202.26
8	307.5	150	609	579.55	352.95
Mech Mezz	150	150	420	547	229.74
Roof	0	0	0	0	0.00
				Total =	1594.00

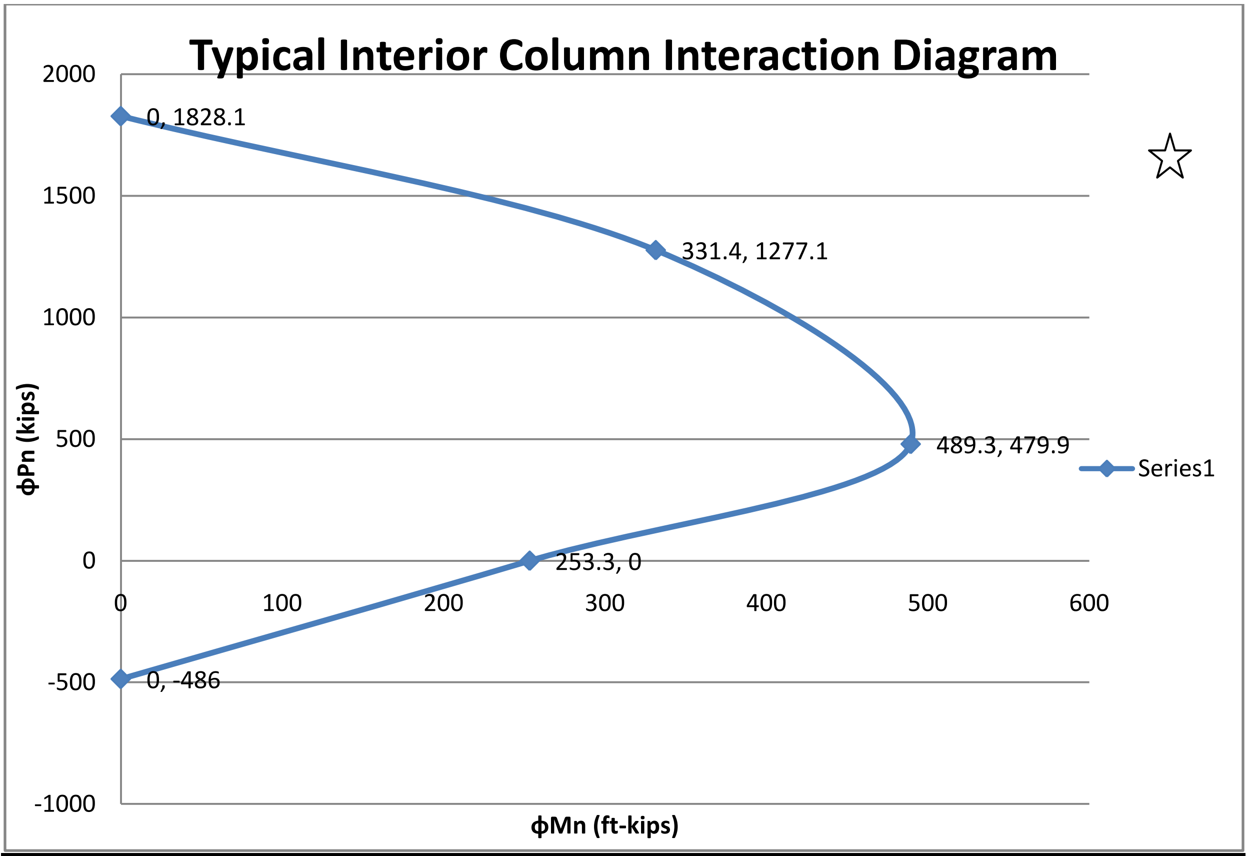

Figure 4.1- Typical Interior Column Interaction Diagram

Conclusions

After an examination of the existing structural system and calculations of various gravity and lateral loads, it was determined that the Imaging Research Building was adequately designed to withstand these forces. Following an analysis of the wind loads using ASCE 7-05 via Method 2 and seismic loads via the Equivalent Lateral Force Procedure, the controlling base shear was due to seismic with V=1822 kips. Mulkey also determined that the controlling lateral force was due to seismic loads, but found V=1125 kips. The possible reason for this, as noted in the report, is that the Equivalent Lateral Force Procedure is not the most accurate method for determining the loads for a building type like IRB's. Because the building has type 2 horizontal irregularities (re-entrant corners), the loads are more accurately determined using Modal Response Procedure which involves computer modeling.

Spot checks done on the typical interior column and the typical interior beam proved that the determination and accumulation of loads done within this report were comparable to those done by Mulkey Engineers and Consultants. These two components were found to be satisfactorily designed based strictly on gravity loads, but the column failed when moment was introduced (though not required for this report). As research continues for IRB, the lateral forces will be more closely examined to see if it can be determined where the errors lies, and what effect these forces will have on the rest of the framing, shearwalls, and foundation.

Appendix A- Gravity Load Calculations

MULKEY ENGINEERS & CONSULTANTS
P.O. Box 33127 • Raleigh, NC 27636-3127
Phone: (919) 851-1912 • Fax: (919) 851-1918

PROJECT NAME UNC Imaging Research Building
PROJECT NO. SHEET 1 OF 1
SUBJECT Snow Load
PREPARED BY DRH DATE 10/1/09 CHECKED BY DATE

Snow Load Calculations

Ground Snow load $P_g = 15$ psf (Fig. 7-1)

Flat roof snowload $p_f = 0.7\, C_e C_t I\, p_g$

Exposure factor: $C_e = 1.0$ (parapet) → partially exposed

Thermal factor: $C_t = 1.0$

Importance Factor: $I = 1.1$ Occupancy Cat: III

$p_f = 0.7(1.0)(1.0)(1.1)(15) = 11.55$ psf

but not less than...

