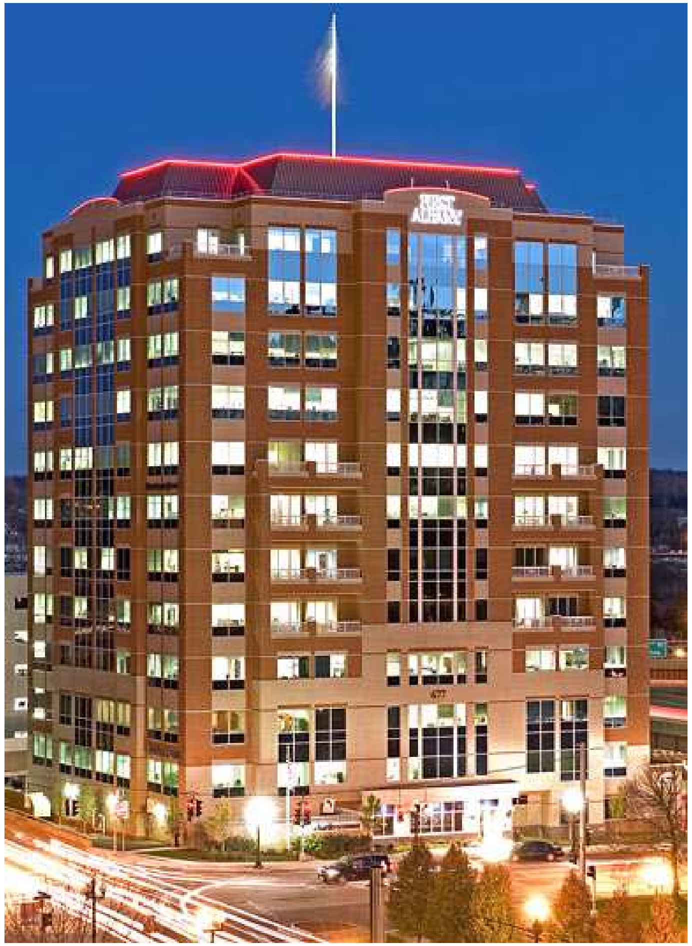


The First Albany Building

677 Broadway, Albany, NY

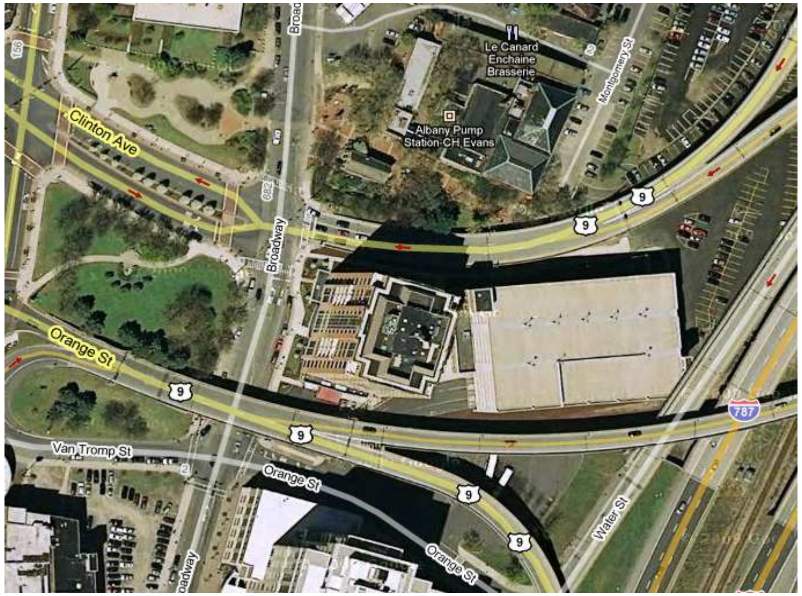
Gerald Craig

Bachelor of Architectural Engineering Candidate
Structural Option

The First Albany Building

677 Broadway, Albany, NY

- Downtown Albany
- Near state and government offices
- Conveniently located off of I-787
- 12 Stories + Elevator Penthouse



<u>Building Uses:</u>

- Angelo's 677 Prime
- General office space
- Condominiums (possible future use)

