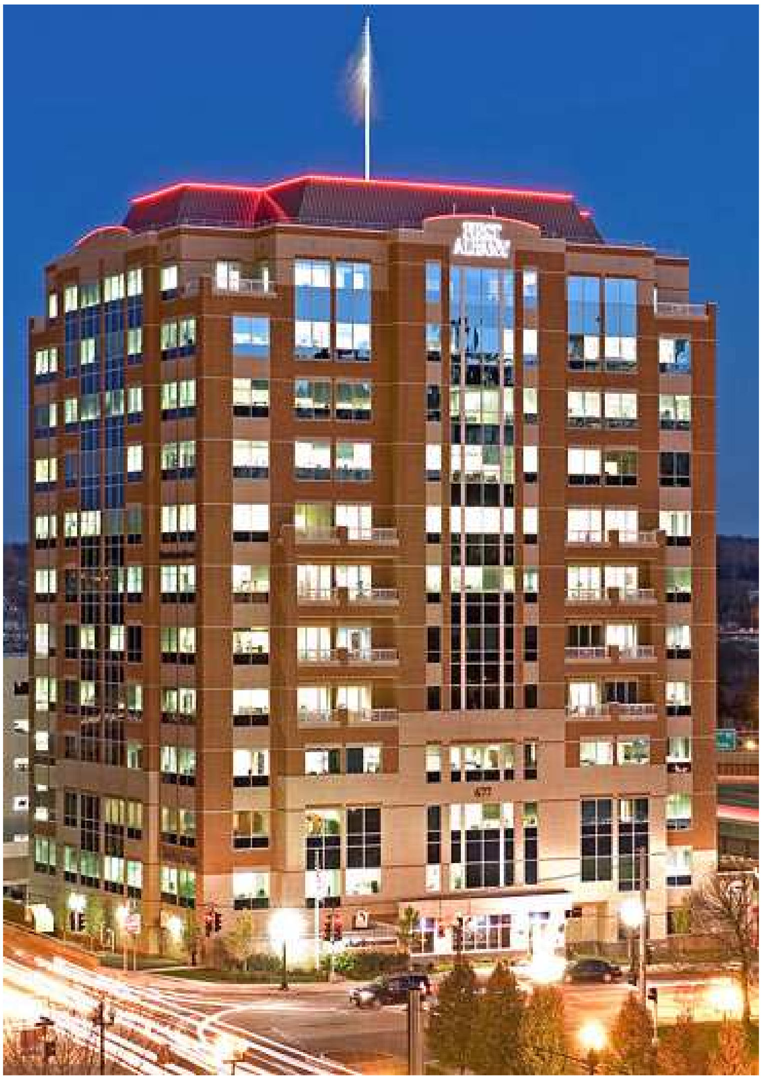
The First Albany Building

677 Broadway
Albany, NY



New Structural Systems Design

New Location: Charleston, SC

Gerald Craig

Architectural Engineering

Structural option

Consultant:
Dr. Boothby

April 7, 2010

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Section 1 – Executive Summary

The First Albany Building is a 12 story, 180,000 square feet structure designed for mixed-use office space and condominiums. The building's footprint is approximately 115' x 137'. It is located in downtown Albany, NY. The foundation is a concrete slab on grade over a network of reinforced concrete grade-beams and pile caps. The first floor is at grade and the building has no basement. H-piles were driven to practical refusal to fully support the building. Gravity loads are resisted by a reinforced concrete slab supported by a grid of simply supported steel beams and girders. Partial composite beam and composite deck design was incorporated in to the building. The main lateral force resisting system is comprised of concentric steel braced frames. There are five braced frames, two in the East – West direction and three in the North – South Direction, all located in the core of the building. The braced frames each act as a vertical, cantilevered truss.

For educational purposes a new building site in Charleston, South Carolina was chosen for a visually identical building with a different structural system. The site was chosen because it poses more risk for significant seismic activity and severe winds from hurricanes. A new floor system was designed using full composite action (slab and beam) for the reason of reducing the weight of the floor system. A new main lateral force resisting system was designed using special reinforced concrete shear walls located around the core of the building. Even though this system adds considerable weight, it was chosen because it has a higher response modification factor for determining seismic loads (R = 6, verses 5 for composite steel and concrete concentrically braced steel frames). To further reduce design seismic forces and base shear, a dynamic analysis was performed (Modal Superposition).

From an architectural standpoint, the building is relatively unchanged. The only difference is a slight layout change to the core of the building having minimal affects on building traffic patterns. Elevator shafts where slightly shifted and re-oriented to obtain a symmetric layout.

Gravity and wind loads were determined from ASCE 7-05 chapters 4 and 6 respectively. Seismic loads were determined by a dynamic analysis and as outlined by chapter 12. To aid in the lateral and modal analyses, a three dimensional mathematical model was created and solved using ETABS. Seismic base shears were found to be slightly higher than the minimum allowed (85% of the seismic response coefficient (Cs) multiplied by the effective seismic weight).

Strength requirements of the lateral system were controlled by seismic forces in the upper stories and by wind forces in the lower stories. However, the factor that controlled the entire design was permissible story drift due to wind (L/400 or 0.25%). In the east-west direction there are four separate shear walls having thicknesses of 16 inches. In the north-south direction there are three shear walls each 20 inches thick each coupled with a single bay concrete moment frame.

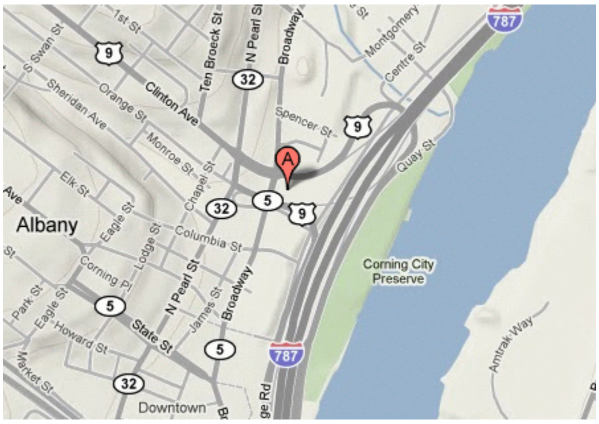
Two other areas of study were also conducted. The new structural system had little effect on construction costs and scheduling (breadth topic 1). The new location poses higher cooling demands so various other systems were looked at to reduce energy demands and consumption. A reflective roof surface and solar array would help reduce energy costs for the new building.

Section 2 - Introduction

Existing Building General Information:

The First Albany Building is a 12 story, 180,000 square feet structure designed for mixed-use office space and condominiums. The building is mostly being used as general office space at present. Floors 9-12 have access flooring providing essentially a plumbing chase if a leased space were to be used as a condominium. The building's footprint is approximately 115' x 137'. It is located along the Hudson River in downtown Albany, NY.

New York State



Albany, NY

The First Albany Building
677 Broadway, Albany, NY

Building Façade & Sustainability Features:

Facade	-	Classic Brick veneer over a gypsum board / sheet membrane vapor barrier / 2” Styrofoam™ brand Cavitymate™ insulated panel exterior wall system with standard insulated window units
Roof	-	Mechanically fastened single ply roof membrane over 4” rigid insulation on 1 ½” metal roof decking
Energy Conservation	-	Exceeds New York State energy code by 20%.

The building’s entrance is secured by an HID Card Access system and a full time security guard. Closed circuit TV cameras and recorders monitor both the interior and exterior of the building 24 hours per day 7 days per week. The building has an intercom system for off hour notification when the security guard is not present. Unique to professional office buildings in Downtown Albany are the building's 12 balconies as well as its heated sidewalks which surround the property. The building has redundant fiber networking service and some added features of this building include; redundant electric; high efficiency lighting with occupancy sensors; a Building Management System (BMS) to monitor all Building and Tenant HVAC equipment; an Uninterrupted Power Source System and an emergency generator. 677 Broadway is located just off I 787 (Clinton Avenue Exit) making the building ideal for clients and employees. It also yields optimum access to surrounding area businesses and restaurants and is part of the Empire Zone, lending its benefits to tenants through its Landlord. The building calls to an earlier era with its use of a glass facade, yet affords all of the efficiencies and energy savings of the present.

Construction Information:

Construction began on September 17, 2003 on what was previously a parking lot. BBL Construction Services served as the construction manager and general contractor. The site was big enough to accommodate the use of a regular mobile crane, thus eliminating the need for a stationary tower crane. There was a moderate amount of room directly behind the site for materials storage and staging. Still, careful planning and scheduling of deliveries was high priority so that the site wouldn't become cluttered, difficult, and dangerous. Delivery of materials and worker transportation was handled with ease as the First Albany Building is located just off from I-787 right at the end of an off-ramp in downtown Albany. With these key conditions, work advanced quickly and smoothly throughout the construction phase.

Existing Building Structural Information:

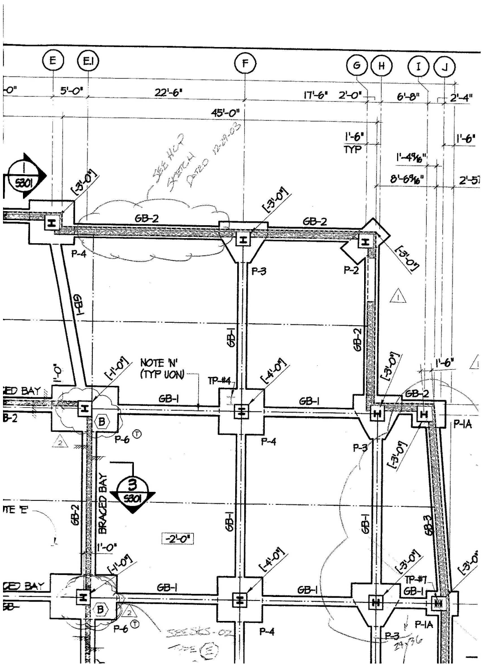


Figure 2.1 – Partial Foundation Plan

The foundation is comprised of a 6" thick concrete slab on grade over a network of reinforced concrete grade-beams and pile caps. The first floor is at grade and the building has no basement. H-piles were driven to practical refusal to fully support the building. Pile capacities are 120 tons, tested and verified on site during installation. A partial plan can be seen in Figure 2.1 (left).

Gravity loads are resisted by a 4.5" reinforced composite concrete deck supported by a grid of simply supported beams and girders. Partial composite beam design was also incorporated in to the building's structural system. Bays are typically 25'x25' with some variations. Sizes of floor members generally range between W12x14 and W18x60 shapes with a determined number of shear stud connectors on each member. Column lines transfer loads directly to the ground through pile caps and to the piles themselves. The piles were carefully laid out as to not cause eccentric forces in any one group of piles.

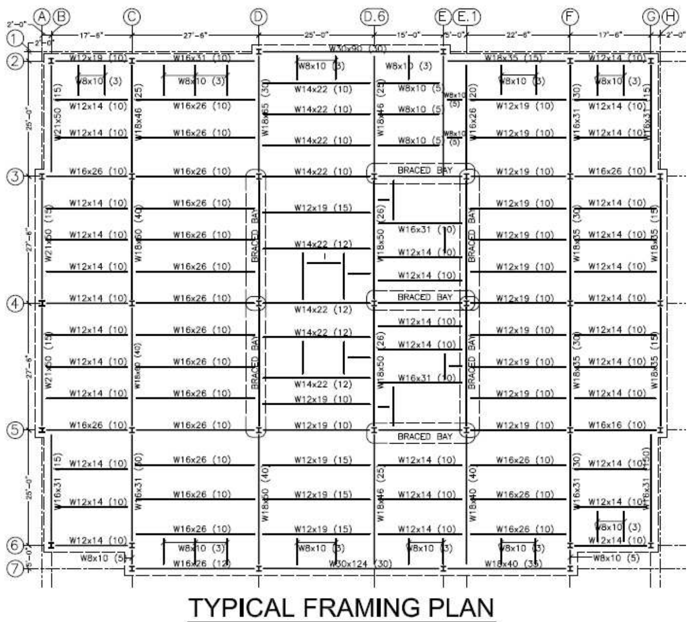


Figure 2.2 - Existing Framing Layout

Wind and seismic loads are resisted by sets of concentrically braced frames around the core of the building. Two frames are oriented in the East – West direction and three narrower frames are oriented in the North – South direction. Bracing patterns include "K", inverted "K", and standard diagonal. The braced frames each act as a vertical, cantilevered truss.

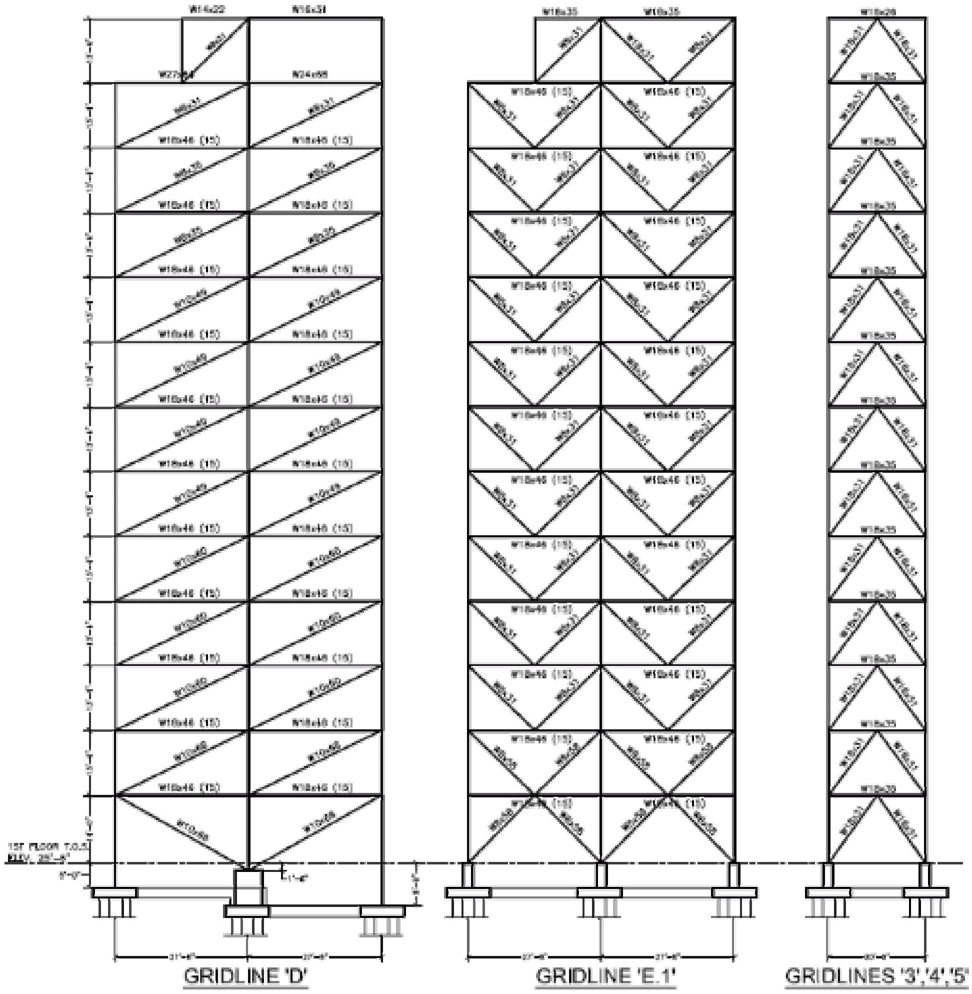


Figure 2.3 – Existing Braced Frame Elevations

Section 3 – Thesis Proposal

For educational purposes and personal experience a new structural system was chosen, analyzed, and designed for a site location that poses more risk for significant seismic activity and severe winds from hurricanes. A site in Charleston, South Carolina would fit these criteria. Even though there are building height limitations throughout the city, zoning variances have been considered and granted if the property is considered beneficial to the area (See Appendix E for example). This report focuses on the educational benefits to designing a building of such size, rather than on zoning limitations.