$p_f = (I)\,p_g = 1.1(15) = \boxed{16.5 \text{ psf}}$

Appendix B- Wind Calculations

MULKEY
ENGINEERS & CONSULTANTS
P.O. Box 33127 • Raleigh, NC 27636-3127
Phone: (919) 851-1912 • Fax: (919) 851-1918

PROJECT NAME UNC Imaging Research Building
PROJECT NO. SHEET 1 OF 8
SUBJECT Wind Calcs
PREPARED BY DRH DATE 9/30/09 CHECKED BY DATE

Wind Calculations

ASCE 7-05 Method 2 - Analytical Procedure

V = 95 mph Occupancy Classification: Cat III

$I_w = 1.15$ Exposure Category: B

K_z

Level	Height (ft)	K_z (Interpolated)
B1	0	0
1	18	0.6
2	34	0.72
3	50	0.81
4	66	0.87
5	82	0.94
6	98	0.98
7	114	1.03
8	130	1.07
Mech Mez.	148.66	1.11
Roof	162	1.13

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

$q_z = 0.00256 (K_z)(1.0)(0.85)(95)^2(1.15)$

Varies at levels

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

N/S: $q_h = 0.00256 (1.12)(1.0)(0.85)(95)^2(1.15) = \boxed{25.29 \text{ psf}}$

$\rightarrow h = 157.66$ (N/S)

E/W: $q_h = 0.00256 (1.09)(1.0)(0.85)(95)^2(1.15) = \boxed{24.62 \text{ psf}}$

$\rightarrow h = 139.66$ (E/W)

MULKEY ENGINEERS & CONSULTANTS
P.O. Box 33127 • Raleigh, NC 27636-3127
Phone: (919) 851-1912 • Fax: (919) 851-1918

PROJECT NAME UNC - IRB
PROJECT NO. SHEET 2 OF 8
SUBJECT Wind Calcs
PREPARED BY DRH DATE 9/30/09 CHECKED BY DATE

Gust Effect Factors, G & G_f

$n_1 = 100/H = 100/157.66 = 0.63 < 1\ Hz$ ∴ **Flexible**

$g_Q = g_v = 3.4$

$$g_R = \sqrt{2\ln(3600 n_1)} + \frac{0.577}{\sqrt{2\ln(3600 n_1)}}$$

$$g_R = \sqrt{2\ln(3600(0.63))} + 0.577/\sqrt{2\ln(3600(0.63))} = 4.08$$

$\bar{z} = 0.6h = 0.6(157.66) = 94.6' > z_{min} = 30'$

$$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6} = 0.30\left(\frac{33}{94.6}\right)^{1/6} = 0.252$$

$$L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}} = 320\left(\frac{94.6}{33}\right)^{1/3.0} = 454.6'$$

$$Q = \sqrt{\frac{1}{1 + 0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}}$$

N/S: $B = 282.33'$ $L = 247.25'$ E/W: $B = 247.25'$ $L = 282.33'$

$Q_{N/S} = 0.786$ $Q_{E/W} = 0.794$

$$\bar{V}_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}} V\left(\frac{88}{60}\right) = 0.45\left(\frac{94.6}{33}\right)^{1/4}(95)\left(\frac{88}{60}\right) = 81.59$$

$$N_1 = \frac{n_1 L_{\bar{z}}}{V_{\bar{z}}} = \frac{0.63\,(454.6)}{81.59} = 3.51$$

$$R_n = \frac{7.47 N_1}{(1+10.3N_1)^{5/3}} = \frac{7.47\,(3.51)}{(1+10.3(3.51))^{5/3}} = 0.0634$$

$$R_B = \frac{1}{n} - \frac{1}{2n^2}\left(1 - e^{-2n}\right) \quad \text{for } n > 0$$

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PROJECT NAME UNC-IRB
PROJECT NO. SHEET 3 OF 8
SUBJECT Wind Calcs
PREPARED BY DRH DATE 9/30/09 CHECKED BY DATE

$\eta = 4.6 n_1 B / \bar{V}_{\bar{z}}$

N/S: $\eta = 4.6(0.63)(282.33)/81.59 = 10.03$

E/W: $\eta = 4.6(0.63)(247.25)/81.59 = 8.78$

R_B N/S: $\frac{1}{10.03} - \frac{1}{2(10.03)^2}\left(1 - e^{-2(10.03)}\right) = 0.0947$

R_B E/W: $\frac{1}{8.78} - \frac{1}{2(8.78)}\left(1 - e^{-2(8.78)}\right) = 0.0569$

$R_h = \frac{1}{\eta} - \frac{1}{2\eta^2}\left(1 - e^{-2\eta}\right)$ for $\eta > 0$

$\eta = 4.6 n_1 h / \bar{V}_{\bar{z}} = 4.6(0.63)(157.66)/81.59 = 5.6$

$R_h = \frac{1}{5.6} - \frac{1}{2(5.6)^2}\left(1 - e^{-2(5.6)}\right) = 0.163$

$R_L = \frac{1}{\eta} - \frac{1}{2\eta^2}\left(1 - e^{-2\eta}\right)$ for $\eta > 0$

$\eta = 15.4 n_1 L / \bar{V}_{\bar{z}}$

N/S: $\eta = 15.4(0.63)(247.25)/81.59 = 29.4$

E/S: $\eta = 15.4(0.63)(282.33)/81.59 = 33.57$

R_L N/S $= \frac{1}{29.4} - \frac{1}{2(29.4)^2}\left(1 - e^{-2(29.4)}\right) = 0.0334$

R_L E/W $= \frac{1}{33.57} - \frac{1}{2(33.57)^2}\left(1 - e^{-2(33.57)}\right) = 0.0293$

$\beta = 2\%$ (concrete)

$R = \sqrt{\frac{1}{\beta} R_n R_h R_B (0.53 + 0.47 R_L)}$

$R_{N/S} = 0.0163$ $R_{E/W} = 0.0126$

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PROJECT NAME UNC- IRB
PROJECT NO.
SHEET 4 OF 8
SUBJECT Wind Calcs
PREPARED BY DRH DATE 9/30/09 CHECKED BY DATE

$$G_f = 0.925\left(\frac{1+1.7 I_{\bar{z}}\sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1+1.7 g_v I_{\bar{z}}}\right)$$

$$G_f\ N/S = 0.925\left(\frac{1+1.7(0.252)\sqrt{3.4^2(0.786)^2+4.08^2(0.0163)^2}}{1+1.7(3.4)(0.252)}\right)$$

$$\boxed{G_f\ N/S = 0.873}$$

$$G_f\ E/W = 0.925\left(\frac{1+1.7(0.252)\sqrt{3.4^2(0.794)^2+4.08^2(0.1012)^2}}{1+1.7(3.4)(0.252)}\right)$$

$$\boxed{G_f\ E/W = 0.878}$$

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PROJECT NAME UNC - IRB
PROJECT NO. SHEET 5 OF 8
SUBJECT Wind Calcs
PREPARED BY DRH DATE 9/30/09 CHECKED BY DATE

Wind Calculations Cont. – MWFRS

Enclosed Building

Parapet Pressure

$q_p = 0.00256 K_z K_{zt} K_d V^2 I$ K_z @ 164.33 = 1.14
↳ Parapet ht N/S

$q_p = 0.00256(1.14)(1.0)(0.85)(95^2)(1.15)$

$q_p = 25.75$ psf (N/S)

$q_p = 0.00256(1.10)(1.0)(0.85)(95^2)(1.15)$ K_z @ 146.33 = 1.10
↳ Parapet ht E/W

$q_p = 24.90$ psf

$GC_{pn} = +1.5$ (windward) $GC_{pn} = -1.0$ (leeward)

$P_p = q_p GC_{pn}$

N/S:

Windward: $P_p = 25.75(1.5) = 38.63$ psf

Leeward: $P_p = 25.75(-1.0) = -25.75$ psf

E/W:

Windward: $P_p = 24.90(1.5) = 37.35$ psf

Leeward: $P_p = 24.90(-1.0) = -24.90$ psf

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PROJECT NO.
SHEET 6 OF 8
SUBJECT Wind Calcs
PREPARED BY DRH DATE 9/30/09 CHECKED BY DATE

Wind Calculations Cont.