The First Albany Building

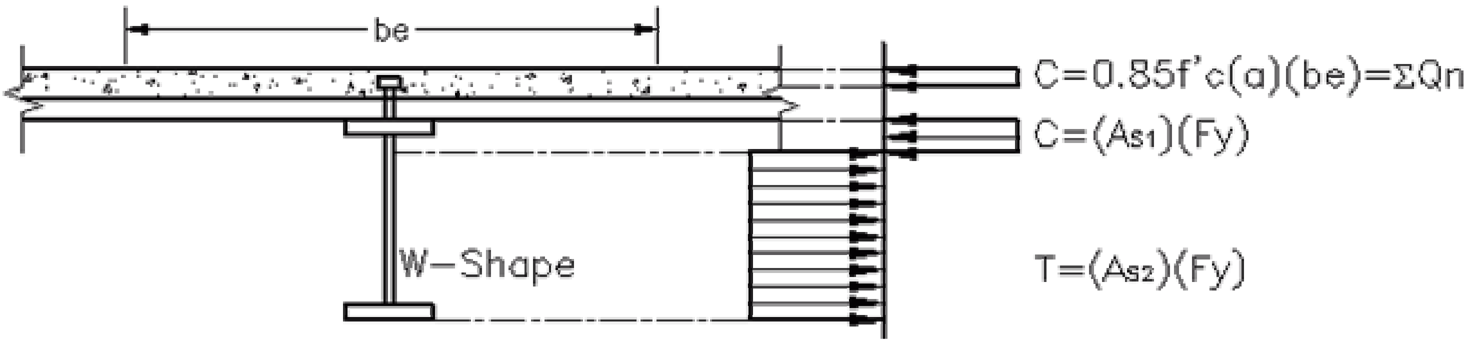
677 Broadway, Albany, NY

Structural Systems:

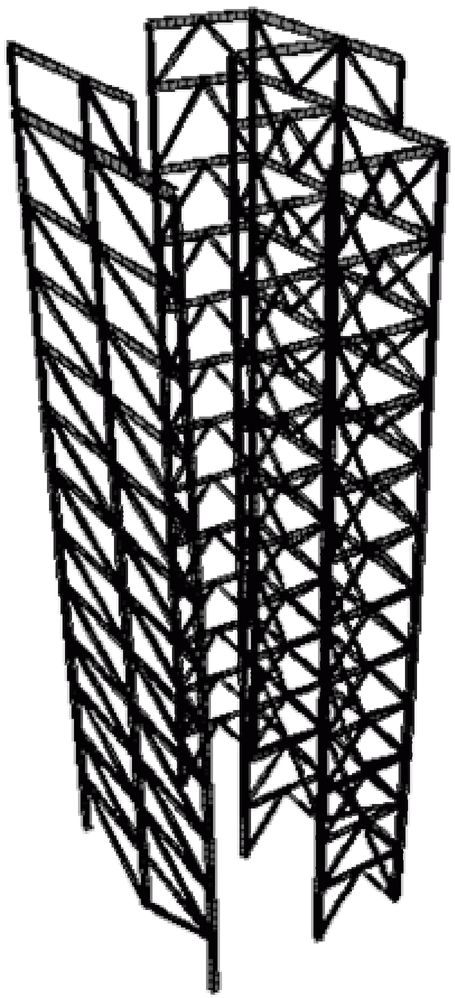
- Foundation
  - First floor at grade, no basement
  - Steel H-Piles
  - Grade Beams

- Floor System
  - Composite Steel & Concrete Design (partial composite action)
  - Resists gravity forces & loads only
  - Simply Supported Beams

- Lateral Force Resisting System
  - Structural Steel, Concentrically Braced Frames



Partial Composite Action



Braced Frames

The First Albany Building

677 Broadway, Albany, NY

Structural Systems Design Requirements:

- Live Loads
  - Office Space (2-8) & Partitions, 75 psf
  - Office Space (9-12) with Access Flooring for computer use & Partitions, 115 psf
  - Office Space (2) with file storage, 125 psf

- Little significant seismic activity
  - Seismic Design Category B
  - Small potential ground accelerations (S_{DS} = 0.28, S_{D1} = 0.12)

- No extreme wind conditions
  - Minimum design wind velocity (v = 90 mph)

FIGURE 6-1C DESIGN WIND VELOCITY

TABLE 11.6-1 SEISMIC DESIGN CATEGORY BASED ON SHORT PERIOD RESPONSE ACCELERATION PARAMETER

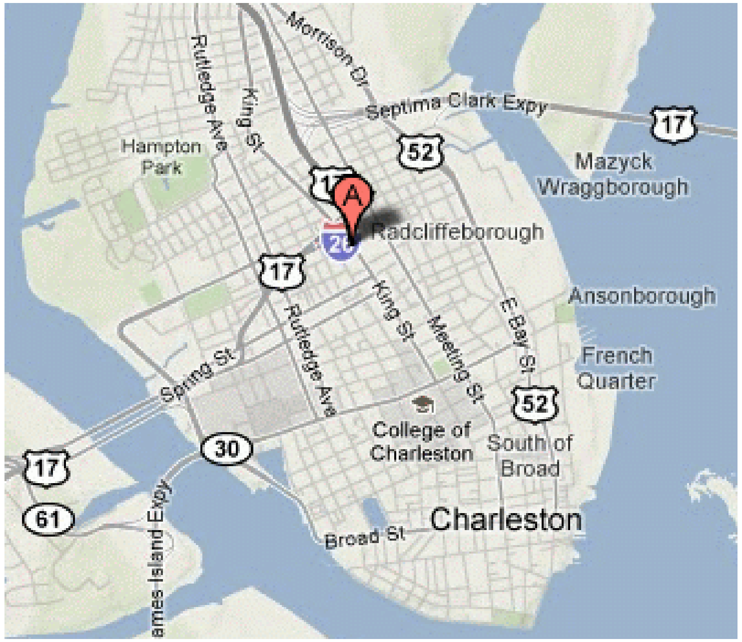
Value of S_{DS}	Occupancy Category		
	I or II	III	IV
$S_{DS} < 0.167$	A	A	A
$0.167 \le S_{DS} < 0.33$	B	B	C
$0.33 \le S_{DS} < 0.50$	C	C	D
$0.50 \le S_{DS}$	D	D	D