Structural Alterations:

Upstate New York is a region of low seismic activity, for which The First Albany Building performs adequately. However, if the owner decided to build a visually identical building in Charleston, South Carolina, significant modifications would be needed. Charleston is located in an area of high seismic risk. A light weight structural system with a higher response modification factor would be ideal to minimize base shears and design requirements. The goal is to reduce the effective seismic weight of the floor system as much as possible to allow greater latitude in choosing a lateral force resisting system.

An alternative structural system consisting of a full composite beam/composite deck floor design and special reinforced concrete shear walls was chosen for investigation. Choosing full composite action over partial composite action results in a lighter structural floor system. Reducing weight in one area would allow for increasing it in another. With that in mind a lateral structural system consisting predominantly of special reinforced concrete shear walls was selected. The driving force behind the selection was that a special reinforced concrete shear wall system has a response modification factor of 6 (ASCE 7-05 Table 12.2-1). The core of the building is altered slightly to reduce natural eccentricities created by lateral loads and eliminated the need for transfer girders at the perimeter. Elevators are repositioned as shown in the new core layout (figure 3.1).

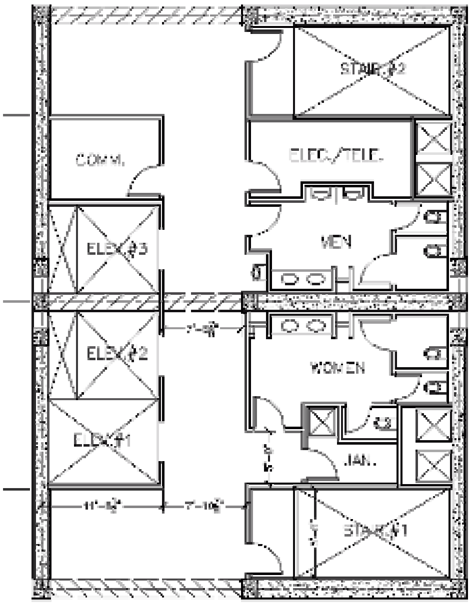


Figure 3.1 – New Core Layout

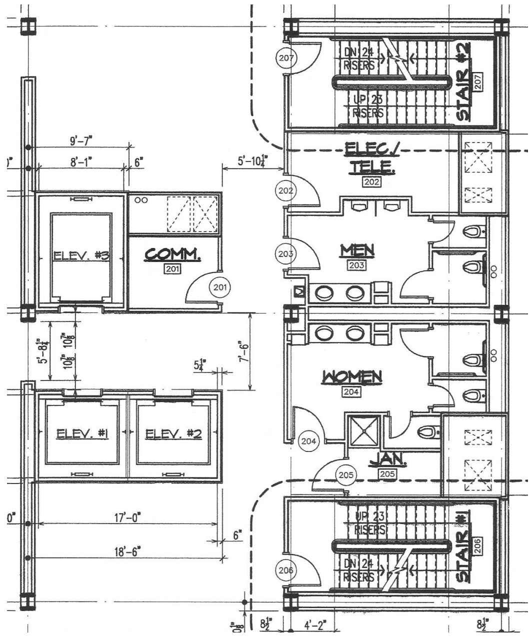


Figure 3.2 – Existing Core Layout

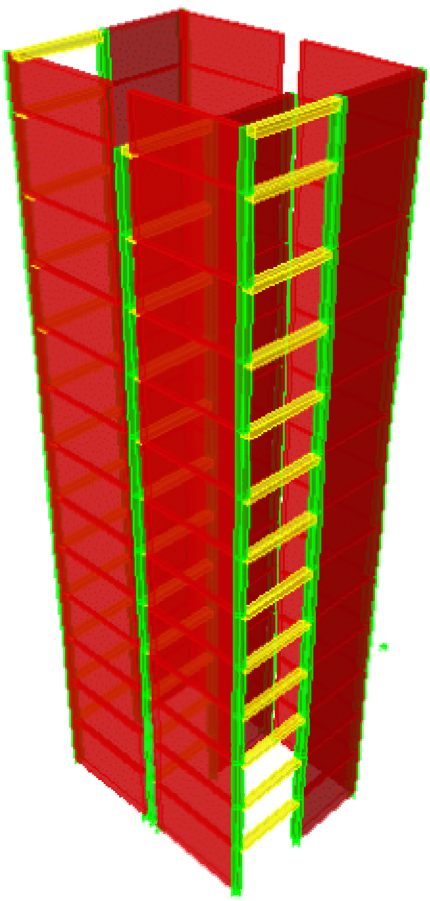


Figure 3.3 - New Core Structure

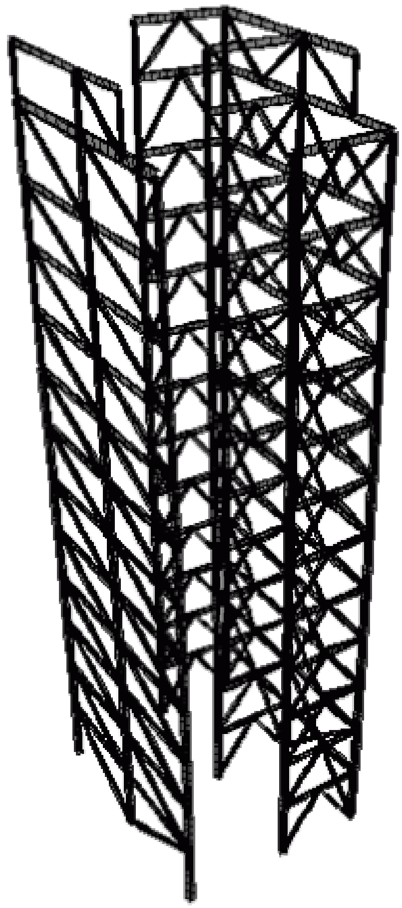


Figure 3.4 - Existing Core Structure

Solutions to Proposed Alterations:

To provide sufficient lateral stability, special reinforced concrete shear walls need to be sized and reinforced appropriately. Lateral and gravity loads have already been calculated based on ASCE 7-05 in Technical Report 1 (revised in Technical Report 3). The walls will be designed in accordance with Chapter 21 of ACI 318-08 (Section 21.9 - Special Structural Walls and Coupling Beams). The need for boundary elements will be determined from section 21.9.6. Transverse reinforcement (hoops and ties) will designed in accordance with Section 21.6. In special reinforced concrete walls, transverse reinforcing spacing is reduced to better confine the concrete and keep other reinforcing from buckling.

To obtain a light weight structural floor system, full composite action beam design and composite deck design will be taken advantage of. Findings presented in Technical Report 2 - Pro-Con Structural Study of Alternate Floor Systems show that weight savings can be attained through the use of full composite action verses partial composite action.

Using special reinforced concrete shear walls will increase the response modification factor from 5 to 6 (ASCE 7-05 Table 12.2-1). This combined with a lighter floor system will lessen the design base shear of a building in an area of high seismic risk. Dual systems have not been considered as options because they require that moment frames capable of resisting 25% of the seismic forces be incorporated into the design. Moment frames generally need larger sections to resist loads, making the structure heavier. A lighter floor system is desired.

Other Areas of Study:

Along with a study of this alternative system, two breadth studies shall be done in the construction management and mechanical options. The breadth in construction management will be an investigation of the scheduling and cost impact of switching to a full composite action beam with reinforced shear wall design. Changes in the geographic location will also be considered (weather, seasonal changes, local labor and material costs). The Mechanical breadth work study will be in energy conservation and energy cost considerations. Time of day usage (energy storage methods) and alternate energy sources will be explored to see where savings can be made. Other areas to check were building envelope parameters for a location in a warmer climate.

Section 4 – Building Codes & General Requirements

The First Albany Building was designed based on the New York State Building Code, and the allowable stress design method was used by the engineer. For the new structural systems the International Building Code is followed. Loads are determined from ASCE 7-05 and the strength design method is used. All factors and calculations are for Charleston, South Carolina.

Applicable Building Codes Used:

International Building Code 2006
ASCE 7-05
ACI 318-08 *Building Code Requirements for Structural Concrete*
AISC 13th Edition *Steel Construction Manual*

Load Combinations:

ASCE 7-05 2.3
• Case #1: 1.4D
• Case #2: 1.2D + 1.6L + 0.5S
• Case #3: 1.2D + 1.6S + 0.8W
• Case #4: 1.2D + 1.6W + 1.0L + 0.5S
• Case #5: 1.2D + 1.0E + 1.0L + 0.2S => $(1.2 + 0.2S_{DS})D + \Omega_O Q_E + L + 0.2S$
• Case #6: 0.9D + 1.6W + 1.6H
• Case #7: 0.9D + 1.0E + 1.6H => $(0.9 - 0.2S_{DS})D + \Omega_O Q_E + 1.6H$

For design of the new structural floor system, cases 1-3 were focused on. For lateral force resisting elements, 4-7 were checked.

Section 5 – Gravity Loads

Live Loads:

Type	Current Required Loading		
Office Space (2-8) Partition Allowance	50 +15	psf	ASCE 7-05 Table 4.1
Office Space (9-12) Access Flooring for Computer Use	100	psf	ASCE 7-05 Table 4.1
Office Space +File Storage	125	psf	ASCE 7-05 Table 4.1
Corridors (1st Floor)	100	psf	ASCE 7-05 Table 4.1
Lobbies & Corridors above 1st floor	80	psf	ASCE 7-05 Table 4.1
Stairways	100	psf	ASCE 7-05 Table 4.1
Balconies	100	psf	ASCE 7-05 Table 4.1
Roof	20	psf	ASCE 7-05 Table 4.1
Restaurants	100	psf	ASCE 7-05 Table 4.1
Roof Live	20	psf	ASCE 7-05 Table 4.1

Table 5.1

Snow Loads:

Load Calculations			
$P_f = 0.7*C_e*C_t*I*P_g$			ASCE 7-05 7.3 Eq 7-1
P_f min = P_g*I =5 psf			
C_e	=	1	ASCE 7-05 Table 7.2
C_t	=	1	ASCE 7-05 Table 7.3
I	=	1	ASCE 7-05 Table 7.4
P_g	=	5	ASCE 7-05 Fig. 7.1
P_f	=	5 psf	

Table 5.2

Live Loads Used:

Live loads used for design of the new structural floor system are 100 psf for all areas within the core of the buildings. Reasoning for this is that the entire area is treated as ‘stairway’ or ‘lobby area’. Live loads used the 2nd through 8th floors are 125 psf for file storage (indicated on plans, 2nd floor only) and 80 psf for all other areas. This loading equals or exceeds the required loads; 50+15 psf for office space and partitions and 80 psf for unknown locations of future corridors. Live loads used the 9th through 12th floors are 115 psf. This loading equals or exceeds the possible required loads; 100+15 psf for access flooring for computer use plus partitions and 80 psf for unknown locations of future corridors.

Live Load Reductions:

Reduction Factor (RF) = 0.25+ 15 / $\sqrt{(K_{LL}*A_{TRIB})}$
For structural members supporting 1 floor; RF $\geq$ 0.5
For structural members supporting 2 or more floors; RF $\geq$ 0.4

Element	K_{LL}[a]
Interior columns Exterior columns without cantilever slabs	4 4
Edge columns with cantilever slabs	3
Corner columns with cantilever slabs Edge beams without cantilever slabs Interior beams	2 2 2
All other members not identified including: Edge beams with cantilever slabs Cantilever beams One-way slabs Two-way slabs Members without provisions for continuous shear transfer normal to their span	1

Table 5.3
(ASCE 7-05 Table 4.2)

Dead Loads:

Types	
MEP (superimposed)	15 psf
Finishes (superimposed)	5 psf
Misc. (superimposed)	10 psf
Lightweight Concrete Slab	27.4 psf
Steel Deck	1.6 psf
Structural Concrete Walls	150 pcf
Structural Concrete Walls (LW)	115 pcf
Structural Steel	As calculated

Table 5.4

Full documentation for structural steel and reinforced concrete wall dead load calculations can be found in Appendix C along with a comprehensive total dead load determination.

Section 6 – Design Wind Loads as per ASCE 7-05

Wind loads were generated using section 6 of ASCE 7-05. All factors are dependent on building location and characteristics as well as experimental data.

Design Criteria:

Height (top of roof screen)	h			174.33’
Dimensions				137’x115’
Wind directionality factor	K_d	6.5.4		0.85
Importance Factor	I	6.5.5		1.0
Wind Exposure Category		6.5.6.3		B
Basic Wind Speed	V			90 MPH
Topographic Factor	K_{zt}	6.5.7		1.0
Gust Factor	Gf	6.5.8		As calculated
External Pressure Coeff.	C_{pf}	6.5.11.2	Windward	0.8
			Leeward	-0.5
			Sides	-0.7

height	V	Kd	I	Kz	Kzt	Gf E-W	Gf N-S	GCpi	Cp	qz	p E-W	p N-S
Wind	Basic Wind Vel. 6.5.4 Fig 6-1	Direct. Factor 6.5.4 Table 6-4	Import. Factor 6.5.5 Table 6-1	V press. exp. coeff. 6.5.6 Table 6-3	Topo Factor 6.5.7	Gust Factor 6.5.8	Gust Factor 6.5.8	Int. Press Coeff. 6.5.11.1 Fig 6-5	Ext Press Coeff. 6.5.11.2 Fig 6-6/8	Vel. Press 6.5.10		
0-15	140.00	0.85	1.00	0.57	1.00	0.8272	0.8306	-0.18	0.80	24.31	24.99	25.06
20	140.00	0.85	1.00	0.62	1.00	0.8272	0.8306	-0.18	0.80	26.44	26.40	26.48
25	140.00	0.85	1.00	0.66	1.00	0.8272	0.8306	-0.18	0.80	28.15	27.53	27.61
30	140.00	0.85	1.00	0.70	1.00	0.8272	0.8306	-0.18	0.80	29.85	28.66	28.74
40	140.00	0.85	1.00	0.76	1.00	0.8272	0.8306	-0.18	0.80	32.41	30.36	30.44
50	140.00	0.85	1.00	0.81	1.00	0.8272	0.8306	-0.18	0.80	34.55	31.77	31.86
60	140.00	0.85	1.00	0.85	1.00	0.8272	0.8306	-0.18	0.80	36.25	32.90	32.99
70	140.00	0.85	1.00	0.89	1.00	0.8272	0.8306	-0.18	0.80	37.96	34.03	34.13
80	140.00	0.85	1.00	0.93	1.00	0.8272	0.8306	-0.18	0.80	39.66	35.15	35.26
90	140.00	0.85	1.00	0.96	1.00	0.8272	0.8306	-0.18	0.80	40.94	36.00	36.11
100	140.00	0.85	1.00	0.99	1.00	0.8272	0.8306	-0.18	0.80	42.22	36.85	36.96
120	140.00	0.85	1.00	1.04	1.00	0.8272	0.8306	-0.18	0.80	44.36	38.26	38.38
140	140.00	0.85	1.00	1.09	1.00	0.8272	0.8306	-0.18	0.80	46.49	39.67	39.80
160	140.00	0.85	1.00	1.13	1.00	0.8272	0.8306	-0.18	0.80	48.19	40.80	40.93
174.33	140.00	0.85	1.00	1.16	1.00	0.8272	0.8306	-0.18	0.80	49.47	41.65	41.78
Lee										qh		
174.33	140.00	0.85	1.00	1.16	1.00	0.8272	0.8306	0.18	-0.50	49.47	-29.37	-29.45
Sides										qh		
174.33	140.00	0.85	1.00	1.16	1.00	0.8272	0.8306	0.18	-0.70	49.47	-37.55	-37.67