Pressure Coefficient C_p (Fig 6-6)

N/S:

Windward: $C_p = 0.8$

Leeward: $L/B = 247.25/282.33 = 0.876$

$C_p = -0.5$

E/W:

Windward: $C_p = 0.8$

Leeward: $L/B = 282.33/247.25 = 1.14$

$C_p = -0.47$

Pressure

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

$p_h = q_h G_f C_p - q_h (GC_{pi})$ (Leeward)

w/ $GC_{pi} = +0.18, -0.18$ for enclosed bldgs (Fig 6-5)

N/S:

Windward $p_z = (q_z)(0.873)(0.8) - 25.29(-0.18)$

$\boxed{p_z = (q_z)(0.6984) + 4.552}$

Leeward $p_h = (25.29)(0.873)(-0.5) - 4.552 = \boxed{-15.59 \text{ psf}}$

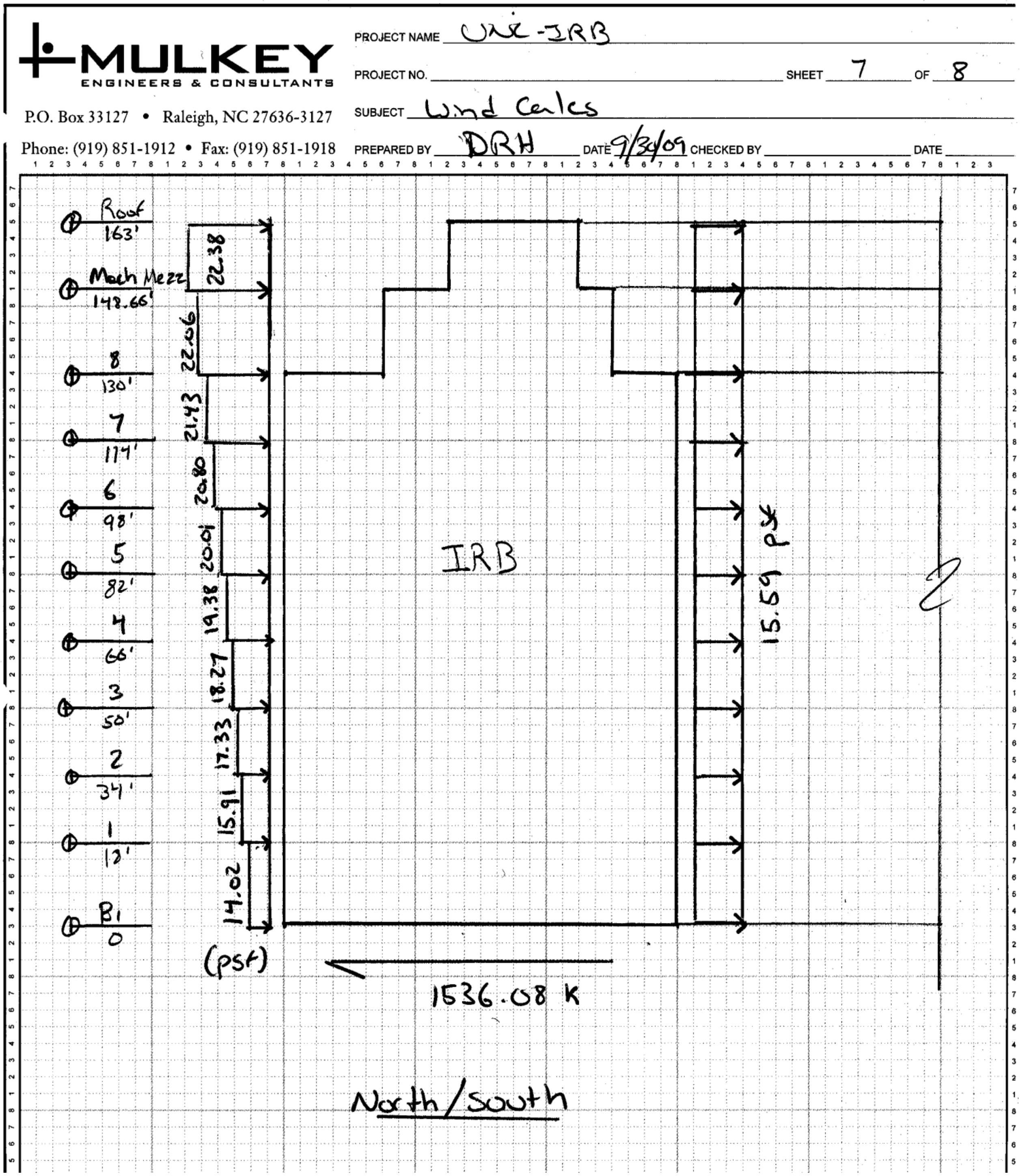
E/W:

Windward $p_z = (q_z)(0.878)(0.8) - 24.62(-0.18)$

$\boxed{p_z = (q_z)(0.7024) + 4.43}$

Leeward $p_h = (24.62)(0.878)(-0.47) - 4.43 = \boxed{-14.59 \text{ psf}}$

ME-02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PROJECT NAME UNC-IRB
PROJECT NO.
SHEET 7 OF 8
SUBJECT Wind Calcs
PREPARED BY DRH
DATE 9/30/09
CHECKED BY
DATE
Roof 163'
Mech Mezz 148.66'
8 130'
7 114'
6 98'
5 82'
4 66'
3 50'
2 34'
1 18'
B1 0
22.38
22.06
21.43
20.80
20.01
19.38
18.27
17.33
15.91
14.02
(psf)
IRB
15.59 psf
1536.08 K
North/south