TABLE 11.6-2 SEISMIC DESIGN CATEGORY BASED ON 1-S PERIOD RESPONSE ACCELERATION PARAMETER

Value of S_{D1}	OCCUPANCY CATEGORY		
	I or II	III	IV
$S_{D1} < 0.067$	A	A	A
$0.067 < S_{D1} < 0.133$	B	B	C
$0.133 \le S_{D1} < 0.20$	C	C	D
$0.20 \le S_{D1}$	D	D	D

A New Building

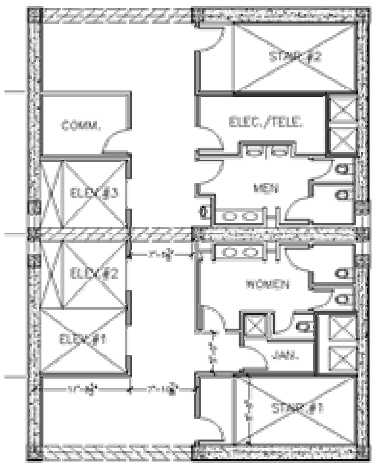
- Visually identical Building
- Identical Building Use
- New proposed location
    - Charleston, South Carolina
    - commercial sector
    - last exit off from I-26
    - height limitations (50-80 ft)
        - zoning variances are considered (report – appendix E)
        - go ahead with project for educational purposes



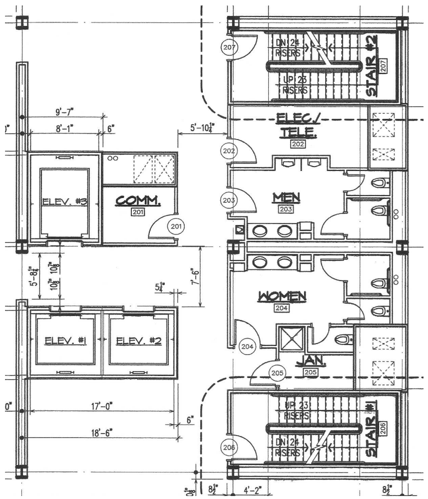
A New Building

- Slightly Altered Building Core Layout
  - Elevators Relocated
  - Symmetric Core Layout
  - Little effect on foot-traffic

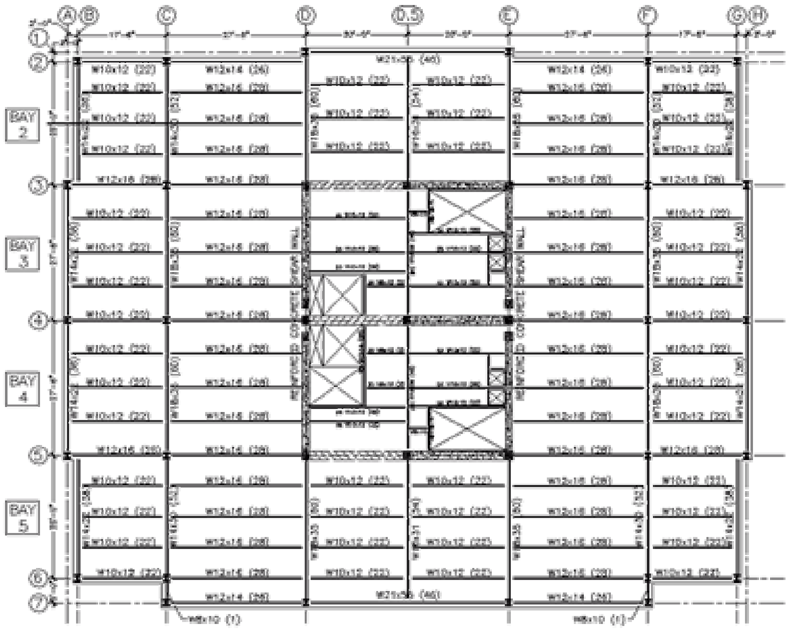
Existing Building Layout



Proposed Building Core Layout



Exiting Building Core Layout



Proposed Building Layout

A New Building

Charleston, SC

<u>Structural Systems Design Requirements</u>:

- Live Loads (same as previous)
  - Office Space (2-8) & Partitions, 75 psf
  - Office Space (9-12) with Access Flooring for computer use & Partitions, 115 psf
  - Office Space (2) with file storage, 125 psf
  - Reduced as allowed per (ASCE 7-05 4.8)

- Snow Loads
  - As calculated, 5 psf

- More chance of significant seismic activity
  - Seismic Design Category D (S_{DS} = 0.991, S_{D1} = 0.406)