Table 6.1 – Wind Pressures

Gust Factors											
E-W						N-S					
V (mph)	140		α	0.25	Tab 6.2	V (mph)	140		α	0.25	Tab 6.2
B (ft)	137.0		b	0.45	Tab 6.2	B (ft)	115.0		b	0.45	Tab 6.2
L (ft)	115.0		Vz	122.82	Eq 6-14	L (ft)	137.0		Vz	122.82	Eq 6-14
h (ft)	171.6		N1	0.0014	Eq 6.12	h (ft)	171.6		N1	0.0006	Eq 6.12
z	103.0		Rn	0.0107	Eq 6-11	z	103.0		Rn	0.0047	Eq 6-11
l (ft)	320.0		η (Rh)	4.5464		l (ft)	320.0		η (Rh)	2.0074	
ε	0.33		η (RB)	3.6283		ε	0.33		η (RB)	1.3447	
Lz	467.6	Eq 6.7	η (RL)	3.0457		Lz	467.6	Eq 6.7	η (RL)	1.6020	
c	0.30		Rh	0.1958	Eq 6-13a	c	0.30		Rh	0.3763	Eq 6-13a
Iz	0.24	Eq 6.5	RB	0.2377	Eq 6-13a	Iz	0.24	Eq 6.5	RB	0.4859	Eq 6-13a
Q	0.82	Eq 6.6	RL	0.2746	Eq 6-13a	Q	0.82	Eq 6.6	RL	0.4373	Eq 6-13a
n1 (Hz)	0.70		β %	5		n1 (Hz)	0.31		β %	5	
gR	4.11	Eq 6.9	R	0.0081	Eq 6-10	gR	3.90	Eq 6.9	R	0.0113	Eq 6-10
gQ	3.40					gQ	3.40				
gv	3.40		G_f	0.8272	Eq 6-8	gv	3.40		G_f	0.8306	Eq 6-8

Table 6.2 – Gust Factors

Wind Forces Level	E-W Total Press. lb/ft²	N-S Total Press. lb/ft²	E-W P K	N-S P K	E-W Total Shear K	N-S Total Shear K	E-W Over Turn ft-K	N-S Over Turn ft-K
SW	70.90	70.93						
RF*	70.42	70.35	175.0	144.4	175.0	144.4	27478.9	22670.4
12	69.66	69.68	133.6	112.2	308.6	256.6	19729.6	16566.7
11	68.70	68.82	125.5	105.5	434.1	362.1	16773.3	14104.9
10	67.65	67.88	123.6	104.1	557.7	466.2	14869.1	12524.1
9	66.35	66.93	121.2	102.6	678.9	568.8	12967.9	10981.0
8	65.43	65.90	119.5	101.0	798.4	669.9	11194.4	9463.9
7	64.58	64.68	118.0	99.2	916.4	769.1	9475.8	7967.3
6	63.18	63.28	115.4	97.0	1031.8	866.1	7731.8	6500.2
5	61.64	61.73	112.6	94.6	1144.4	960.7	6041.9	5079.0
4	59.82	59.90	109.3	91.9	1253.6	1052.6	4406.8	3704.3
3	57.40	57.48	104.8	88.1	1358.5	1140.7	2830.1	2379.0
2	55.22	55.30	103.4	86.9	1461.9	1227.6	1412.7	1187.5
			Total 1461.9	Total 1227.6			Total 134912.4	Total 113128.3

Table 6.3 – Story Forces due to Wind

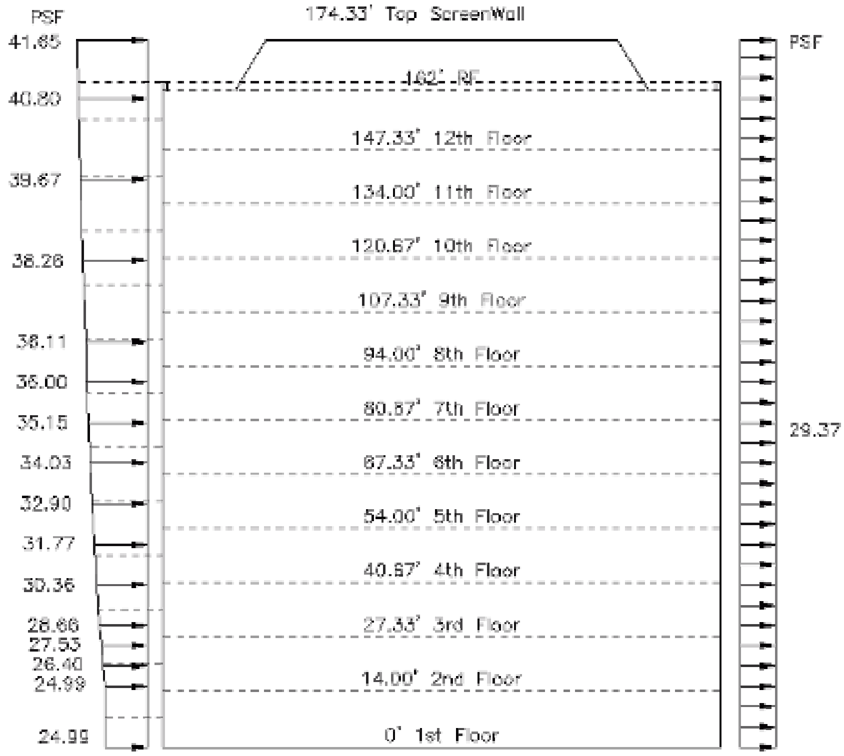


Figure 6.1 East-West Wind Pressures

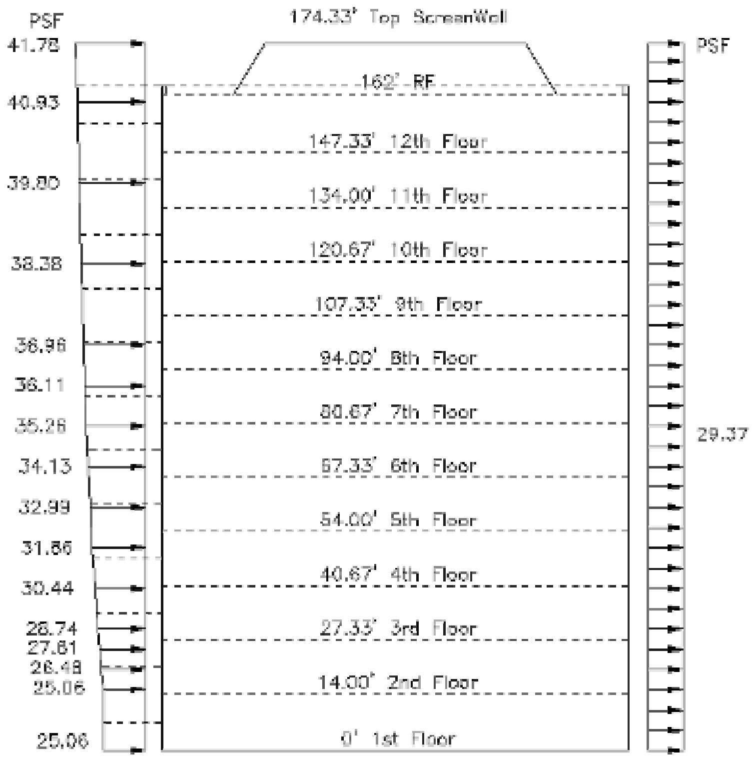


Figure 6.2 North-South Wind Pressures

Section 7 – Seismic Calculations for Charleston, SC

Conterminous 48 States
2003 NEHRP Seismic Design Provisions
Latitude = 32.795
Longitude = -79.943
Spectral Response Accelerations Ss and S1
Ss and S1 = Mapped Spectral Acceleration Values
Site Class D - $F_a = 1.0$,$F_v = 1.67$
Data are based on a 0.05000000074505806 deg grid spacing

Period (sec)	Sa (g)
0.2	1.487 (S_S, Site Class D)
1.0	0.365 (S_1, Site Class D)

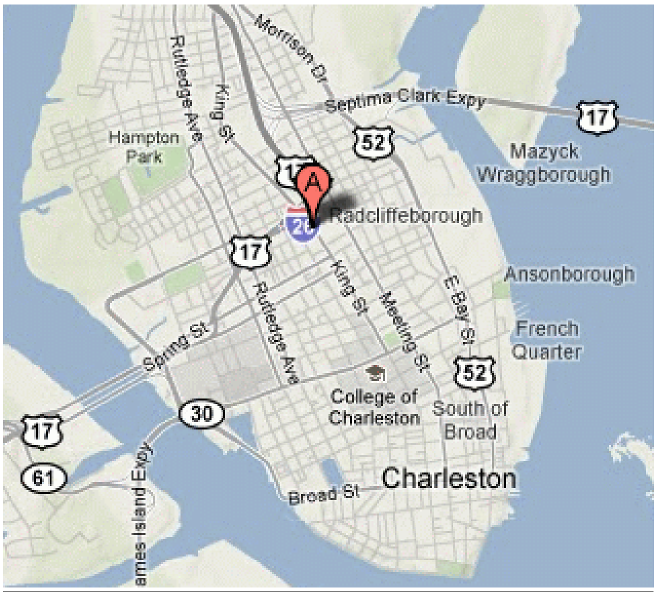
$S_{MS} = F_a \times S_S$ and $S_{M1} = F_v \times S_1$

0.2	1.487 (S_{MS}, Site Class D)
1.0	0.609 (S_{M1}, Site Class D)

$S_{DS} = 2/3 \times S_{MS}$ and $S_{D1} = 2/3 \times S_{M1}$

0.2	0.991 (S_{DS}, Site Class D)
1.0	0.406 (S_{D1}, Site Class D)

earthquake.usgs.gov/research/hazmaps/design



Occupancy Category	- II
Importance Factor	- 1.0
Response Mod. Factor	- 6 (Table 12.2-1)
T_L	- 8 (Figure 22-15)
Seismic Design Category	- D (Table 11.6.1,2 & based on various nearby site reports)
Effective Seismic Weight	- 18850 K

$T_s = S_{D1} / S_{DS} = 0.406 / 0.991 = 0.4097$
$T_a = C_t * h_n^{(x)} = 0.02\,(171.67)^{(0.75)} = 0.949$

$C_t = 0.02$ (Table 12.8-2)
$x = 0.75$
$h_n = 171.67'$
$C_u = 1.457$ (Tab 12.8-1)
$T_n = C_u T_a = 1.457*0.950 = 1.382$ sec

The approximate fundamental period, Ta, in s for masonry or concrete shear wall structures is permitted to be determined from Eq. 12.8-9 as follows:

$$T_a = \frac{0.0019\, h_n}{\sqrt{C_w}} \qquad (12.8\text{-}9)$$

where h_n is as defined in the preceding text and C_w is calculated from Eq. 12.8-10 as follows:

$$C_w = \frac{100}{A_B} \sum \left(\frac{h_n}{h_i}\right)^2 \frac{A_i}{1+0.83(h_i / D_i)^2} \qquad (12.8\text{-}10)$$

A_B = area of base of structure, ft²
A_i = web area of shear wall "*i*" in ft²
D_i = length of shear wall "*i*" in ft
h_i = height of shear wall "*i*" in ft
x = number of shear walls in the building effective in resisting lateral forces in the direction under consideration.

East-West (Walls D & E)
$C_w = 0.0205$
$T_a = 2.2766$
$C_uT_a = 1.457*2.2799 = 3.3218$
$T_c = 1.2702$ (calculated)

Base Shear:

$V = C_s * W_{TOTAL}$
= 0.0533 * 18850 = 1004.7 K
85% = 854.0 K

$$C_s = \min \begin{cases} \dfrac{S_{DS}}{(R/I)} = \dfrac{0.991}{(6/1)} = 0.1652 \\[2ex] \dfrac{S_{D1}}{T(R/I)} = \dfrac{0.406}{1.2702\,(6/1)} = 0.0533 \\[2ex] \dfrac{S_{D1}(T_L)}{T^2(R/I)} = \dfrac{0.406(8)}{1.2702^2(6/1)} = 0.3355 \end{cases}$$

$T_n < 3.5T_s$? (3.5*0.4097=1.434) No

$S_1 > 0.6g$? - No

North-South (Walls 3,4,5)
$C_w = 0.0126$
$T_a = 2.9305$
$C_uT_a = 1.457*2.9305 = 4.2700$
$T_c = 1.6199$ (calculated)

Base Shear:

$V = C_s * W_{TOTAL}$
= 0.0417 * 18850 = 786.0 K
85% = 668.1 K

$$C_s = \min \begin{cases} \dfrac{S_{DS}}{(R/I)} = \dfrac{0.991}{(6/1)} = 0.1652 \\[2ex] \dfrac{S_{D1}}{T(R/I)} = \dfrac{0.406}{1.6199\,(6/1)} = 0.0417 \\[2ex] \dfrac{S_{D1}(T_L)}{T^2(R/I)} = \dfrac{0.406\,(8)}{1.6199^2(6/1)} = 0.2063 \end{cases}$$

$T_n < 3.5T_s$? (3.5*0.4097=1.434) No

$S_1 > 0.6g$? - No

$T_n < 3.5T_s$? FALSE - Equivalent Lateral Force Method not permitted.
(ASCE 7-05 Table 12.6-1)

A Modal Response Spectrum Analysis is permitted.

Conterminous 48 States
2003 NEHRP Seismic Design Provisions
Latitude = 32.795
Longitude = -79.943

Site Modified Response Spectrum
$S_{MS} = F_aS_S$ and $S_{M1} = F_VS_1$
Site Class D - $F_a = 1.0$,$F_V = 1.67$

Period (sec)	Sa (g)	Sd (inches)
0.000	0.595	0.000
0.082	1.487	0.098
0.200	1.487	0.581
0.410	1.487	2.438
0.500	1.218	2.976
0.600	1.015	3.571
0.700	0.870	4.166
0.800	0.761	4.761
0.900	0.677	5.356
1.000	0.609	5.951
1.100	0.554	6.547
1.200	0.508	7.142
1.300	0.469	7.737
1.400	0.435	8.332
1.500	0.406	8.927
1.600	0.381	9.522
1.700	0.358	10.117
1.800	0.338	10.712
1.900	0.321	11.308
2.000	0.305	11.903

Design Response Spectrum
$S_{DS} = 2/3 \times S_{MS}$ and $S_{D1} = 2/3 \times S_{M1}$
Site Class D - $F_a = 1.0$,$F_V = 1.67$

Period (sec)	Sa (g)	Sd (inches)
0.000	0.397	0.000
0.082	0.991	0.065
0.200	0.991	0.387
0.410	0.991	1.625
0.500	0.812	1.984
0.600	0.677	2.381
0.700	0.580	2.777
0.800	0.508	3.174
0.900	0.451	3.571
1.000	0.406	3.968
1.100	0.369	4.364
1.200	0.338	4.761
1.300	0.312	5.158
1.400	0.290	5.555
1.500	0.271	5.951
1.600	0.254	6.348
1.700	0.239	6.745
1.800	0.226	7.142
1.900	0.214	7.538
2.000	0.203	7.935

12.2.5.4 Increased Building Height Limit for Steel Braced Frames and Special Reinforced Concrete Shear Walls.
The height limits in Table 12.2-1 are permitted to be increased from 160 ft (50 m) to 240 ft (75 m) for structures assigned to Seismic Design Categories D or E and from 100 ft (30 m) to 160 ft (50 m) for structures assigned to Seismic Design Category F that have steel braced frames or special reinforced concrete cast-in-place shear walls and that meet both of the following requirements:
1. The structure shall not have an extreme torsional irregularity as defined in Table 12.2-1 (horizontal structural irregularity Type 1b).
2. The braced frames or shear walls in any one plane shall resist no more than 60 percent of the total seismic forces in each direction, neglecting accidental torsional effects.