Daniel R. Hesington Chapel Hill, NC

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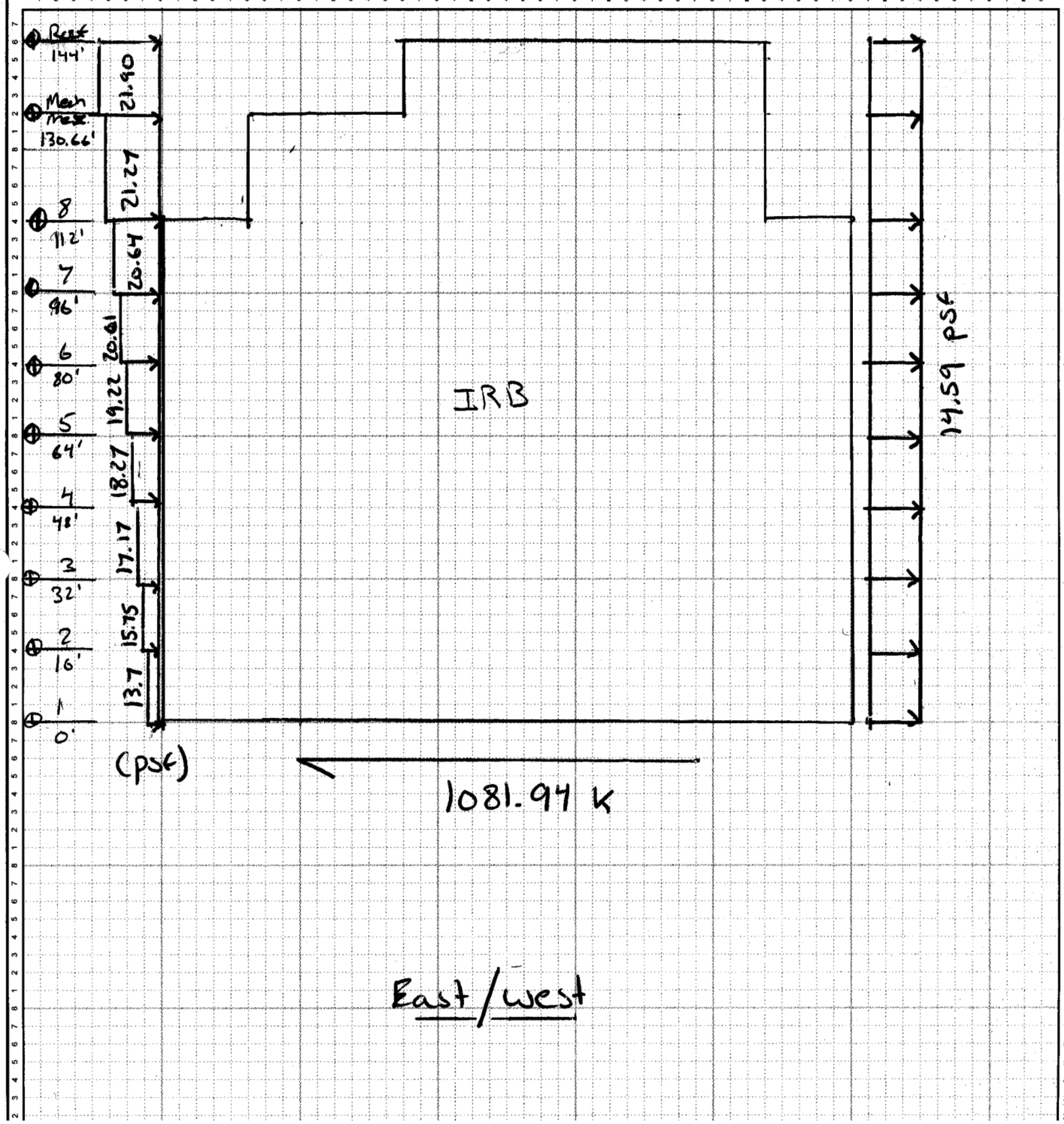
PROJECT NO. ____ SHEET 8 OF 8
SUBJECT Wind Calcs
PREPARED BY DRH DATE 9/30/09 CHECKED BY ____ DATE ____



Appendix C - Seismic Calculations

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PROJECT NAME UNC Imaging Research Building
PROJECT NO. SHEET 1 OF 2
SUBJECT Seismic Calculations
PREPARED BY DRH DATE 10/1/09 CHECKED BY DATE

Seismic Calculations (Assuming rigid diaphragm for Tech 1 analysis)

Ss: 0.209 g ; S1: 0.081 g (From USGS.gov)

Fa = 1.2 ; Fv: 1.7 Site Class C

Is: 1.25

Sms: FaSs: 1.2(0.209) = 0.251

Occupancy Cat: III

Sm1 = FvS1 = 1.7(0.081) = 0.138

SDS = 2/3 Sms = 2/3(0.251) = 0.167 g

SD1 = 2/3 SM1 = 2/3(0.138) = 0.092 g

Seismic Design Category based on Short Period Response Acceleration Parameter

SDC = B

Seismic Design Category based on 1-s period Response Acceleration Parameter

SDC = B

∴ Seismic Design Category → $\boxed{B}$

Determine structure fundamental period, T

Ts = SD1/SDS = 0.092/0.167 = 0.551

Ta: Ct hn^x: 0.02(162)^0.75 = 0.92 s

↳ ordinary Reinforced Concrete Shear Walls

TL = 8 g

T = Ta = 0.92 < Tmax : CuTa: 1.7(0.92) = 1.56

T = 0.92 < 3.5Ts = 3.5(0.551) = 1.93

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PROJECT NAME UNC-IRB

PROJECT NO. SHEET 2 OF 2

SUBJECT Seismic Calcs

PREPARED BY DRH DATE 10/1/09 CHECKED BY DATE

Seismic Calculations Cont.

Determination of Type 2 Horizontal Irregularity - Re-Entrant Corners

West elevation: $\frac{247.25-114.25}{247.25} = 0.54\,(100) = 54\%$

North elevation: $\frac{179.75}{282.33} = 0.64\,(100) = 64\%$

∴ Type 2 Horizontal Irregularity - Reentrant Corners

- ASCE 7-05 requires modal Response spectrum Analysis or seismic response history procedure.
- For Tech 1 - Using Equivalent Lateral Force Procedure, differences will be noted in report

$$C_s = \min \begin{cases} \dfrac{S_{DS}}{\left(\frac{R}{I}\right)} = \dfrac{0.167}{\left(\frac{5}{1.25}\right)} = 0.04175 \\[2ex] \dfrac{S_{D1}}{T(R/I)} = \dfrac{0.092}{0.92(5/1.25)} = \boxed{0.025} \geq 0.01 \\[2ex] \dfrac{S_{D1}T_L}{T^2(R/I)} = \dfrac{0.092(8)}{(0.92)^2(5/1.25)} = 0.217 \end{cases}$$

$V = C_sW = 0.025\,(72{,}705.68) = 1817.6^k$

$k = 1.21$ (From interpolation)