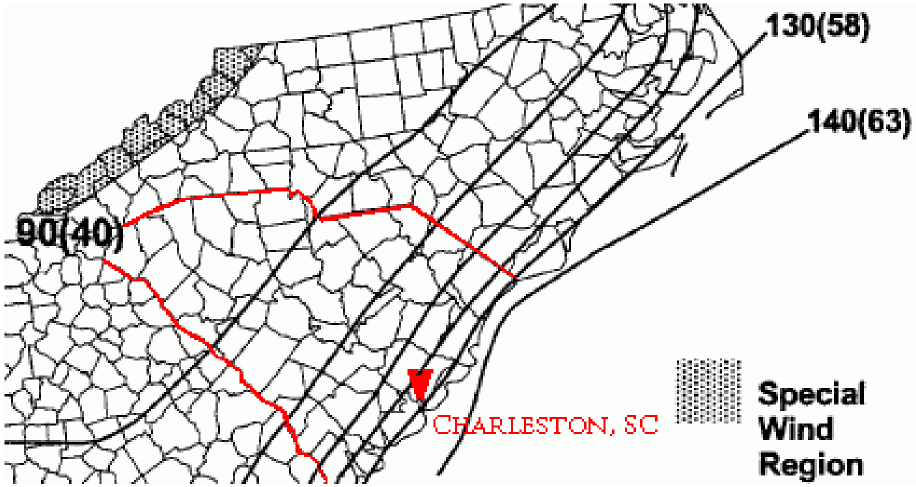
- Hurricane prone area
  - High design wind velocity (v = 140 mph)

TABLE 11.6-1 SEISMIC DESIGN CATEGORY BASED ON SHORT PERIOD RESPONSE ACCELERATION PARAMETER

Value of S_{DS}	Occupancy Category		
	I or II	III	IV
$S_{DS} < 0.167$	A	A	A
$0.167 \le S_{DS} < 0.33$	B	B	C
$0.33 \le S_{DS} < 0.50$	C	C	D
$0.50 \le S_{DS}$	D	D	D

TABLE 11.6-2 SEISMIC DESIGN CATEGORY BASED ON 1-S PERIOD RESPONSE ACCELERATION PARAMETER

Value of S_{D1}	OCCUPANCY CATEGORY		
	I or II	III	IV
$S_{D1} < 0.067$	A	A	A
$0.067 \le S_{D1} < 0.133$	B	B	C
$0.133 \le S_{D1} < 0.20$	C	C	D
$0.20 \le S_{D1}$	D	D	D



A New Building

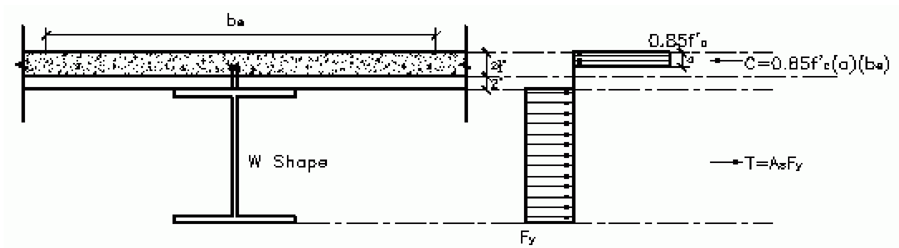
Charleston, SC

Structural Floor System Design:

- Main Objective: Reduce Effective Seismic Weight

- Thinner Floor Deck & Slab
  - 22 gage B-LOK 1.5”x 6” DECK, (United Steel Deck Catalog)
  - f’c = 4 ksi (lightweight concrete)
  - Total Slab Thickness = 4”
  - Weight = 1.6 psf (Composite Weight = 29 psf)
  - ΦVnt = 2980 # (Vu = 930.9 #)
  - ΦMno = 25.66 in-K (no studs present, conservative, Mu = 19.2 in-K)
  - Maximum Un-shored Span = 6.91’ (maximum beam spacing = 6.88’)
  - AWWF = 0.023 in^2 per ft

- Full Composite Action (Beams)
  - Floor member strength controlled by the amount of concrete in compression rather than the shear stud connectors ability to transfer forces from the steel beam to the concrete slab.
  - Reduces structural steel weight by ~20%
  - Increases shear stud connector count by ~130%
  - Design method & calculations found in Appendix A



A New Building
Charleston, SC

Selecting a Lateral Force Resisting System :

- Main Objective: Provide Stability & Stiffness

- Special Reinforced Concrete Shear Walls
  - Response Modification Factor (R) = 6
    (verses R = 5 for Composite Steel & Concrete Concentrically Braced Frames)
  - Educational value of earthquake resistant shear wall design
  - Adds considerable seismic weight
  - ASCE 7-05 Table 12.2-1 Height Limitation of 160':

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.