Section 8 – Material Specifications

Structural Steel:

Miscellaneous shapes, plates, bars	–	ASTM A36, Fy = 36 ksi
Structural Shapes, W8 and larger	–	ASTM A992
Anchor Bolts	–	ASTM A307

Cast-in-place Concrete:

Slab on Grade	–	3500 psi (28 day compressive strength)
Supported Floor Slabs	–	4000 psi, (lightweight, 115 pcf)
Grade Beams, Pile Caps, Foundation Walls	–	4000 psi
Shear Walls & Core Columns 2nd-4th	–	8000 psi
Shear Walls & Core Columns 5th-12th	–	5000 psi (lightweight, 115 pcf)
Foundation Piers	–	6000 psi
Reinforcing bars	–	ASTM A615, Grade 60, deformed
Welded Reinforcing bars	–	ASTM A706, Grade 60
Welded Wire Fabric	–	ASTM A185 (Sheet type only)

Steel Deck:

Roof Deck	–	1 ½” x 22 Gage Type B Rib Deck
Floor Deck	–	1 ½” x 22 Gage Composite Floor Deck (B-LOK)

Section 9 – Structural Floor System Design

Composite Deck/Slab Design:

Utilizing the design procedure as prescribed by the United Steel Deck design manual and catalog a deck/slab section with the following properties was chosen.

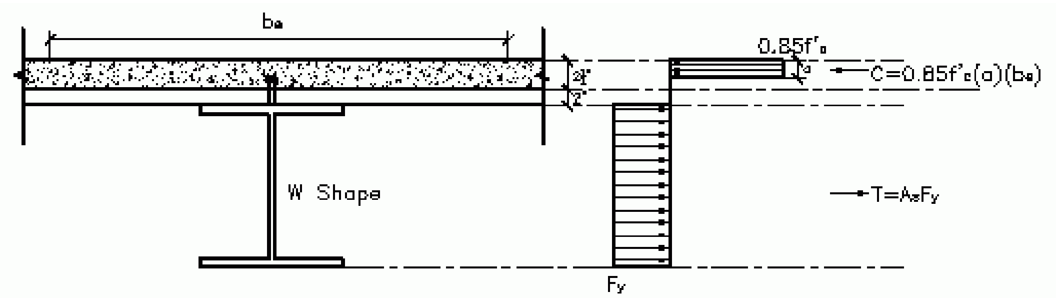
B-LOK 1.5"x 6" DECK
f'c = 4 ksi (lightweight concrete)
Total Slab Thickness = 4"
22 gage
Weight = 1.6 psf
Composite Weight = 29 psf
ΦV_{nt} = 2980 #
ΦM_{no} = 25.66 in-K (no studs present, conservative)
Maximum Un-shored Span = 6.91' (82.92")
A_{WWF} = 0.023 in² per ft

Dead Load = 60.6 plf (4.92 pli)
Maximum Live Load = 125 plf (10.42 pli)
Maximum Span = 6'-10½" (82.5")
Maximum Moment (Mu) = 1.2(4.92*82.5² / 8)+1.6(10.42*82.5² / 8) = 19.2 in-K
Maximum Shear (Vu) = 1.2(4.92*82.5 / 2)+1.6(10.42*82.5 / 2) = 930.9 #

Δ_{LL} = 0.013W_L(l^4)/E*I_{AV} = 0.013(125)(6.91^4)(1728)/(29500000*3.1) = 0.07"
(L/1179, OK)

Full Composite Beam Action:

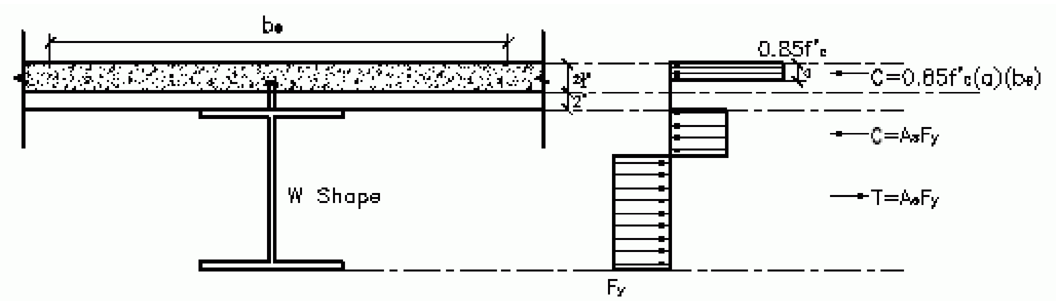
This system utilizes 'full composite action' rather than 'partial composite action'. This allows the concrete floor slab to play a more significant role in the Compression = Tension equation for beam design. All of the compressive forces are taken by the concrete slab while all the tensile forces are carried by the structural steel shape. Rather than the number of shear stud connectors controlling the strength, the number of shear stud connectors is determined by material properties and geometries.



Configuration Stress Force

Figure 2.1 – Full Composite Beam Action

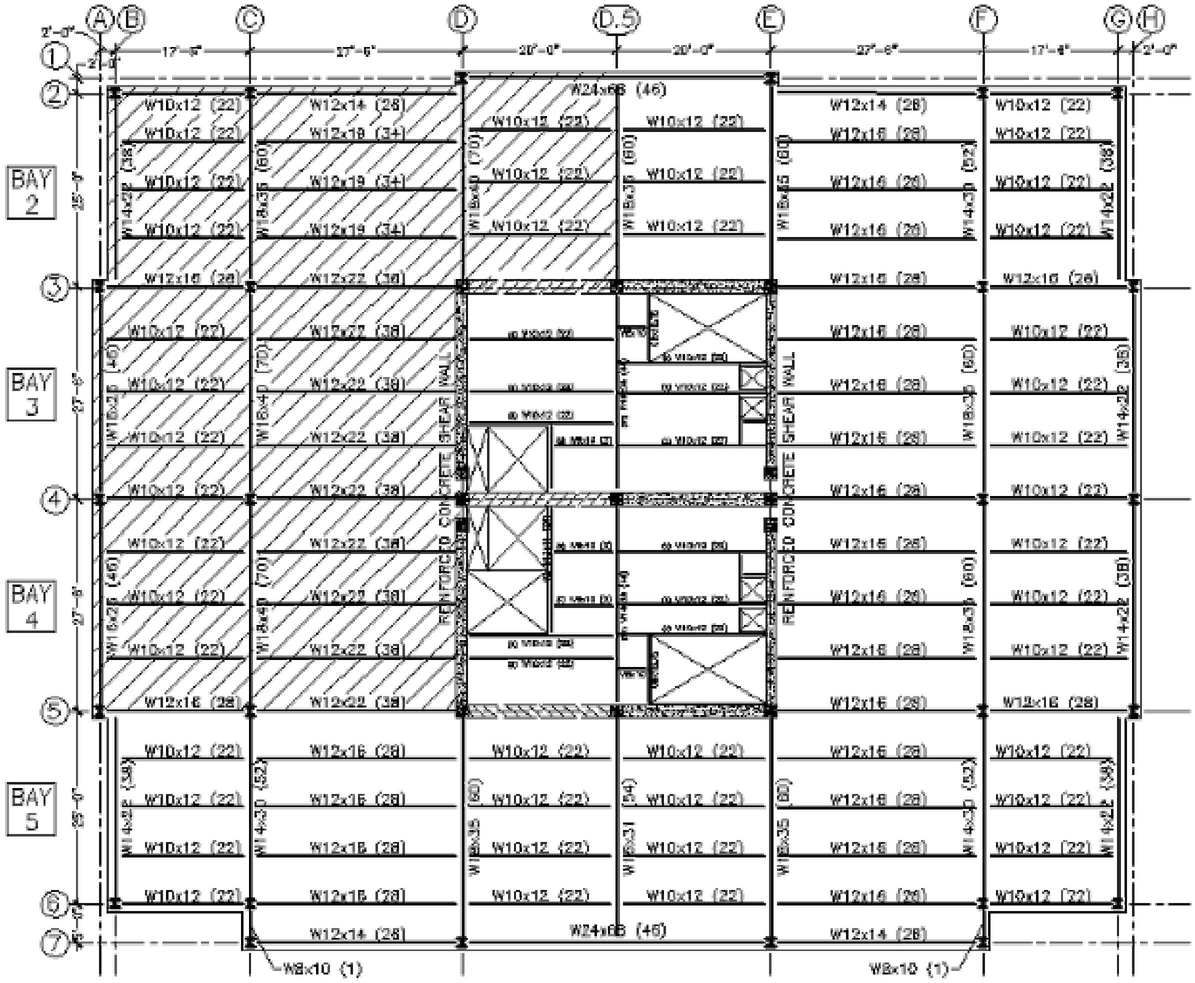
Partial composite action is where when the shear stud connectors only transfer a portion of the compressive forces from the structural shape to the concrete slab. A quick spot check easily determines that full composite action wasn't taken advantage of. The number, and therefore capacity of shear stud connectors to transfer stresses from the steel beam to the concrete slab are less than full potential shear stress between them. Basically, it doesn't take full advantage of the concrete's ability to take stresses. (0.85f'c(a)(be) > Σ Qn)



Configuration Stress Force

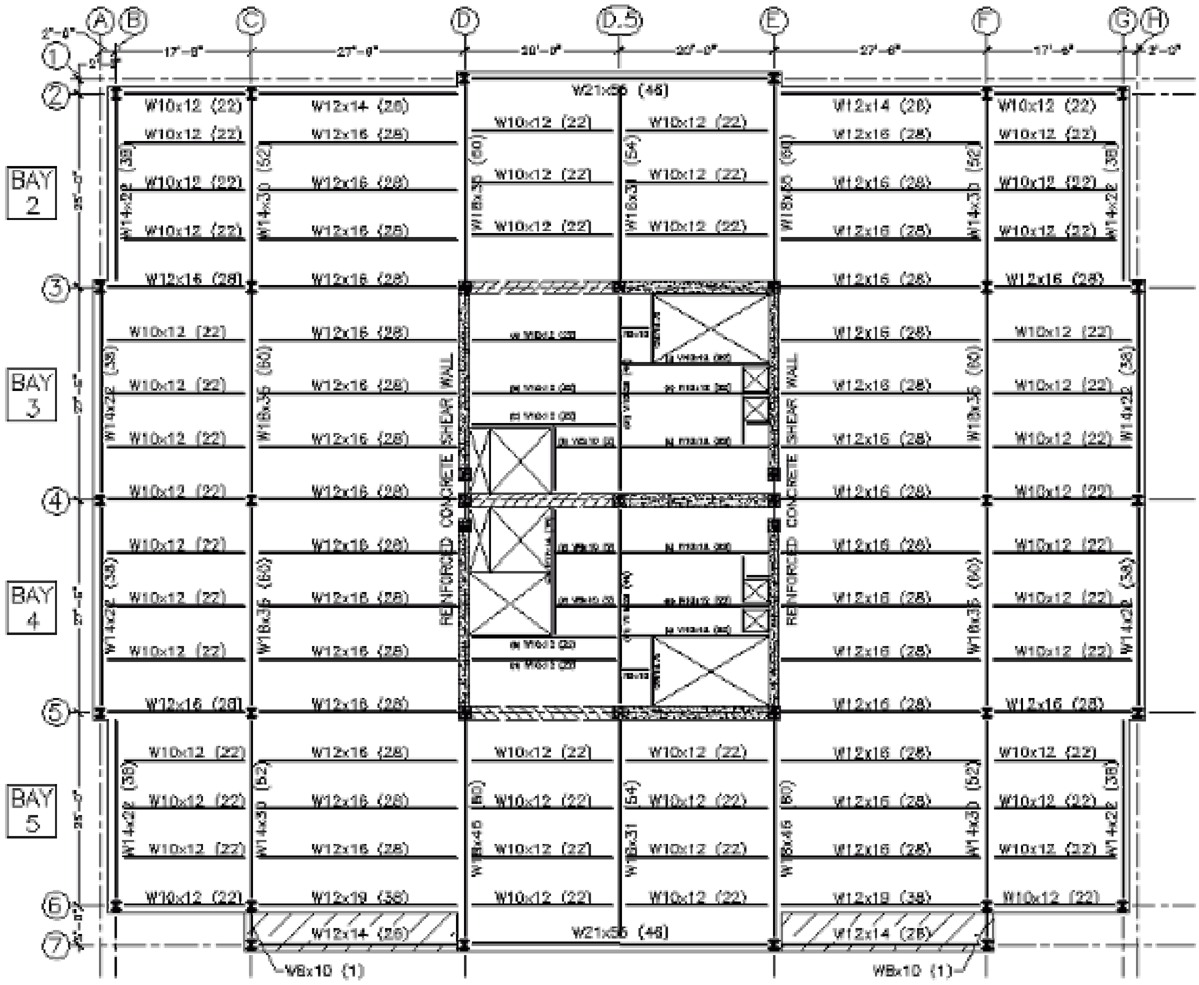
Figure 2.2 – Partial Composite Beam Action

Full documentation for calculations pertaining to individual structural members can be found in Appendix A. All calculations were completed with the use of custom made spread-sheets.