$W_x h_x^k$ = Varies $\quad \sum w_i h_i^k =$

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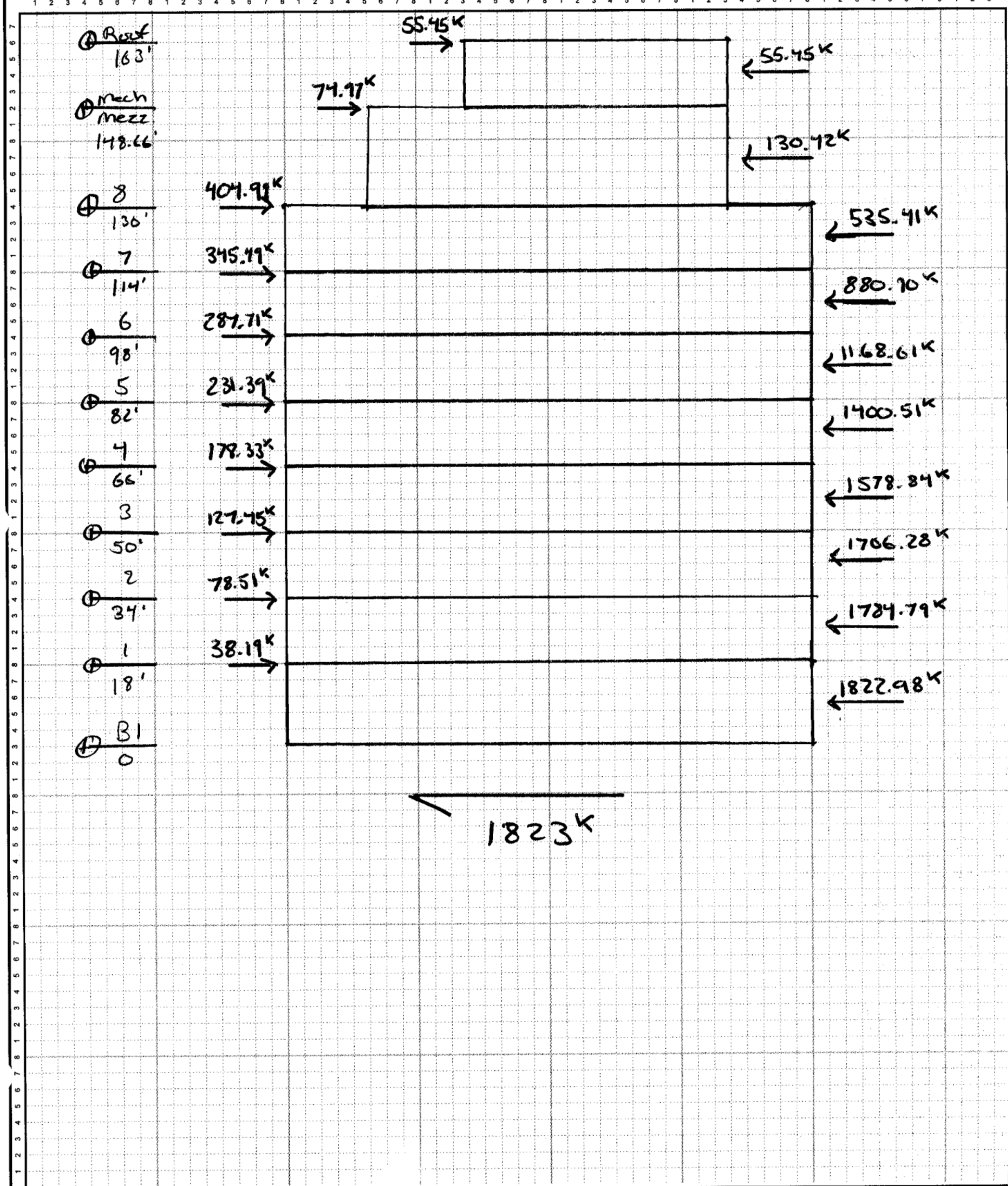
PROJECT NAME UNC - IRB

PROJECT NO. SHEET 1 OF 1

SUBJECT Seismic Calcs

PREPARED BY DRH DATE 10/1/09 CHECKED BY DATE



ME-02

Building Weight				
Level B1				
SLABS				
Designation	Thickness	Area (sf)		Weight (k)
S_1	6"	32086		2406.45
COLUMNS				
Size	Quantity			Weight (k)
20"x20"	43			58.05
24"x24"	27			36.45
30"x20"	2			2.70
30" Dia.	2			2.70
Total	74			99.90
BEAMS				
Designation	Quantity	Size	Length	Weight (k)
bt-14	8	18"x20"	14.167	42.50
	9	18"x20"	13.083	44.16
	4	18"x20"	15.83	23.75
	3	18"x20"	17.167	19.31
	1	18"x20"	19	7.13
	2	18"x20"	20.167	15.13
bb-16	14	18"x20"	7.5	39.38
	4	18"x20"	19	28.50
bt-16	1	18"x20"	7.5	2.81
bb-8	3	18"x20"	17.66	19.87
	11	18"x20"	19	78.38
bb-2	1	24"x20"	17.66	8.82
bb-10	1	18"x20"	15.83	5.94
	3	18"x20"	11.25	12.66
bb-11	2	18"x20"	17.66	13.25
18x20	4	18"x20"	19	28.50
bb-4	3	33"x78"	19	152.83
bt-25	40	18"x20"	19	285.00
	2	18"x20"	17.66	13.25
bt-13	15	18"x20"	19	106.88
bt-34	1	18"x20"	14	5.25
	5	18"x20"	19	35.63
bb-32	1	24"x20"	19	9.49
bb-3	4	18"x20"	19	28.50
bt-15	1	18"x14"	17.66	4.64

bb-23	2	18"x20"	7	5.25
bb-19	2	18"x20/18"	19	14.25
bt-33	2	18"x20/18"	19	14.25
bb-29	1	18"x20/18"	16	6.00
bb-30	1	48"x20"	16	15.98
	1	48"x20"	19	18.98
bb-34	1	18"x20"	19	7.13
bb-21	1	18"x18"	19	6.41
bb-17	4	18"x18"	19	25.65
bb-31	6	48"x20"	19	113.89
bb-18	6	18"x18"	9	18.23
bb-26	4	18"x20"	19	28.50
bb-12	3	18"x30"	28	47.25
18x30	2	18"x30"	19	21.38
bb-24	2	18"x20"	19	14.25
18x42	1	18"x42"	19	14.96
bb-25	5	18"x18"	19	32.06
bb-22	1	48"x42"	19	39.90
bb-8	8	18"x20"	19	57.00
bb-6	2	20"x30"	19	23.75
bb-7	2	20"x30"	19	23.75
bb-27	2	18"x20"	19	14.25
bb-28	2	12"x20"	19	9.50
Total	204		832.794	1604.07