A New Building

Charleston, SC

Dynamic Lateral Analysis (Seismic):

- 3D Mathematical Model created using ETABS
  - "Cracked" sections were considered
    - Beams - 0.35Ig
    - Column - 0.70Ig
    - Shear Walls – $0.5f_{22}$
  - Effective seismic weight (applied as a uniform additional mass)
    - Steel Beams & Columns ~5.1 psf
    - Other Dead Loads ~77.1 psf
    - Core Structure as calculated (in report, Appendix C)

- Modal Superposition
  - Less conservative / more accurate
  - Equivalent Lateral Force Method not permitted by ASCE 7-05 Table 12.6-1

    $Ts = S_{D1} / S_{DS} = 0.406 / 0.991 = 0.4097$

    T < 3.5Ts ? (3.5*0.4097=1.434)

    Tc = 1.6199 (calculated north-south)

    Tc = 1.2702 (calculated east-west)

  - Modal Participating Mass ratios > 90% as per ASCE 7-05 12.9.1
  - Modal Combination, Complete Quadratic Combination
  - Base Shear lower limit (12.9.4)
    - 85% of base shear from ELF method
      - scale factor of 1.047 added to E-W direction
      - N-S direction base shear > 85% ELF method

Modal Participating Mass Ratios			
Mode	Period	UX	UY
1	1.946	0.146	0.615
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	3.008
	Totals -	95.609	92.600

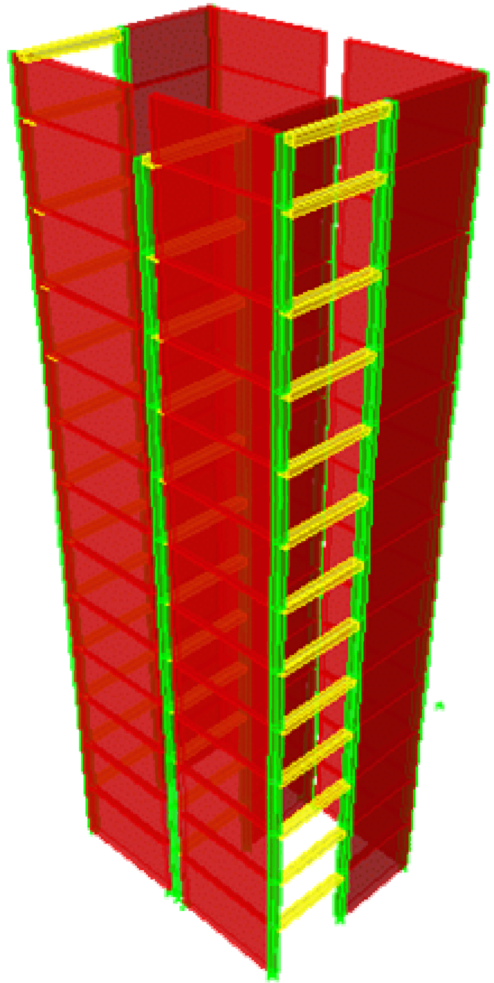
Story	F_x (K)	F_x (K)		F_x (K)	F_x (K)
	E-W			N-S	
	ELF	Modal		ELF	Modal
PH	32.2	24.5		25.2	20.2
RF	140.7	119.3		110.1	108.5
12	170.0	147.8		133.0	130.8
11	146.9	126.6		114.9	111.4
10	125.0	103.3		97.8	92.7
9	104.4	87.0		81.7	74.6
8	86.7	69.8		67.8	62.5
7	68.5	52.8		53.6	49.2
6	51.9	38.8		40.6	38.6
5	36.9	29.0		28.9	28.2
4	23.9	22.7		18.7	23.3
3	12.9	18.5		10.1	18.4
2	4.6	13.8		3.6	9.3
	Total 1004.7	854.0		Total 786.0	767.7

A New Building

Charleston, SC

Dynamic Lateral Analysis (Seismic):

- Structure Irregularities (12.3.2)
  - Torsional & Extreme Torsional Irregularity
    - Maximum ratio of extreme drift and average drift is about 1.1 (<1.2 for torsional irregularity)
  - No major Re-entrant Corner Irregularity
    - Ratios all less than 15%
  - No Diaphragm Discontinuities
  - No Out of Plane Offsets
  - System is parallel & symmetric
  - No Vertical Irregularities
    - No single story stiffness is less than the story above
    - Effective seismic weight is relatively constant
    - Lateral system is uniform
- Redundancy Factor (12.3.4.2)
  - The value of ρ shall be taken as 1.3
    - Removal of a shear wall results in extreme torsional irregularity
- Torsional Amplificaiton Factor (12.8.4.3)
  - Unnecessary, Type 1a or 1b torsional irregularity does not exist
- Direction of Loading (12.5.4)
  - Since the structure does not display any horizontal irregularities (specifically type 5), loads in each of the orthogonal directions are considered independently.