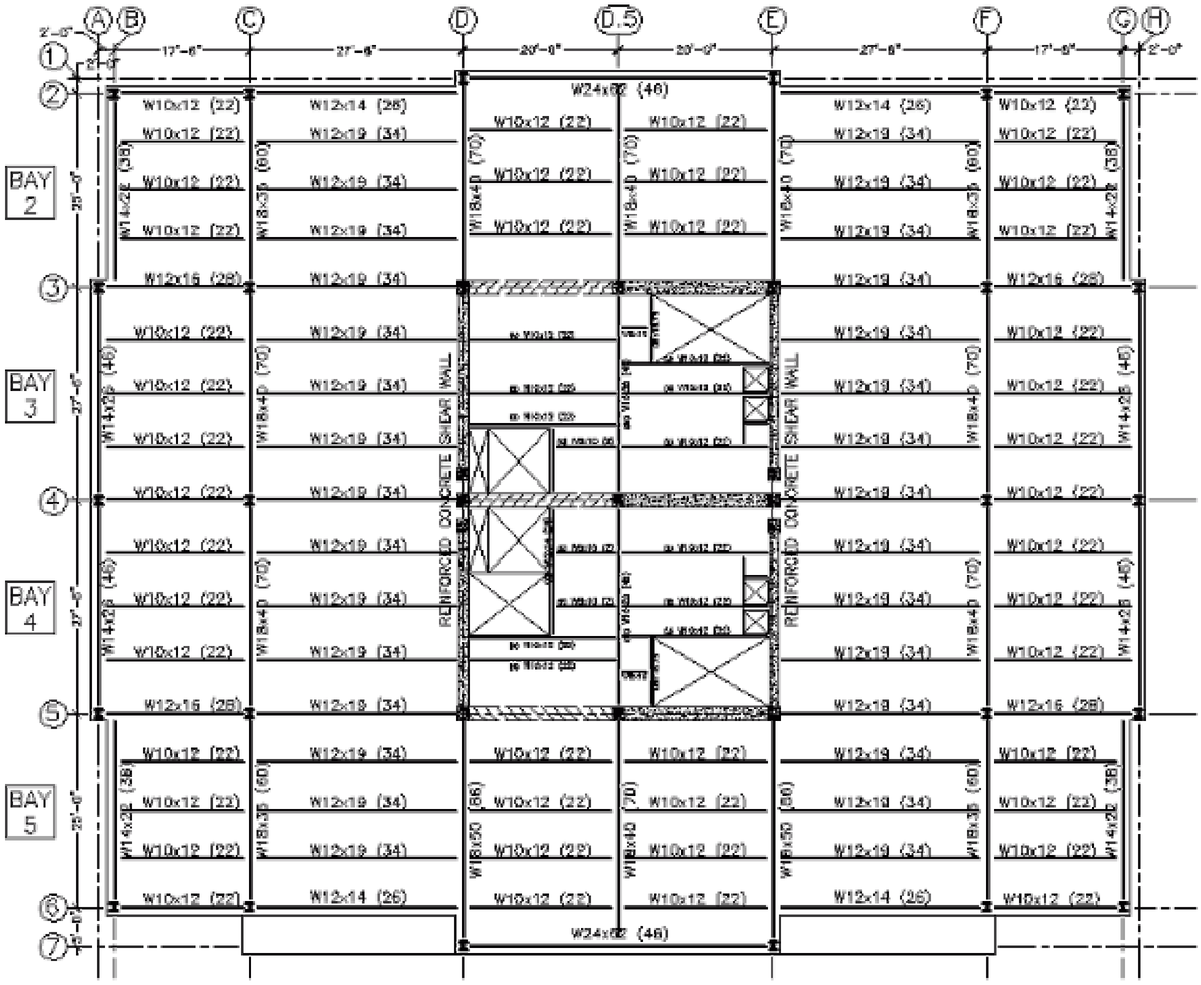


2nd FLOOR FRAMING PLAN

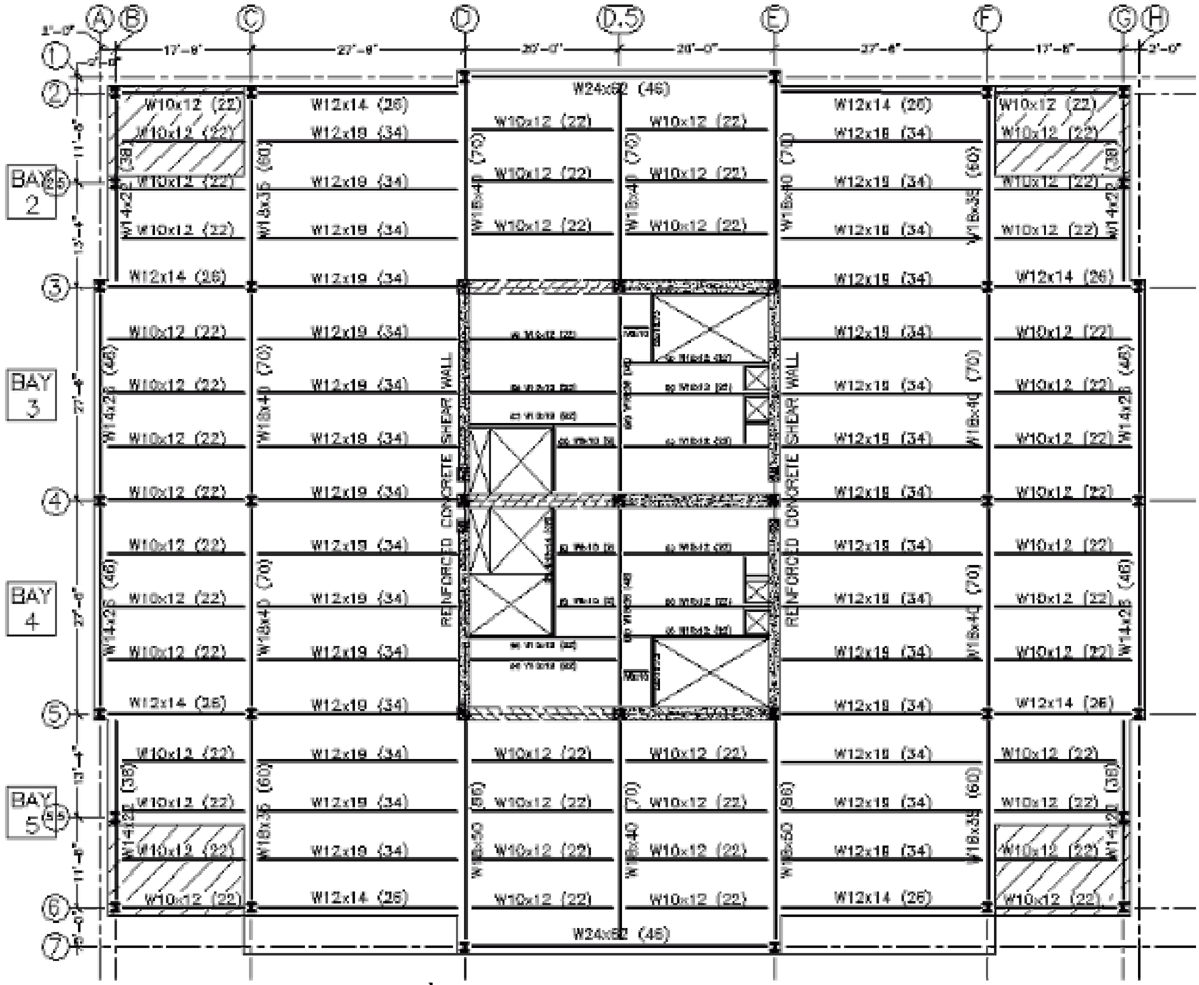
3^{rd} – 5^{th} FLOOR FRAMING PLAN



<u>**6th – 8th FLOOR FRAMING PLAN**</u>



9^{th} – 11^{th} FLOOR FRAMING PLAN



12th FLOOR FRAMING PLAN

Section 10 – Gravity Column Design

Column Table:

Column Line

Level	A3	A4	A5	B2	B6	C2	C3	C4	C5	C6	C7	D1	D7	E1	E7	F2	F3	F4	F5	F6	F7	G2	G6	H3	H4	H5	B2.5	B5.5	G2.5	G5.5	
12																											W6x15	W6x15	W6x15	W6x15	
11–12	W6x20	W6x20	W6x20	W6x15	W6x15	W6x20	W6x25	W6x25	W6x25	W6x20		W8x31	W8x31	W8x31	W8x31	W6x20	W6x25	W6x25	W6x25	W6x25		W6x15	W6x15	W6x20	W6x20	W6x20					Splice
9–11	W8x31	W8x31	W8x31	W6x25	W6x25	W8x31	W8x40	W8x40	W8x40	W8x31		W10x45	W10x45	W10x45	W10x45	W8x31	W8x40	W8x40	W8x40	W8x31		W6x25	W6x25	W8x31	W8x31	W8x31					Splice
7–9	W10x45	W10x45	W10x45	W8x31	W8x31	W10x45	W10x49	W10x54	W10x49	W10x45	W6x15	W10x54	W12x58	W10x54	W12x58	W10x45	W10x49	W10x54	W10x49	W10x45	W6x15	W8x31	W8x31	W10x45	W10x45	W10x45					Splice
5–7	W10x49	W10x49	W10x49	W10x33	W10x33	W10x49	W12x65	W12x65	W12x65	W10x49	W6x15	W12x65	W12x72	W12x65	W12x72	W10x49	W12x65	W12x65	W12x65	W10x49	W6x15	W10x33	W10x33	W10x49	W10x49	W10x49					Splice
3–5	W10x54	W12x58	W10x54	W10x39	W10x39	W12x58	W12x79	W12x79	W12x79	W12x58	W6x20	W12x79	W12x87	W12x79	W12x87	W12x58	W12x79	W12x79	W12x79	W12x58	W6x20	W10x39	W10x39	W10x54	W12x58	W10x54					Splice
1–3	W12x65	W12x65	W12x65	W10x45	W10x45	W12x65	W12x96	W12x96	W12x96	W12x65	W8x31	W12x96	W12x106	W12x96	W12x106	W12x65	W12x96	W12x96	W12x96	W12x65	W8x31	W10x45	W10x45	W12x65	W12x65	W12x65					

Table 9.1 Column Table

Axial Capacities of Wide Flange Shapes:

$\Phi = 0.90$
$\Phi Pn = \Phi(Ag)(Fcr)$ (E2-1)

For $\lambda c < 1.5$ $Fcr = (0.658^{\wedge}(\lambda c^2))Fy$ (E2-2)
For $\lambda c > 1.5$ $Fcr = (0.877/\lambda c^2)Fy$ (E2-3)
$\lambda c = (KL / r\pi)\sqrt{(Fy / E)}$ (E2-4)

K	L	Shape				Ag	r_y	Fy	E	λc	Eq.	Fcr	ΦPn	Kl/r <200
	ft					in²	in	ksi	ksi			ksi	K	
1.00	14.67	W	6	x	15	4.43	1.45	50	29000	1.60	E2-3	17.04	67.92	121.38
1.00	13.33	W	6	x	15	4.43	1.45	50	29000	1.46	E2-2	20.53	81.84	110.34
1.00	14.67	W	6	x	20	5.87	1.50	50	29000	1.55	E2-3	18.23	96.32	117.33
1.00	13.33	W	6	x	20	5.87	1.50	50	29000	1.41	E2-2	21.76	114.96	106.67
1.00	14.67	W	6	x	25	7.34	1.52	50	29000	1.53	E2-3	18.72	123.67	115.79
1.00	13.33	W	6	x	25	7.34	1.52	50	29000	1.39	E2-2	22.24	146.91	105.26
1.00	13.33	W	8	x	35	10.30	2.03	50	29000	1.04	E2-2	31.75	294.35	78.80
1.00	13.33	W	8	x	40	11.70	2.04	50	29000	1.04	E2-2	31.89	335.85	78.41
1.00	13.33	W	10	x	33	9.71	1.94	50	29000	1.09	E2-2	30.41	265.72	82.47
1.00	13.33	W	10	x	39	11.50	1.98	50	29000	1.07	E2-2	31.02	321.11	80.79
1.00	13.33	W	10	x	45	13.30	2.01	50	29000	1.05	E2-2	31.46	376.57	79.60
1.00	13.33	W	10	x	49	14.40	2.54	50	29000	0.83	E2-2	37.41	484.87	62.98
1.00	13.33	W	10	x	54	15.80	2.56	50	29000	0.83	E2-2	37.58	534.42	62.48
1.00	13.33	W	10	x	60	17.60	2.57	50	29000	0.82	E2-2	37.67	596.63	62.24
1.00	13.33	W	12	x	45	13.10	1.95	50	29000	1.08	E2-2	30.57	360.41	82.03
1.00	13.33	W	12	x	58	17.00	2.51	50	29000	0.84	E2-2	37.15	568.44	63.73
1.00	13.33	W	12	x	65	19.10	3.02	50	29000	0.70	E2-2	40.73	700.09	52.97
1.00	13.33	W	12	x	79	23.20	3.05	50	29000	0.69	E2-2	40.89	853.79	52.45
1.00	13.33	W	12	x	87	25.60	3.07	50	29000	0.69	E2-2	41.00	944.58	52.10
1.00	13.33	W	12	x	96	28.20	3.09	50	29000	0.68	E2-2	41.10	1043.18	51.77
1.00	13.33	W	12	x	106	31.20	3.11	50	29000	0.68	E2-2	41.21	1157.06	51.43

Table 9.2 Axial Load Capacity Worksheet

The equations used above are from the AISC LRFD Manual for Steel Construction (2nd edition). With some manipulation it can be shown that they are equivalent to equations provided in the most recent edition of the AISC manual. For design purposes the gravity columns are considered with pin-pin end conditions and buckling about the weak axis (Y-Y) controls strength. Full documentation for load and column requirements can be found in Appendix C. All calculations were completed with the use of custom made spread-sheets.

Section 11 – Modal Response Spectrum Analysis (Dynamic Analysis)

From the seismic design criteria determined from ASCE 7-05 Section 12, it was determined that a dynamic analysis was not only preferred (to more accurately find story forces and base shear), it was necessary.

$T_s = S_{D1} / S_{DS} = 0.406 / 0.991 = 0.4097$
$T_c < 3.5T_s$? (3.5*0.4097=1.434)
$T_c = 1.6199$ (calculated north-south)
$T_c = 1.2702$ (calculated east-west)

From Table 12.6-1 it was found that the Equivalent Lateral Force Procedure is not permitted.

TABLE 12.6-1 PERMITTED ANALYTICAL PROCEDURES

Seismic Design Category	Structural Characteristics	Equivalent Lateral Force Analysis Section 12.8	Modal Response Spectrum Analysis Section 12.9	Seismic Response History Procedures Chapter 16
B, C	Occupancy Category I or II buildings of light-framed construction not exceeding 3 stories in height	P	P	P
	Other Occupancy Category I or II buildings not exceeding 2 stories in height	P	P	P
	All other structures	P	P	P
D, E, F	Occupancy Category I or II buildings of light-framed construction not exceeding 3 stories in height	P	P	P
	Other Occupancy Category I or II buildings not exceeding 2 stories in height	P	P	P
	Regular structures with $T < 3.5T_s$ and all structures of light frame construction	P	P	P
	Irregular structures with $T < 3.5T_s$ and having only horizontal irregularities Type 2, 3, 4, or 5 of Table 12.2-1 or vertical irregularities Type 4, 5a, or 5b of Table 12.3-1	P	P	P
	All other structures	NP	P	P

NOTE: P: Permitted; NP: Not Permitted

Mathematical Model – ETABS:

A three dimensional mathematical model was created using ETABS analysis software. For the sake of simplicity the model was used to analyze lateral forces only. After applying design wind loads to various trial models, a final model was selected for seismic analysis. The initial controlling factor used for trial model selection was drift limitations for wind loads.

In the model, cracked sections were considered as required by ASCE 7-05. To represent cracked sections the following section properties were modified.

Beams - $0.35I_g$
Column - $0.70I_g$
Shear Walls – $0.5f_{22}$

Modal Analysis Results:

Modal Participating Mass Ratios			
Mode	Period	UX	UY
1	1.946	0.146	3.615
2	1.618	64.574	0.016
3	1.268	0.001	60.672
4	0.356	0.131	0.748
5	0.296	20.265	0.009
6	0.226	0.001	20.214
7	0.146	0.034	0.267
8	0.119	7.005	0.002
9	0.092	0.001	6.040
10	0.087	0.013	1.008
11	0.069	3.432	0.001
12	0.063	0.006	0.008
	Totals -	95.609	92.600

Table 10.1

Totals satisfy the requirements of ASCE 7-05 12.9.1

12.9.1 Number of Modes. An analysis shall be conducted to determine the natural modes of vibration for the structure. The analysis shall include a sufficient number of modes to obtain a combined modal mass participation of at least 90 percent of the actual mass in each of the orthogonal horizontal directions of response considered by the model.