GIRDERS				
Designation	Quantity	Size	Length (ft)	Weight (k)
gb-2	2	32"x30"	28	55.99
gb-3	2	32"x30"	32	63.99
gb-5	1	16"x20"	26	8.67
	1	16"x20"	6	2.00
gb-6	1	28"x20"	16	9.33
gb-7	1	28"x20"	10	5.83
gb-8	1	28"x20"	22	12.83
gb-9	1	24"x20"	7	3.50
gb-10	1	24"x20"	13	6.50
gb-11	1	28"x30"	34	29.75
gb-12	1	28"x20"	20	11.66
gb-13	2	30"x30"	30	56.25
gb-14	3	30"x30"	34	95.63
gb-15	2	28"x20"	20	23.33
gb-16	1	28"x30"	3	2.62
gb-17	1	28"x30"	16	14.00

gb-18	1	28"x78"	19	43.23
gb-19	1	24"x20"	16	8.00
gb-20	1	24"x20"	3	1.50
gb-21	1	16"x20"	15	5.00
gb-22	1	28"x30"	34	29.75
gb-23	1	28"x30"	20	17.50
gb-24	2	20"x20"	21.33	17.77
gb-25	1	20"x20"	19	7.92
gb-26	2	28"x20"	19	22.16
gb-27	4	28"x18"	21.66	45.49
gb-28	3	28"x30/28"	31.333	82.24
gb-29	1	12"x20"	20	5.00
gb-30	1	18"x20"	10	3.75
gb-31	1	21"x20"	19	4.99
gb-32	1	28"x42"	20	24.50
gb-33	1	28"x42"	19	23.27
gb-34	1	28"x30"	20	17.50
gb-35	1	28"x30"	31.33	27.41
gb-36	1	28"x20"	20	11.66
gb-37	3	28"x20"	20	34.99
gb-38	3	28"x20"	21.66	37.90
gb-39	3	28"x20"	20	34.99
gb-40	1	20"x20"	31.33	13.05
gb-41	1	28"x20"	31.33	18.27
Totals	59		819.97	939.72

SHEARWALLS

Designation	Quantity	Thickness (")	Length	Weight (k)
sw-1	1	16	33	59.40
sw-2	1	16	13.25	23.85
sw-3	1	16	13.25	23.85
sw-4	1	16	33	59.40
sw-5	1	16	13.25	23.85
sw-6	1	12	17.41667	23.51
sw-7	1	12	12.8333	17.32
sw-8	1	12	12.8333	17.32
sw-9	1	12	12.8333	17.32
sw-10	1	12	17.4167	23.51
sw-11	1	12	12.8333	17.32
sw-12	1	16	32.666	58.80
sw-13	1	16	12.5	22.50
sw-14	1	16	32.666	58.80

sw-15	2	16	33.8333	121.80
sw-16	1	16	32.666	58.80
sw-17	1	16	11.5	20.70
sw-18	1	16	32.666	58.80
sw-19	1	12	13.58333	18.34
sw-20	1	12	12.5	16.88
sw-21	1	12	13.58333	18.34
sw-22	1	12	12.5	16.88
sw-23	1	12	12.8333	17.32
sw-24	1	12	12.8333	17.32
sw-25	1	12	25	33.75
sw-26	1	12	12.5	16.88
sw-27	1	12	25	33.75
sw-28	1	12	12.5	16.88
sw-29	1	12	13.5	18.23
sw-30	1	12	34.6667	46.80
sw-31	1	12	14.5	19.58
sw-32	1	12	34.666	46.80
sw-33	1	16	13.5	24.30
Total	34		644.08	1068.89
EXTERIOR WALLS				
Perimeter		Thickness (")		Weight (k)
1068.33		24		2884.49
SUPERIMPOSED DL				
Description	Weight/Unit Area	Area		Weight (k)
(Partion's, finishes, MEP)	25	32086		802.15
Total Weight (k)				9805.68

Appendix D – Spot Checks

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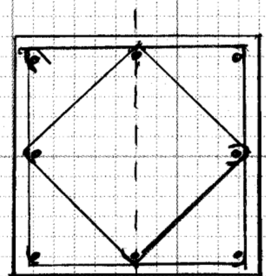
PROJECT NAME UNC Imaging Research Building
PROJECT NO. SHEET 1 OF 9
SUBJECT Spot Checks
PREPARED BY DRH DATE 10/3/09 CHECKED BY DATE

Spot Checks Calculations

Column Check

20"x20" (8) #9, #3 ties @ 18" o/c

20"



20"

$f'c = 7000$ psi

$fy = 60$ ksi

$As = (8)(1.00) = 8 \text{ in}^2$

a) Pure Axial Strength, Po

$\varepsilon_y = \frac{f_y}{E_s} = \frac{60}{29{,}000} = 0.00207$

$Po = 0.85 f'c A_c + As f_y$

$Po = 0.85(7)[(20 \cdot 20) - 8] + 8(60)$

$Po = 2812.4^k$

b) Balanced Strain Condition (Mb, Pb)

$C = \frac{0.003}{0.003 + \varepsilon_y} d_2 = \frac{0.003\,(17.5625)}{0.003 + 0.00207} = 10.39"$

$d_2 = 20 - 1.5 - 3/8 - \frac{9/8}{2} = 17.5625$

$a = B_1 C = 0.7(10.39) = 7.27"$

$\varepsilon_{s1} = \frac{0.003}{c}(c - d_1) = \frac{0.003}{7.27}(7.27 - 2.4375) = 0.00199 < \varepsilon_y$

$f_{s1} = 0.00199(29{,}000) = 57.71$ ksi

$\varepsilon_{s2} = \frac{0.003}{7.27}(7.27 - 10) = -0.00113$

$f_{s2} = -0.00113(29{,}000) = -32.77$ ksi

$\varepsilon_{s3} = \frac{0.003}{7.27}(7.27 - 17.5625) = -0.00425 \rightarrow f_{s3} = -f_y = -60$ ksi

ME-02

Daniel R. Hesington Chapel Hill, NC

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PROJECT NAME UNC-IRB
PROJECT NO. SHEET 2 OF 9
SUBJECT Spot Checks
PREPARED BY DRH DATE 10/3/09 CHECKED BY DATE