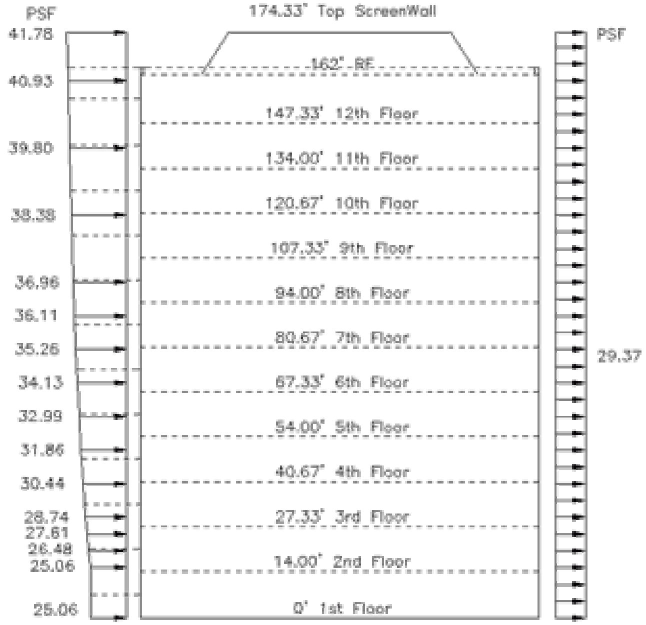
Core Structure

A New Building

Charleston, SC

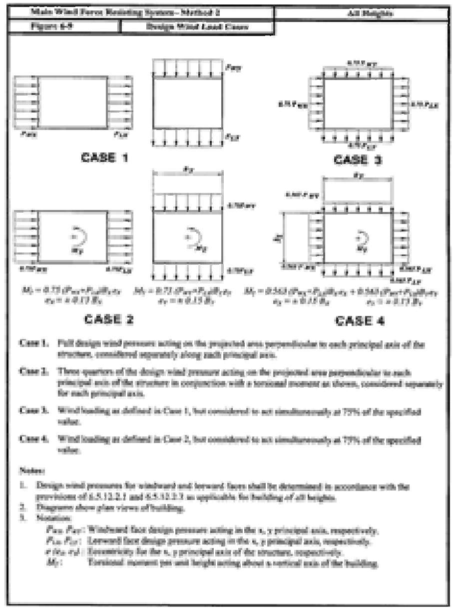
Lateral Analysis (Wind):

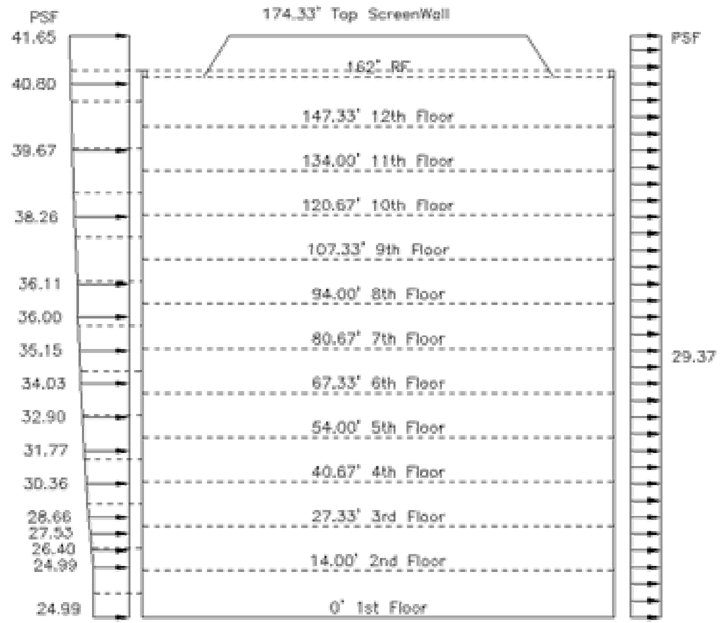
- Design Wind Pressures
  - Design wind velocity, 140 mph
  - Open, Flat Terrain
  - Applied as per ASCE 7-05 Figure 6.9



Wind Pressure N-S

Wind Forces Level	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						
RF*	175.0	144.4	175.0	144.4	27478.9	22670.4
12	133.6	112.2	308.6	256.6	19729.6	16566.7
11	125.5	105.5	434.1	362.1	16773.3	14104.9
10	123.6	104.1	557.7	466.2	14869.1	12524.1
9	121.2	102.6	678.9	568.8	12967.9	10981.0
8	119.5	101.0	798.4	669.9	11194.4	9463.9
7	118.0	99.2	916.4	769.1	9475.8	7967.3
6	115.4	97.0	1031.8	866.1	7731.8	6500.2
5	112.6	94.6	1144.4	960.7	6041.9	5079.0
4	109.3	91.9	1253.6	1052.6	4406.8	3704.3
3	104.8	88.1	1358.5	1140.7	2830.1	2379.0
2	103.4	86.9	1461.9	1227.6	1412.7	1187.5
	Total 1461.9	Total 1227.6			Total 134912.4	Total 113128.3





Wind Pressure E-W

A New Building

Charleston, SC

Design Forces:

- Load Combinations:

ASCE 7-05 2.3

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

- Strength Requirements:

- Wind loads controlled strength requirements for the lower floors
- Seismic loads controlled strength requirements for the upper floors

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

Element Forces

A New Building

Charleston, SC

<u>Lateral Structural System Design:</u>

Lightweight concrete was taken advantage of for floors 5 through the roof. All appropriate properties of lightweight concrete were considered (λ = 0.75, $E_c = 33w^{1.5}\sqrt{f'_c}$ = 2900 ksi). Normal weight concrete was used in the lower floors due to higher required strengths and 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.*