Applying the Design Spectrum determined from 2003 NEHRP Seismic Design Provisions (Section 7 of this report) gives the following results:

Base Shears		85% ELF
U1 (North-South)	767.66 K	668.1 K
U2 (East-West)	815.68 K	854.0 K

Table 10.2 Base Shear

ASCE 7-05 12.9.4 dictates that minimum values shall be equal to or greater than 85% of the base shear calculated by the Equivalent Lateral Force Procedure (ELF). A scale factor of 1.047 was added to the east-west directional analysis to meet the required minimum. After doing so several checks were made.

- Horizontal Building Irregularities
- Vertical Building Irregularities
- Redundancy factor (ρ)
- Amplification of Accidental Torsional Moment (A_x)

Torsional Irregularity Check:

	$U_{X\ or\ Y} / U_{AVE} < 1.2$?			
Story	CHU1E	CHU1NE	CHU2E	CHU2NE
12	1.03	1.03	1.09	1.08
11	1.03	1.03	1.09	1.09
10	1.03	1.03	1.09	1.09
9	1.02	1.03	1.09	1.09
8	1.02	1.03	1.09	1.09
7	1.02	1.03	1.09	1.09
6	1.03	1.03	1.09	1.09
5	1.03	1.03	1.09	1.09
4	1.03	1.03	1.10	1.10
3	1.03	1.03	1.11	1.11
2	1.03	1.03	1.13	1.13

Table10.3 Torsional Irregularity Check

When comparing average drifts to extreme drifts at the edges of the building it is found that the structure doesn't exhibit any torsional irregularity as defined in Table 12.3-1.

Full documentation of check can be found in Appendix D

No other horizontal or vertical irregularities are present in the structure.

Redundancy Factor:

For structures assigned to Seismic Design Category D, E, or F, ρ shall equal 1.3 unless one of the following two conditions is met, whereby ρ is permitted to be taken as 1.0:

a. Each story resisting more than 35 percent of the base shear in the direction of interest shall comply with Table 12.3-3.

b. Structures that are regular in plan at all levels provided that the seismic force–resisting systems consist of at least two bays of seismic force–resisting perimeter framing on each side of the structure in each orthogonal direction at each story resisting more than 35 percent of the base shear. The number of bays for a shear wall shall be calculated as the length of shear wall divided by the story height or two times the length of shear wall divided by the story height for lightframed construction.

When one of the shear walls is removed (in either direction) the structure suffers from extreme torsional irregularity. The value of ρ shall be taken as 1.3.

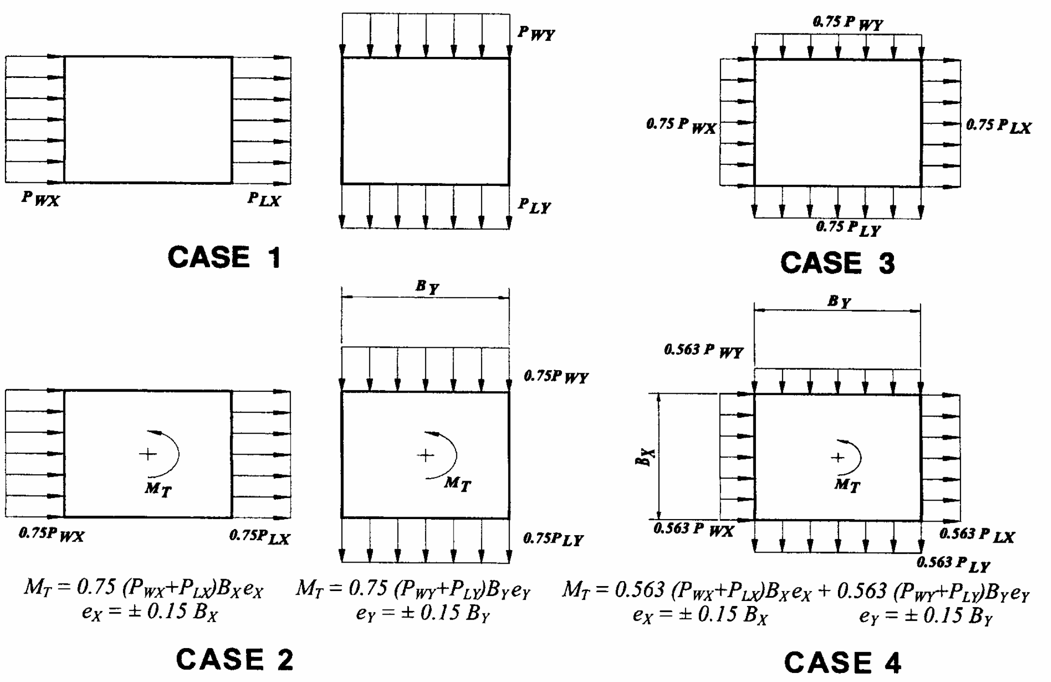
Amplification of Accidental Torsional Moment (A_x):

The structure does not display a type 1a or 1b torsional irregularity. An amplification factor need not be applied.

Section 12 – Lateral Analysis Results (Wind, Seismic, Gravity)

Direction and Combinations of Wind Loading:

Main Wind Force Resisting System–Method 2		All Heights
Figure 6-9	Design Wind Load Cases	



Case 1. Full design wind pressure acting on the projected area perpendicular to each principal axis of the structure, considered separately along each principal axis.

Case 2. Three quarters of the design wind pressure acting on the projected area perpendicular to each principal axis of the structure in conjunction with a torsional moment as shown, considered separately for each principal axis.

Case 3. Wind loading as defined in Case 1, but considered to act simultaneously at 75% of the specified value.

Case 4. Wind loading as defined in Case 2, but considered to act simultaneously at 75% of the specified value.

Notes:

1. Design wind pressures for windward and leeward faces shall be determined in accordance with the provisions of 6.5.12.2.1 and 6.5.12.2.3 as applicable for building of all heights.
2. Diagrams show plan views of building.
3. Notation:

   P_{WX}, P_{WY}: Windward face design pressure acting in the x, y principal axis, respectively.
   P_{LX}, P_{LY}: Leeward face design pressure acting in the x, y principal axis, respectively.
   $e\ (e_X, e_Y)$: Eccentricity for the x, y principal axis of the structure, respectively.
   M_T: Torsional moment per unit height acting about a vertical axis of the building.

Direction and Combination of Seismic Loading:

> **12.5.4 Seismic Design Categories D through F.** Structures assigned to Seismic Design Category D, E, or F shall, as a minimum, conform to the requirements of Section 12.5.3. In addition, any column or wall that forms part of two or more intersecting seismic force–resisting systems and is subjected to axial load due to seismic forces acting along either principal plan axis equaling or exceeding 20 percent of the axial design strength of the column or wall shall be designed for the most critical load effect due to application of seismic forces in any direction. Either of the procedures of Section 12.5.3 a or b are permitted to be used to satisfy this requirement.

Since the structure does not display any horizontal irregularities (specifically type 5), loads in each of the orthogonal directions are considered independently.

Maximum Forces Resulting from ASCE 7-05 Load Combinations:

Supported Story	Column Maximums			Beam Maximums		Walls 3,4,5 Maximums			Walls D,E Maximums		
	Axial	Shear	Moment	Shear	Moment	Axial	Shear	Moment	Axial	Shear	Moment
	K	K	ft-K	K	ft-K	K	K	ft-K	K	K	ft-K
PH	60.0	93.6	170.0	33.6	256.7	112.2	112.7	388.9	95.3	117.1	544.4
RF	138.4	79.2	108.3	36.0	276.5	438.7	230.3	1147.6	493.9	232.0	2341.3
12	227.1	74.2	119.7	35.3	273.0	742.3	342.3	2589.7	826.0	328.4	4680.5
11	355.6	71.9	116.9	35.4	273.3	1101.9	394.0	4181.8	1249.4	393.5	7497.5
10	484.2	76.7	119.1	35.0	269.2	1467.1	418.4	5739.6	1679.9	435.5	10422.1
9	608.0	76.6	119.7	34.3	262.8	1866.7	442.2	7200.1	2106.5	472.0	13434.0
8	726.8	75.2	119.0	33.2	252.8	2276.3	501.9	8590.4	2565.9	547.6	16618.0
7	846.5	72.4	116.4	31.8	238.8	2700.6	581.0	10020.7	3141.8	622.7	21914.5
6	977.4	67.9	111.8	29.8	220.0	3214.1	671.0	11664.6	3829.6	695.7	27913.8
5	1130.7	64.3	104.6	27.2	195.9	3813.8	778.1	13696.5	4573.0	762.6	34597.4
4	1314.2	65.4	95.3	24.0	165.8	4451.4	873.0	16302.1	5364.5	813.8	41897.5
3	1539.0	69.5	97.8	20.1	128.9	5113.6	939.2	19621.2	6187.8	832.2	49653.7
2	1855.7	75.2	103.7	16.4	85.7	5651.2	966.0	23320.8	6957.8	805.0	57470.5

Table 11.1 Maximum Forces

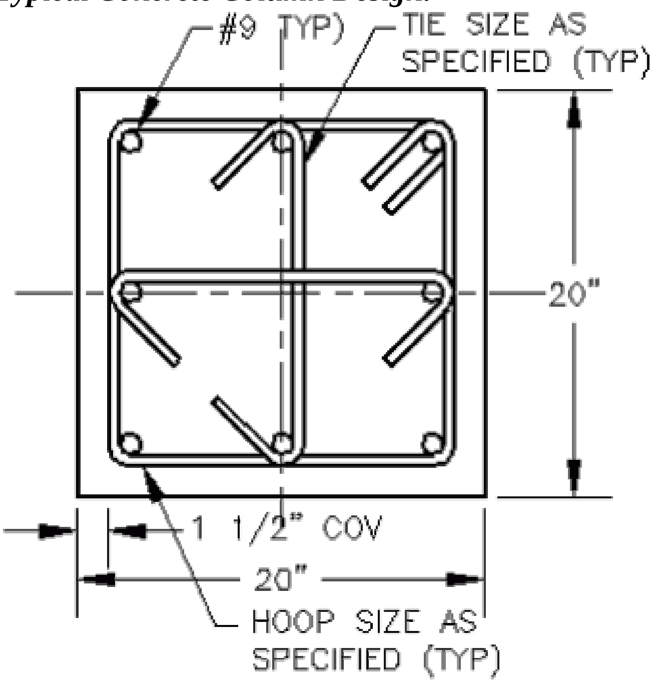
Full documentation for design forces resulting from the required load combinations can be found in Appendix F.

Section 13 – Lateral Structural System Design

Lightweight concrete was taken advantage of for floors 5 through the roof. All appropriate properties of lightweight concrete were considered (λ, E_c). Normal weight concrete was used in the lower floors due to limitations in ACI 3-18 21.1.4.3

> *Specified compressive strength of lightweight concrete, f'c, shall not exceed 5000 psi unless demonstrated by experimental evidence that structural members made with that lightweight concrete provide strength and toughness equal to or exceeding those of comparable members made with normal weight concrete of the same strength.*

Typical Concrete Column Design:



As per ACI 3-18 Sections 21.6.4 & 21.6.5, transverse reinforcement (hoops) must be spaced at maximum of 3" for a distance greater than or equal to 1/6th of the clearspan from each joint face, and at 6" (maximum) along the rest of the length. The first hoop shall be placed less than 2" from joint face.

Transverse Reinforcement (within l_o):

$A_{sh} \geq 0.3[(s)(b_c)(f'_c)/(f_{yt})]/[(A_g/A_{ch})-1] = 0.49$ in² (for s = 3" and $f'_c = 5$ ksi)
$A_{sh} \geq 0.09[(s)(b_c)(f'_c)/(f_{yt})] = 0.38$ in² (for s = 3" and $f'_c = 5$ ksi)

$A_{sh} \geq 0.3[(s)(b_c)(f'_c)/(f_{yt})]/[(A_g/A_{ch})-1] = 0.81$ in² (for s = 3" and $f'_c = 8$ ksi)
$A_{sh} \geq 0.09[(s)(b_c)(f'_c)/(f_{yt})] = 0.61$ in² (for s = 3" and $f'_c = 8$ ksi)

From these requirements, #4 hoops & ties are selected ($A_{sh} = 0.60$ in²) for where $f'_c = 5$ ksi and #5 hoops & ties are selected ($A_{sh} = 0.93$ in²) for where $f'_c = 8$ ksi.

Shear Strength:

Φ = 0.75 for shear
ΦVn = ΦVc + ΦVs
ΦVc = Φ4(λ√f'c)•(bw)•(d) (without shear reinforcing)
ΦVc = Φ2(λ√f'c)•(bw)•(d) (with shear reinforcing)
ΦVs = Φ(Av•Fy•d) / S

S = hoop / stirrup spacing
Smax = 21.6.4
Min Reinforcement - #3's @ d / 2 (if Vs provided)
Av = area of shear reinforcement

h in	bw in	d in	Fy ksi	λ	f'c psi	Size # Hoops / Stirrups	# of legs	S in	Av in²	ΦVc K	ΦVs K	ΦVn K
20.00	20.00	15.38	60	0.75	5000	4	3	3.00	0.589	24.5	135.8	160.3
20.00	20.00	15.38	60	0.75	5000	4	3	6.00	0.589	24.5	67.9	92.4
20.00	20.00	15.38	60	1.00	8000	5	3	3.00	0.920	41.3	212.3	253.5
20.00	20.00	15.38	60	1.00	8000	5	3	6.00	0.920	41.3	106.1	147.4

"PCA Column" was used to check flexural and axial combinations. Full documentation and interaction diagrams can be found in Appendix G.