$P_b = 0.85(7)(20)(0.7)(7.27) + 3(57.71) + 2(-32.77) + 3(-60)$

$P_b = 533.2^K$

$M_b = 0.85(7)(20)(0.7)(7.27)\left(10 - \frac{0.7(10.39)}{2}\right) +$

$+ 3(57.71)(10-2.4375) + 2(-32.77)(10-10) +$

$+ 3(-60)(10-17.5625) = 6524.22''K$

$M_b = 543.69^K$

$M_b, P_b = (543.7, 533.2^K)$

c) Pure Bending

Assume ε_{s_1} does not yield, ε_{s_2} & ε_{s_3} do

$f_{s_1} = \frac{0.003}{c}(c-2.4375)(29{,}000) = 87 - \frac{212.1}{c}$

$f_{s_2} = -60\text{ ksi}$

$f_{s_3} = -60\text{ ksi}$

$\Sigma F = 0 = 0.85(7)(20)(0.7)(c) + 3f_{s_1} + 2f_{s_2} + 3f_{s_3}$

$0 = 83.3c + 3(87 - 212.1/c) + 2(-60) + 2(-60)$

$240 = 83.3c + 261 - 636.3/c$

$240c = 83.3c^2 + 261c - 636.3$

$0 = 83.3c^2 + 21c - 636.3$

$c = 2.64''$

Verify: $\varepsilon_{s_1} = (0.003/2.64)(2.64 - 2.4375) = 0.000230 < 0.00207$

$f_{s_1} = 6.67\text{ ksi} \therefore \text{ok}$

$\varepsilon_{s_2} = (0.003/2.64)(2.64 - 10) = -0.008 > 0.00207$

$\therefore f_{s_2} = -60\text{ ksi}\quad f_{s_3} = -60\text{ ksi}$

ME-02

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PROJECT NAME UNC-IRB

PROJECT NO. SHEET 3 OF 9

SUBJECT Spot Checks

PREPARED BY DRH DATE 10/3/09 CHECKED BY DATE

$M_o = 0.85(7)(20)(0.7)(2.64)\left(10 - \frac{0.7(2.64)}{2}\right) +$

$+ 3(6.67) + 2(-60)(10-10) + 3(-60)(10-17.5625)$

$= 3377.18\text{"K}$

$M_o = 281.4\text{'K}$

d) Pure Tension, T_o

$C = -\infty \quad \therefore \varepsilon_s = -\varepsilon_y$

$T_o = \Sigma A_{si} f_{si}$ where $f_{si} = -f_y = -60$ ksi

$T_o = (9 \text{ in}^2)(-60 \text{ ksi}) = -540$ K

e) A point between a) & b)

Choose $C = h = 20"$

$\varepsilon_{s1} = \frac{0.003}{20}(20-2.4375) = 0.00263 > \varepsilon_y$

$\therefore f_{s1} = 60$ ksi

$\varepsilon_{s2} = \frac{0.003}{20}(20-10) = 0.0015 < 0.00207$

$\therefore f_{s2} = 43.5$ ksi

$\varepsilon_{s3} = \frac{0.003}{20}(20-17.5625) = 0.000366 < 0.00207$

$\therefore f_{s3} = 10.60$ ksi

$P_n = 0.85(7)(20)(0.7)(20) + 3(60) + 2(43.5) + 3(10.60)$

$P_n = 1964.8^K$

$M_n = 0.85(7)(20)(0.7)(20)\left(10 - \frac{0.7(20)}{2}\right) + (3)(60)(10-2.4375)$

$+ 2(43.5)(10-10) + 3(10.6)(10-17.5625)$

$M_n = 6118.8\text{"K} = 509.9\text{'K}$

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PROJECT NAME UNC-IRB
PROJECT NO.
SHEET 4 OF 9
SUBJECT Spot checks
PREPARED BY DRH DATE 10/3/09 CHECKED BY DATE

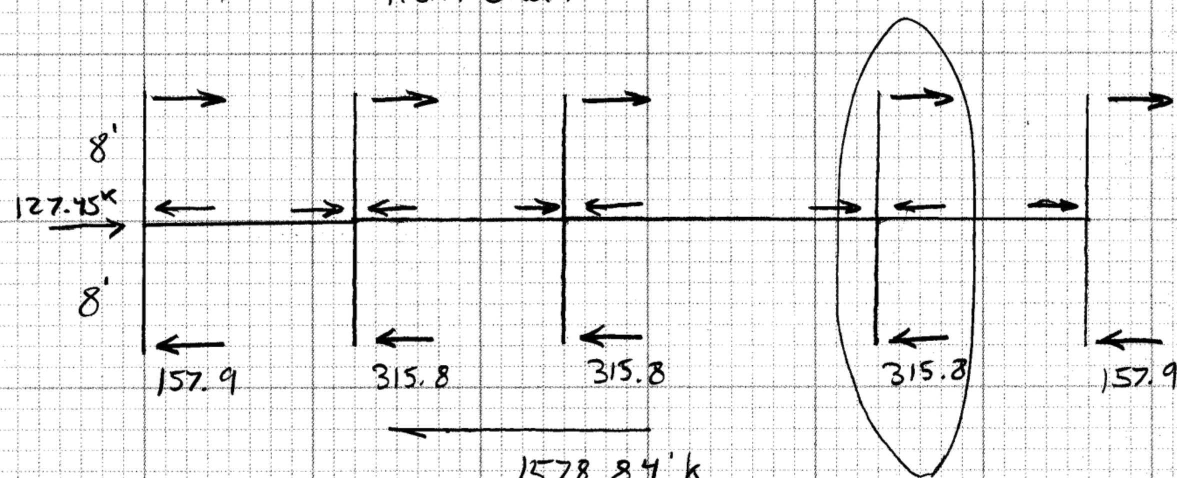
Column Spot Check Cont.

ϕ - Factor Chart:

P_n (k)	M_n ('k)	C (in)	ε_t : ε_s	ϕ	ϕP_n(k)	ϕM_n('k)
2812.4	0	∞	0.003	0.65	1828.1	0
1964.8	509.9	20"	0.000366	0.65	1277.1	331.4
533.2	543.7	10.39"	-0.00425	0.9	479.9	489.3
0	281.4	2.64"	-0.0169	0.9	0	253.3
-540	0	$-\infty$	—	0.9	-486	0

Level 3

From chart: P_u = 1409 k



Seismic

$M_u = 315.8\,(16/2)$

$= 2526.4\text{'k}$

Pt: (2526.4, 1409)