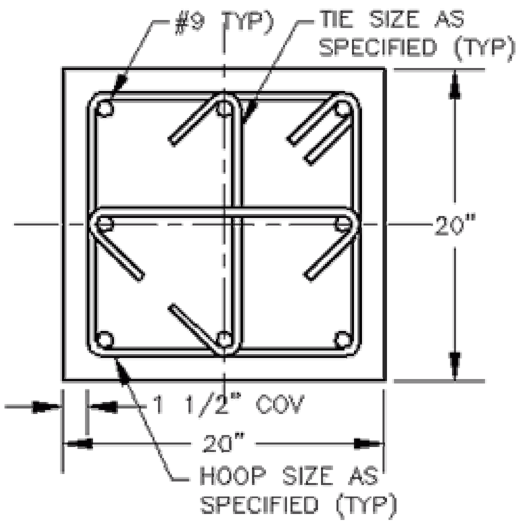
A New Building

Charleston, SC

<u>Lateral Structural System Design (Continued) :</u>

- Typical Column Design:

To satisfy 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 (l_o) 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 lo):

Ash ≥ 0.3[(s)(bc)(f’c)/(fyt)]/[(Ag/Ach)-1] = 0.49 in² (for s = 3” and f’c = 5 ksi)
Ash ≥ 0.09[(s)(bc)(f’c)/(fyt)] = 0.38 in² (for s = 3” and f’c = 5 ksi)
Ash ≥ 0.3[(s)(bc)(f’c)/(fyt)]/[(Ag/Ach)-1] = 0.81 in² (for s = 3” and f’c = 8 ksi)
Ash ≥ 0.09[(s)(bc)(f’c)/(fyt)] = 0.61 in² (for s = 3” and f’c = 8 ksi)

From these requirements, #4 hoops & ties are selected (Ash = 0.60 in²) for where f’c = 5 ksi and #5 hoops & ties are selected (Ash = 0.93 in²) for where f’c = 8 ksi.

- Column 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	0.60	24.5	138.4	162.8
20.00	20.00	15.38	60	0.75	5000	4	3	6.00	0.60	24.5	69.2	93.6
20.00	20.00	15.38	60	1.00	8000	5	3	3.00	0.92	41.3	212.3	253.5
20.00	20.00	15.38	60	1.00	8000	5	3	6.00	0.92	41.3	106.1	147.4

(Vu max = 93.5 K)

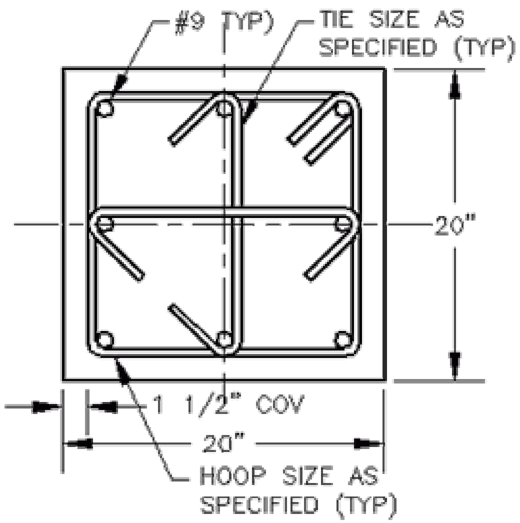
(Vu max = 76 K)

A New Building
Charleston, SC

Lateral Structural System Design (Continued) :

- Column Axial & Flexural Strength:

  "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

(Moment Capacities are for biaxial flexure @ each axial load)

-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. within 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. within l_o (1/6th of the clearspan from each joint face)
  - #5 hoops & ties @ 6" O.C. in middle sections

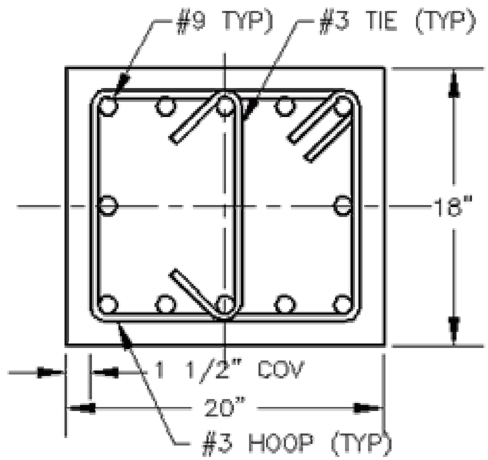
A New Building

Charleston, SC

<u>Lateral Structural System Design (Continued):</u>

- Typical 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 (l_{o}) 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.