Flexural/Axial Strength:

Story	Pu (K)	Mu (ft-K)	Moment Capacity @ Pu
2	1856.0	104	226.4
3	1539.0	98	296.0
4	1315.0	96	320.3

f'c = 8 ksi
normalweight concrete

Flexural/Axial Strength (continued):

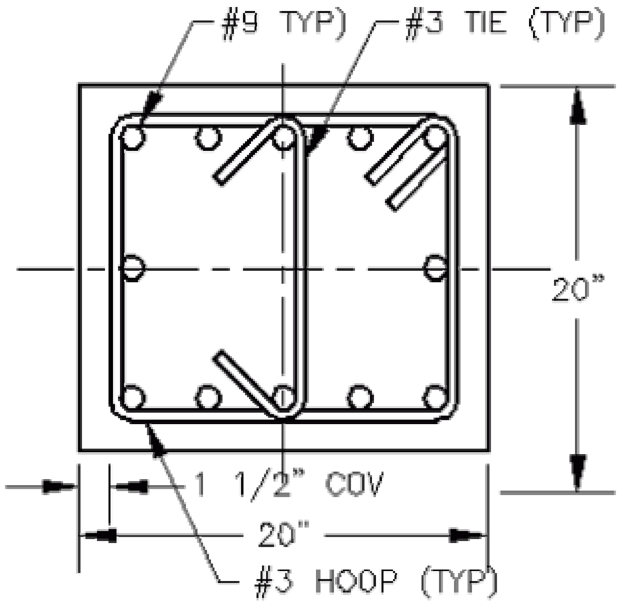
Story	Pu (K)	Mu (ft-K)	Moment Capacity @ Pu
5	1106	105	186.6
6	914	112	219.7
7	739	117	235.0
8	582	119	239.2
9	442	120	238.4
10	333	119	243.5
11	248	117	246.0
12	163	120	242.9
RF	116	109	239.6
PH	60	170	234.0

f'c = 5 ksi
lightweight concrete

Column Design Summary:

Longitudinal (Flexural & Axial) Reinforcement –
(8) #9s distributed evenly around 4 faces
Transverse Reinforcement –
Supporting Floors 5 - PH
#4 hoops & ties @ 3" O.C. with in l_o (1/6th of the clearspan from each joint face)
#4 hoops & ties @ 6" O.C. in middle sections
Supporting Floors 2 - 4
#5 hoops & ties @ 3" O.C. with in l_o (1/6th of the clearspan from each joint face)
#5 hoops & ties @ 6" O.C. in middle sections

Typical Concrete Beam Design:



As per ACI 3-18 Sections 21.5.3, transverse reinforcement (hoops) must be spaced at maximum of 3.5" (d/4 = 3.86") for a distance greater than or equal to twice the member depth from face of each support, and at 7" (d/2 = 7.72") along the rest of the length. The first hoop shall be placed less than 2" from face of support.

Shear Strength:

Φ = 0.75 for shear
$\Phi Vn = \Phi Vc + \Phi Vs$
$\Phi Vc = \Phi 4(\lambda\sqrt{f'c})\bullet(bw)\bullet(d)$ (without shear reinforcing)
$\Phi Vc = \Phi 2(\lambda\sqrt{f'c})\bullet(bw)\bullet(d)$ (with shear reinforcing)
$\Phi Vs = \Phi(Av\bullet Fy\bullet d) / S$

S = hoop / stirrup spacing
Smax = 21.6.4
Min Reinforcement - #3's @ d / 2 (if Vs provided)
Av = area of shear reinforcement

h in	bw in	d in	Fy ksi	λ	f'c psi	Size # Hoops / Stirrups	# of legs	S in	Av in²	ΦVc w/ Vs K	ΦVs K	ΦVn K
18.00	20.00	15.44	60	0.75	5000	3	3	3.50	0.331	24.6	65.8	90.3
18.00	20.00	15.44	60	0.75	5000	3	3	7.0	0.331	24.6	32.9	57.5
18.00	20.00	15.44	60	1.00	8000	3	3	3.50	0.331	41.4	65.8	107.2
18.00	20.00	15.44	60	1.00	8000	3	3	7.0	0.331	41.4	32.9	74.3

Vu = 36.0 k
$A_{V\ MIN}$ = 0.12 in² @ s = 7"

Flexural Strength:

Φ = 0.90 for tension control
Es = 29000 ksi
Fs' = (0.003/c)(c-d')(Es) ≤ 60

ρ max = 0.75ρb + ρ'(Fs' / Fy)
ρb = 0.85(β1)(f'c / Fy)(87,000 / (87,000 + Fy))
ρ = As / (bw)(d)
ρ' = As' / (bw)(d)

a=(β1)c
β1 = 0.85 (f'c ≤ 3000 psi); 0.65 (f'c ≥ 8000 psi); linear between

If Fs' = 60, a = ((As•Fy)-(As'•Fy)) / (0.85•f'c•b)
If Fs' < 60, (As Fy) = (As'•(0.003 / c)•(c - d')•Es') + (0.85•f'c•b•β1•c)
ΦMn = Φ [0.85 (f'c)(a)(b)(d - a/2) + (As')(Fs')(d-d')]

Tension Steel		
bar	# of bars	As
9	5	5.0
		0.00
		0.00
	As =	5.0

Compression Steel		
bar	# of bars	As
9	5	5.0
		0.00
		0.00
	As' =	5.0

$$c = \frac{(-\alpha) \pm \sqrt{(\beta^2 - 4\alpha\gamma)}}{2\alpha}$$

h	bw	Fy	(f'c)	Tension Steel				Compression Steel			
				Bars	Max Bar Size	# of Layers	As	Bars	Max Bar Size	# of Layers	As'
in	in	ksi	psi				in²				in²
18.00	20.00	60	5000	5#9	9	1	5.0	5#9	9	1	5.0
18.00	20.00	60	8000	5#9	9	1	5.0	5#9	9	1	5.0

d	d'	Quad. Eq. Coefficients			c	a	Fs'	β1	ρ min	ρ	ρ max	ΦMn	
in	in	α	β	γ	in	in	ksi					ft-K	
15.44	2.56	68.00	134.19	-1108.0	3.17	2.54	16.65	0.80	0.0035	0.0161	0.0250	308.88	Pu = 256.7 ft-k
15.44	2.56	88.40	134.19	-1108.0	2.86	1.86	9.10	0.65	0.0045	0.0161	0.0250	318.93	As, A's > 4.1 in²

Torsion:

$\Phi T_n = \Phi 2(A_O)(A_t)(f_{yt})\cot\theta / s$ = 463.2 ft-k
A_t = 0.11 in² @ s = 7"
$A_l = (A_t/s)p_h(f_{yt}/f_y)\cot^2\theta$ = 1.06 in² ($A_{l\ Available}$) = 2.9 in², (2)#9s + flexural excess

Beam Design Summary:

Longitudinal (Flexural & Axial) Reinforcement –
(5) #9s distributed evenly @ each face
Transverse Reinforcement –
All Supported Floors
#3 hoops & ties @ 3.5" O.C. with in l_o (2x member depth from face of support)
#3 hoops & ties @ 7" O.C. in middle sections

Concrete Shear Wall Design:

Shear Strength:

$\Phi V_{n\,MAX} = \Phi 10 A_{cv}\sqrt{f'c}$ (per pier/wall)

$\Phi V_n = \Phi A_{cv}[\alpha_c\, \lambda\, \sqrt{f'c} + \rho_t(f_y)]$

Wall I.D.	Φ	A_{cv} in²	α_c	λ	f'_c psi	ρ_t	f_y psi	Steel Desc	A_s in²	spacing in	wall t in	L_w in	ΦV_n K
D,E	0.75	4640	2	0.75	5000	0.0032	60000	2#5s	0.62	16	16	290	1043.4
$\Phi V_{n\,MAX}$ = 2460.7 K (upper limit)								Supported Floors 5 - PH					Vu = 762.6 k
D,E	0.75	4640	2	1	8000	0.0032	60000	2#5s	0.62	16	16	290	1296.8
$\Phi V_{n\,MAX}$ = 3112.6 K (upper limit)								Supported Floors 2 - 4					Vu = 805.0 k
3,4,5	0.75	4400	2	0.75	5000	0.0032	60000	2#5s	0.62	16	20	220	989.4
$\Phi V_{n\,MAX}$ = 2333.5 K (upper limit)								Supported Floors 5 - PH					Vu = 778.1 k
3,4,5	0.75	4400	2	1	8000	0.0032	60000	2#5s	0.62	16	20	220	1229.7
$\Phi V_{n\,MAX}$ = 2951.6 K (upper limit)								Supported Floors 2 - 4					Vu = 966.0 k

$\Phi V_{n\,MAX} = \Phi 8 A_{cv}\sqrt{f'c}$ (all piers/walls in D,E)
$\Phi V_{n\,MAX}$ = 7874.3 K (for f'c = 5 ksi)
$\Phi V_{n\,MAX}$ = 9960.3 K (for f'c = 8 ksi)

$\Phi V_{n\,MAX} = \Phi 8 A_{cv}\sqrt{f'c}$ (all piers/walls in 3,4,5)
$\Phi V_{n\,MAX}$ = 7467.0 K (for f'c = 5 ksi)
$\Phi V_{n\,MAX}$ = 9445.2 K (for f'c = 8 ksi)

$\rho_t = 0.0032 > 0.0025$ OK
$\rho_l = 0.0032 > 0.0025$ OK

Boundary Element Requirements:

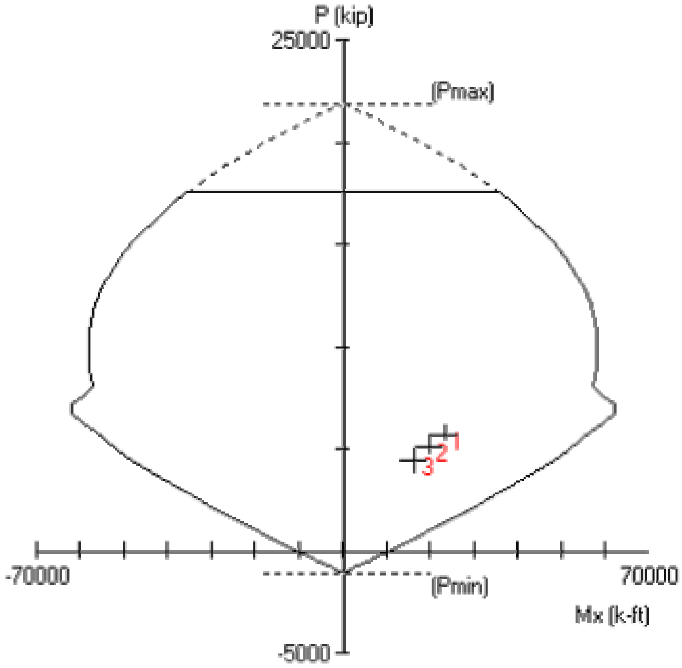
Wall I.D.	Pu K	Mu ft-K	y in	Ig in^4	Ag in²	lw in	t in	fc ksi	f'c ksi	0.2f'c ksi
D,E-2/3/4	6957.8	57470.5	145	9.430E+09	4640	290.00	16.00	1.510	8	1.6
D,E-5-PH	4573.0	34597.4	145	9.430E+09	4640	290.00	16.00	0.992	5	1.0
3,4,5-2/3/4	5651.0	23320.8	110	3.904E+09	4400	220.00	20.00	1.292	8	1.6
3,4,5-5/PH	3813.8	13375.9	110	3.904E+09	4400	220.00	20.00	0.871	5	1.0

Boundary elements are not required, but provided through the use of concrete columns (See floorplans)

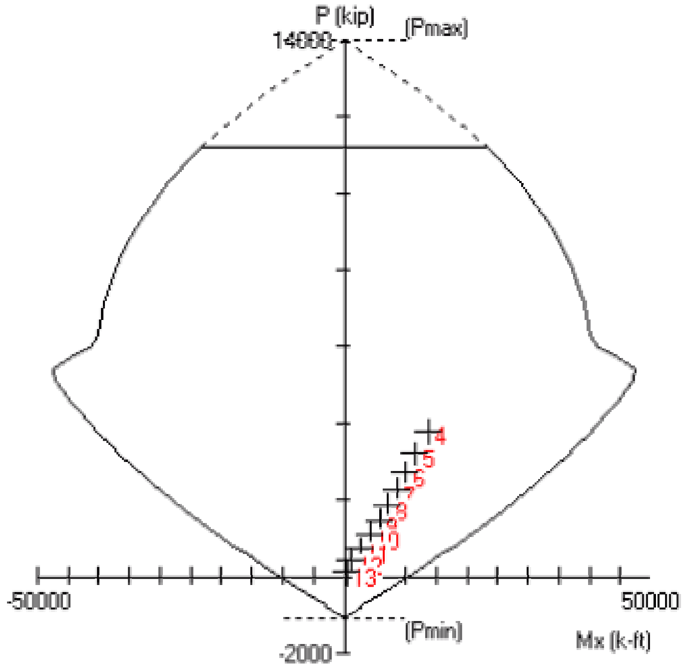
Flexural & Axial Strength:

Shear Walls 3,4,5 - Considering (20)#9s as flexural reinforcement for each of the walls, the following results are calculated.

Supporting Floors 2-4 (f'c = 8 ksi):

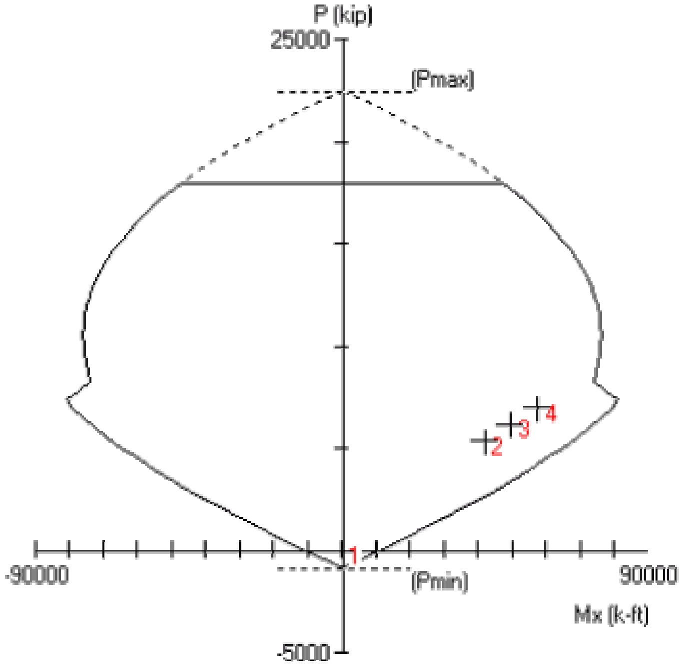


Supporting Floors 5-PH (f'c = 5 ksi):

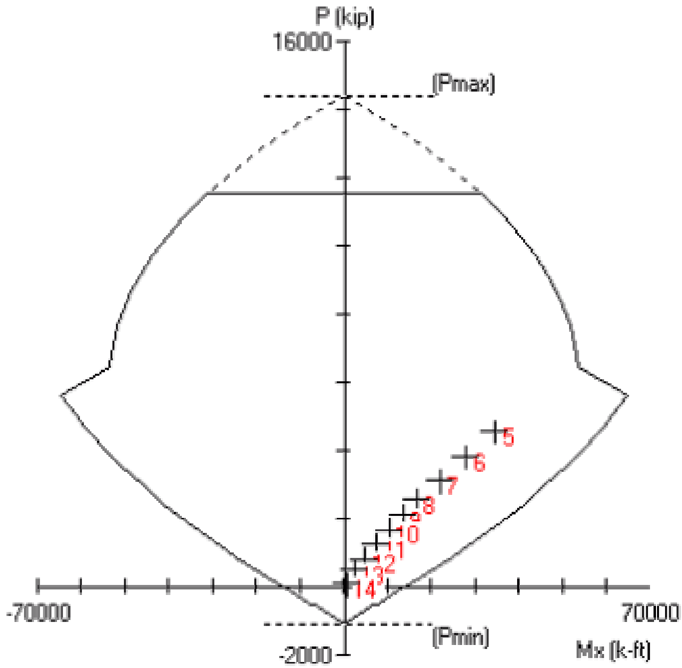


Shear Walls D,E - Considering (20)#9s as flexural reinforcement for each of the walls, the following results are calculated.