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PROJECT NAME UNC-IRB

PROJECT NO. SHEET 5 OF 9

SUBJECT Spot Checks

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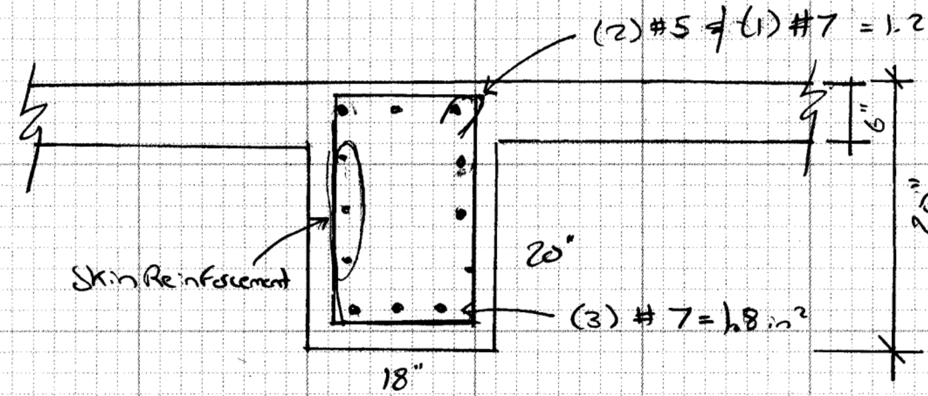
Spot Check Calculations Cont.

Typical Interior Beam Check – Level 3 : BT-6 (Designation on plan)



$b_{eff} = \begin{cases} \frac{21.33 \cdot 12}{4} = 64'' \\ 18 + (16 \cdot 6) = 114'' \\ 18 + \frac{1}{2}(76) = 94'' \end{cases}$

$b_{eff} = 64''$

Singly Reinforced T-Beam Analysis "+" ϕM_n

Assume $f_s > f_y$ & $a \le h_f$:

From $T = C$:

$d_t = 20 - 1.5 - 3/8 - \frac{7/8}{2} = 17.69''$

$A_s f_y = 0.85 f'_c a b$

$(1.8)(60) = 0.85(5)a(64) \rightarrow a = 0.397 < h_f \therefore OK$

$c = a/\beta_1 = 0.397/0.8 = 0.496$ Verify $f_s > f_y$: $\varepsilon_t = \frac{\varepsilon_u}{c}(d_t - c) =$

$\frac{0.003}{0.496}(17.6875 - 0.496) =$

$= 0.1042 > 0.005 \therefore OK$

Determine ϕM_n: $\phi = 0.9$

$\phi M_n = A_s f_y (d - a/2) = 0.9(1.8)(60)(17.6875 - \frac{0.496}{2})$

$\phi M_n = 1695.1''^k = \boxed{141.3'^k}$

Check $A_{s,min}$:

$A_{s,min} \ge \begin{cases} (3\sqrt{f'_c}/f_y) b_w d = (3\sqrt{5000}/60{,}000)(18)(17.6875) = 1.12\ in^2 \\ \frac{200}{f_y} b_w d = \frac{200}{60{,}000}(18)(17.6875) = 1.06\ in^2 \therefore OK \end{cases}$

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PROJECT NAME UNC-IRB
PROJECT NO.
SHEET 6 OF 9
SUBJECT Spot Checks
PREPARED BY DRH DATE 10/3/09 CHECKED BY DATE

" - " ϕM_n

Assume $f_s > f_y$

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{1.22(60)}{0.85(5)(18)} = 0.957"$$

$$C = a/\beta_1 = 0.957/0.8 = 1.2"$$

Check $\varepsilon_s > \varepsilon_y$ $\quad \varepsilon_s = \frac{\varepsilon_u}{c}(d - c) =$

$$= \frac{0.003}{1.2}(17.6875 - 1.2)$$

$$= 0.0412 > \varepsilon_y = 0.00207$$

Since $\varepsilon_t > 0.005$ $\phi = 0.9$

$$\phi M_n = A_s f_y (d - a/2) = 0.9(1.22)(60)(17.6875 - 1.2/2)$$

$$\phi M_n = 1125.7\text{"K} = \boxed{93.8\text{'K}}$$

Check $A_{s,min}$:

$$A_{s,min} \geq \begin{cases} \frac{3\sqrt{5000}}{60000}(18)(17.6875) = 1.12 \text{ in}^2 \\ 200(18)(17.6875)/60{,}000 = 1.06 \text{ in}^2 \end{cases} \quad \therefore \text{ OK}$$

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PROJECT NAME UNC-IRB
PROJECT NO. SHEET 8 OF 9
SUBJECT Spot Checks
PREPARED BY DRH DATE 10/3/09 CHECKED BY DATE

Determination of Vu

$V_u = 1.0\, w_u l_n / 2$

ASCE 7-05 → $w_u = 1.907$ K/'

Mulkey → $w_u = 2.4$ K/'

$V_u = 1.0\,(1.9)(19)/2$

$V_u = \underline{18.05^K}$ (ASCE)

$V_u = 1.0\,(2.4)(19)/2$

$V_u = \underline{22.8^K}$ (Mulkey)

Check ϕV_n - Ch 11 ACI 318-08

#3 Stirrups - 10 @ 8", R @ 16" Ea End

$A_v = 2(0.11) = 0.22\ in^2$

$V_c = 2\sqrt{f'c}\, b_w d = 2\sqrt{5000}\,(18)(17.6875) = 45.02$ K

$V_s = A_v \cdot f_{yt} \cdot d/s = (0.22)(60)(17.6875/16) = 23.35^K$

$\phi V_n = \phi V_c + \phi V_s = 0.75(45.02 + 23.35)$

$\phi V_n = 51.3^K$

Check min thickness for Deflection Control - Table 9.5(a)

Beams or Ribbed one way slabs $\}$ $l/21 = 19'(12)/21 = 10.9" < 20"$ ∴ ok

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PROJECT NAME UNC-IRB
PROJECT NO. SHEET 9 OF 9
SUBJECT Spot Checks
PREPARED BY DRH DATE 10/3/09 CHECKED BY DATE

Spot Check Calculation Conclusions:

Typical Interior Beam – After performing analysis on a typical floor beam, Mulkey's design appears to be pretty acute. For flexure, $\phi M_n^+ = 141.3'k > M_u^+ = 43'k$. And, $\phi M_n^- = 93.8'k > M_u^- = 62.5'k$. While these numbers may appear to have a large gap, in actuality, mulkey designed for $M_u^+ = 54.2'k$ & $M_u^- = 78.92'k$.

Shear was also checked, $\phi V_n = 51.3^k > 22.8^k$ ∴ ok

Typical Interior Column – It seems that the typical interior column at level 3 is in adequate in a combined max loading situation based on my calculations according to ASCE 7-05. It is noted though my seismic base shear is roughly 700^k higher than Mulkey's design, which would certainly change the moment on the column. Axial load alone, the column is sufficient.

ME-02