Supporting Floors 2-4 (f'c = 8 ksi):



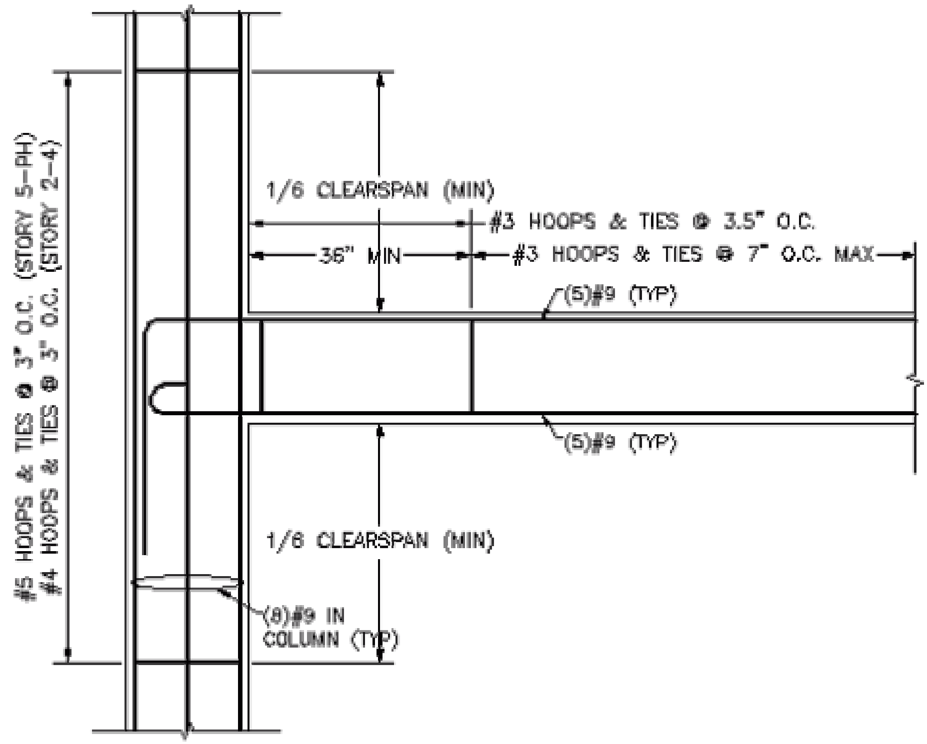
Supporting Floors 5-PH (f'c = 5 ksi):

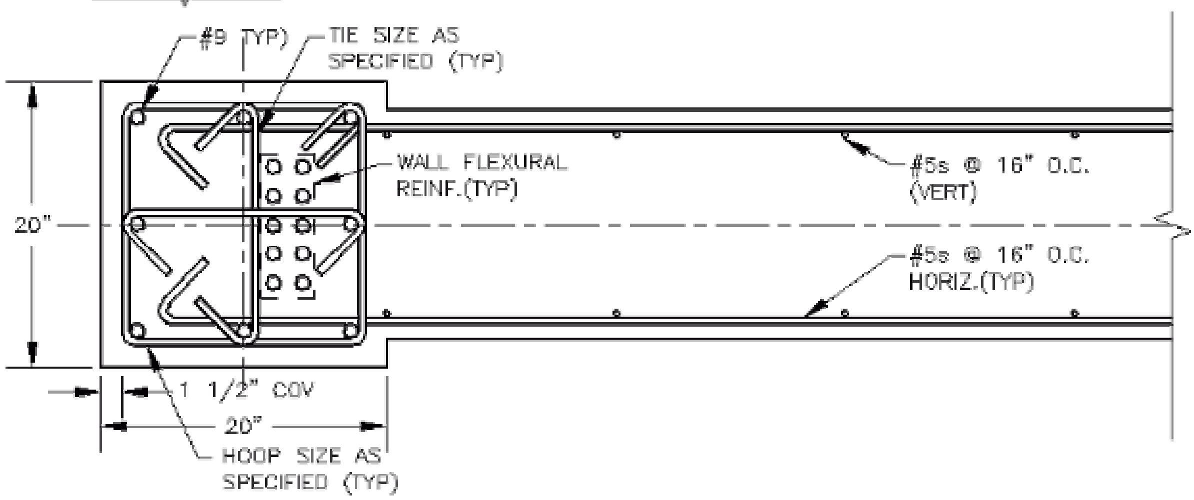


Wall Design Summary:

Flexural Reinforcement – All Supported Floors
(20) #9s distributed evenly, 10 @ each edge
Longitudinal Reinforcement – All Supported Floors
(2)#5s at 16" O.C. Max (1 each face)
Transverse (Shear) Reinforcement – All Supported Floors
(2)#5s at 16" O.C. Max (1 each face)

Typical Details:





Section 14 – Foundation Considerations

Overturning:

The increased design wind velocity results in an increased overturning moment. The current foundation would have to be altered, namely the number of piles supporting the shear walls. The overturning moments of the existing building are roughly 80,000 ft-k (between 2 wide frames) in one direction and 60,000 ft-k in the other (between 3 narrow frames). Overturning moments in the new structure are roughly 135,000 ft-k and 113,000 ft-k.

Other Considerations:

Other foundation changes would be needed to fully support the shear walls along their lengths. A foundation design for the new structure was beyond the scope of this project. This project focused on the lateral force resisting system.

Section 15 – Drift

Story	Item	Load	DriftX	DriftY
PHRF	Max Drift X	WIND	0.002077	
PHRF	Max Drift Y	WIND		0.001868
RF	Max Drift X	WIND	0.002165	
RF	Max Drift Y	WIND		0.002368
STORY12	Max Drift X	WIND	0.00217	
STORY12	Max Drift Y	WIND		0.002364
STORY11	Max Drift X	WIND	0.002163	
STORY11	Max Drift Y	WIND		0.002345
STORY10	Max Drift X	WIND	0.002135	
STORY10	Max Drift Y	WIND		0.002304
STORY9	Max Drift X	WIND	0.002072	
STORY9	Max Drift Y	WIND		0.002232
STORY8	Max Drift X	WIND	0.001992	
STORY8	Max Drift Y	WIND		0.002125
STORY7	Max Drift X	WIND	0.001863	
STORY7	Max Drift Y	WIND		0.001974
STORY6	Max Drift X	WIND	0.001686	
STORY6	Max Drift Y	WIND		0.001772
STORY5	Max Drift X	WIND	0.001453	
STORY5	Max Drift Y	WIND		0.001514
STORY4	Max Drift X	WIND	0.001158	
STORY4	Max Drift Y	WIND		0.001193
STORY3	Max Drift X	WIND	0.000791	
STORY3	Max Drift Y	WIND		0.000801
STORY2	Max Drift X	WIND	0.000335	
STORY2	Max Drift Y	WIND		0.000286

For the overall design, drifts due to wind loads are the controlling factor. The level of structural stiffness needed to limit drifts (due to wind) to 0.25% or L/400 provides enough lateral strength to carry all seismic and wind load combinations. L/400 is used as a drift limit due the presence of exterior brick veneer that is sensitive to excessive displacements.

Drifts due to seismic loads, which are limited to $0.020h_x$ (2%), can be found in Appendix H.

TABLE 12.12-1 ALLOWABLE STORY DRIFT, $\Delta_a^{a,b}$

Structure	Occupancy Category		
	I or II	III	IV
Structures, other than masonry shear wall structures, 4 stories or less with interior walls, partitions, ceilings and exterior wall systems that have been designed to accommodate the story drifts.	$0.025h_{sx}^{c}$	$0.020h_{sx}$	$0.015h_{sx}$
Masonry cantilever shear wall structures [d]	$0.010h_{sx}$	$0.010h_{sx}$	$0.010h_{sx}$
Other masonry shear wall structures	$0.007h_{sx}$	$0.007h_{sx}$	$0.007h_{sx}$
All other structures	$0.020h_{sx}$	$0.015h_{sx}$	$0.010h_{sx}$

[a] h_{sx} is the story height below Level x.
[b] For seismic force–resisting systems comprised solely of moment frames in Seismic Design Categories D, E, and F, the allowable story drift shall comply with the requirements of Section 12.12.1.1.
[c] There shall be no drift limit for single-story structures with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the story drifts. The structure separation requirement of Section 12.12.3 is not waived.
[d] Structures in which the basic structural system consists of masonry shear walls designed as vertical elements cantilevered from their base or foundation support which are so constructed that moment transfer between shear walls (coupling) is negligible.

Section 16 – Construction Schedule & Cost Impact (Breadth Topic 1)

The First Albany Building took 24 months to build and cost roughly $25 million (excluding design service and property costs). The original schedule was a rotating 5 week schedule per floor (generally speaking) and total construction time was projected at 26 months. Taking into consideration the changes made through out this thesis project, the overall schedule was minimally affected. The same rotating 5 week schedule is projected to be sufficient. The original schedule was controlled by the time needed by the mechanical, electrical, and plumbing trades; roughly 5 weeks per floor level. Construction of the concrete shear walls could be completed nearly in parallel with the structural steel erection. Shifting the shear wall construction phase (per floor) to slightly lead the steel erection phase would provide the time necessary to remove concrete formwork and allow the structural steel to be connected to the shear wall. The building layout and size would allow for a single crane to operate from one location for the entire project, with the location depending on the exact site layout (property setbacks, surrounding space). A projected construction schedule can be found in Appendix I. Considerations specifically taken into account for creating the schedule include coordinating the three principle trades (MEP) in such a fashion that they aren't interfering in each other's work and which tasks/phases can be overlapped. The projected schedule spans 26 months from breaking ground to installing the last outlet cover.

Considering a building that is identical to the First Albany Building except for the new structural system designed for a location in Charleston, the cost of the building is projected to increase. This is mainly due to the need for a more robust lateral structural system. Switching to full composite action and choosing a thinner floor slab (decrease from 4.5" to 4") does reduce material costs, but is offset by extra labor required for shear stud connector installation. Labor costs remain unchanged for the slab since costs are based square footage rather than volume of concrete placed. Overall Steel fabrication and erection costs for the floor system also remain relatively unchanged because the only factor that changed was raw tonnage of steel (same number of pieces, but smaller shapes).

Material:

Structural Steel (floor)	-117 (ton)	-$110,500
Structural Steel (lateral)	-101.5 (ton)	-$96,000
Slab Concrete	-293 (CY)	-$23,500
Wall Concrete	+1500 (CY)	+$120,000

Labor:

Shear Stud Connectors	+9500 (EA)	+$166,000
Structural Steel (lateral)	–101.5 (ton)	-$256,000
Shear Walls (+forms & reinf)	+1500 (CY)	+$600,000

Estimated Cost Difference:

Total +$400,000

A rough estimate used by many engineers and estimators for structural steel is $3500 per ton (erected). Although shear stud installation costs vary widely by region, one installed shear stud (on average) equates to 10 lb of steel. Data from 2008 indicates that costs associated with structural steel can be broken down to 27% for materials, 33% for shop labor, 27% for erection labor, and 13% for other costs. Various sources price concrete at $80 CY (2008 National average was $75) and placed structural concrete at about $500 per CY.

Considering the new structural system is designed to resist wind velocities 50% higher than the original and increased seismic forces, the projected cost increase is minimal.

(Reference - $ave More Money, by Charles J. Carter, P.E., S.E., and Thomas J. Schlafly; http://www.whysteel.org/uploadedData/Design Economy - Modern Steel.pdf)

Section 17 –Energy Cost Saving Efforts (Breadth Topic 2)

Cooling demands for South Carolina are obviously much higher than New York, and the opposite for heating demands. For purposes of minimizing energy usage (to reduce operating costs) several options have been considered.

Installing a reflective roof surface (rather than a black asphalt based surface) can reduce cooling loads of the upper floors.

Details of Comparison:

Cooling degree days for location chosen [Annual °F-day]	2010
Solar load for location chosen [Annual Average Btu/ft² per day]	1462.4
Cooling load for black roof (SR=5%;IE=90%) [Btu/ft² per year]	5088
Cooling load for proposed roof (SR=80%;IE=60%) [Btu/ft² per year]	1848
Difference [Btu/ft² per year]	3240

(http://www.ornl.gov/sci/roofs+walls/facts/CoolCalcEnergy.htm)

Installing solar panels to help meet energy needs is viable option. South Carolina receives significantly more solar radiation than New York. On average throughout the year in South Carolina, a flat plate collector facing south at fixed tilt (33 degrees) can collect about 5.5 kwh/m² (0.51 kwh/ft²). To be commercially viable, the efficiency needed by solar cells is about 15%. It is found that "cost effective" systems can have such efficiency ratings. Considering that there is a screen wall on the roof with roughly 2500 ft² facing in any orthogonal direction (see Appendix J – Photos), a solar array coving such area could produce upwards of 200 kwh per day. If the price of electricity is $0.0845 per kwh, the array could save over $6000 per year to help offset the initial cost of the array.

(http://rredc.nrel.gov/solar/old_data/nsrdb/redbook/atlas/)
(http://www.sciencedaily.com/releases/2007/05/070502153700.htm)
(http://www.solarpanelmanual.com/solar-panel-efficiency.php)
(http://www.eia.doe.gov/cneaf/electricity/epm/table5_6_a.html)

Another energy saving idea that was considered but not explored in depth was using a highly reflective glazing with a lower "e" value to reduce solar gains by the building (window specifications were not made available).

Energy storage systems, such as generating ice overnight to meet cooling demands the next day were abandoned early on in this project. The building electricity demands aren't high enough to qualify for a significant rate decrease during off-peak time periods.

Section 18 – Summary & Conclusions

This project was an excellent exercise in structural design. It incorporated a variety of topics necessary for the design of a multistory structure. The original proposal was expected to have the structural design controlled by seismic forces. It wasn't until after wind load calculations that it became clear that drift limitations under those wind loads would control the design. Even though conditions present during a hurricane (high wind velocities) ended up being the controlling factor, this project was an excellent exercise in seismic analysis and design. Strength requirements were controlled by seismic forces in the upper stories and wind forces in the lower stories. An added benefit of wind velocities and drift limitations being the ultimate controlling factors, the same design could be used in a variety of locations along the east coast. Shearwall thicknesses for the structure ended up being 16" and 20" in each of the orthogonal directions and uniform though the height of the building.

Topics incorporated into this project ranged from proper usage of computer modeling software to what was basically hand calculation (through the use of custom made spread sheets). Other significant topics were reinforced concrete design, composite structural steel & concrete design, dynamic analysis, and earthquake resistant design.

The resulting structural design incorporated a lightweight composite action steel floor system and a special reinforced shear wall lateral system. Reinforcement detailing of the shearwalls and core elements was mostly prescribed, rather than truly designed. Chapter 21 of ACI 3-18 prescribes hoop/tie sizes and spacing of them in a stringent fashion. From the commentary in the code, the aim is to keep the concrete 'contained' to retain individual member strength. In the recent strong earthquake in Haiti, it was clearly evident why such code requirements exist. In picture after picture there were collapsed concrete structures. In many of these pictures, the reinforcement, or lack there of, was exposed. Hoops and ties were barely present and the concrete shook and fell apart into rubble.

Architecturally speaking, only minor changes would have to be made. Other than shifting elevator locations, the building layout remains largely unchanged.

Moderate foundation changes would be required due to the increased overturning moment generated by increased design wind velocities. However, a foundation design for the new structure was beyond the scope of this project. This project focused on the lateral force resisting system.

From a construction stand point, the new structural system poses few obstacles. The total time needed to complete the construction process, 26 months, is relatively unchanged. The only significant factor is the added cost (estimated at $400,000), mostly due to an increase in the required strength/stiffness for higher lateral loads. Efforts were made during the design process to keep the walls and columns a uniform size so that concrete formwork could be reused from floor to floor.

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