- Beam Shear Strength:

$\Phi =$ 0.75 for shear

$\Phi V_n = \Phi V_c + \Phi V_s$

$\Phi V_c = \Phi 4(\lambda\sqrt{f'c})\cdot(bw)\cdot(d)$ (without shear reinforcing)

$\Phi V_c = \Phi 2(\lambda\sqrt{f'c})\cdot(bw)\cdot(d)$ (with shear reinforcing)

$\Phi V_s = \Phi(Av\cdot Fy\cdot d) / S$

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

Av_{MIN} = 0.12 in² @ s = 7”

A New Building

Charleston, SC

Lateral Structural System Design (Continued):

- Beam Flexural Strength:

$\Phi =$ 0.90 for tension control

$E_s = 29000$ ksi

$F_s' = (0.003/c)(c-d')(E_s) \leq 60$

$\varepsilon_s > 0.005$

$\rho b = 0.85(\beta 1)(f'c / Fy)(87{,}000 / (87{,}000 + Fy))$

$\rho = As / (bw)(d)$

$\rho' = As' / (bw)(d)$

$a = (\beta 1)c$

$\beta 1 = 0.85$ ($f'c \leq 3000$ psi); 0.65 ($f'c \geq 8000$ psi); linear between

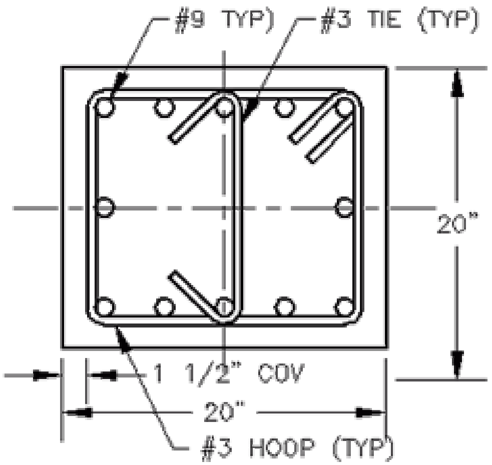
If $F_s' = 60$, $a = ((As \cdot Fy) - (As' \cdot Fy)) / (0.85 \cdot f'c \cdot b)$

If $F_s' < 60$, $(As\ Fy) = (As' \cdot (0.003 / c) \cdot (c - d') \cdot Es') + (0.85 \cdot f'c \cdot b \cdot \beta 1 \cdot c)$

$\Phi Mn = \Phi\ [0.85 (f'c)(a)(b)(d - a/2) + (As')(Fs')(d-d')]$

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	ε_s	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	0.015	308.88	Mu = 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	0.016	318.93	As, A's min = 4.1 in²



- Beam Design Summary:

Longitudinal (Flexural) Reinforcement

All Supported Floors

(5) #9s distributed evenly @ each face

(2) #9s placed in middle on each side

Transverse Reinforcement –

All Supported Floors

#3 hoops & ties @ 3.5" O.C. within l_o (2x member depth from face of support)

#3 hoops & ties @ 7" O.C. in middle sections

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Charleston, SC

<u>Lateral Structural System Design (Continued):</u>

- Shear Wall Design:

$\Phi V_{n\,MAX} = \Phi 10 A_{cv}\sqrt{f'c}$ (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

A New Building
Charleston, SC

<u>Lateral Structural System Design (Continued):</u>

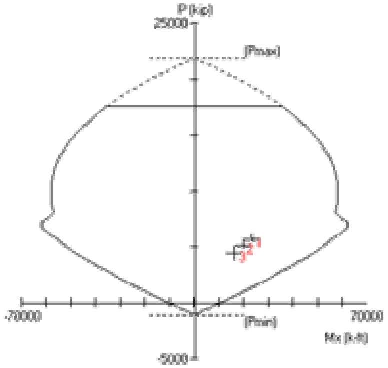
- Boundary Elements:

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

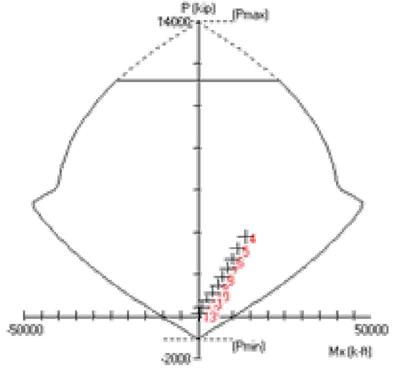
Special boundary elements not needed

Flexural & Axial Strength:

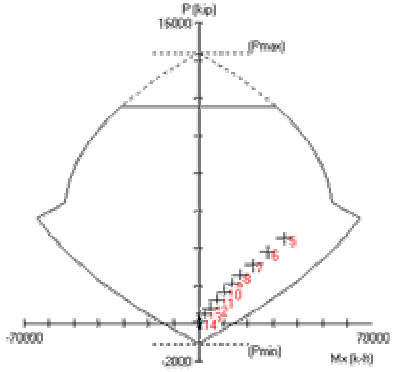
Shear Walls - Considering (20)#9s as flexural reinforcement for each of the walls, the following results are calculated.



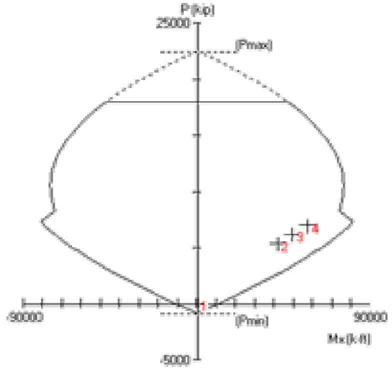
Walls 3,4,5
Supporting Floors 2-4 (f'c = 8 ksi):



Walls 3,4,5
Supporting Floors 5-PH (f'c = 5 ksi)



Walls D,E
Supporting Floors 2-4 (f'c = 8 ksi):



Walls D,E
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).

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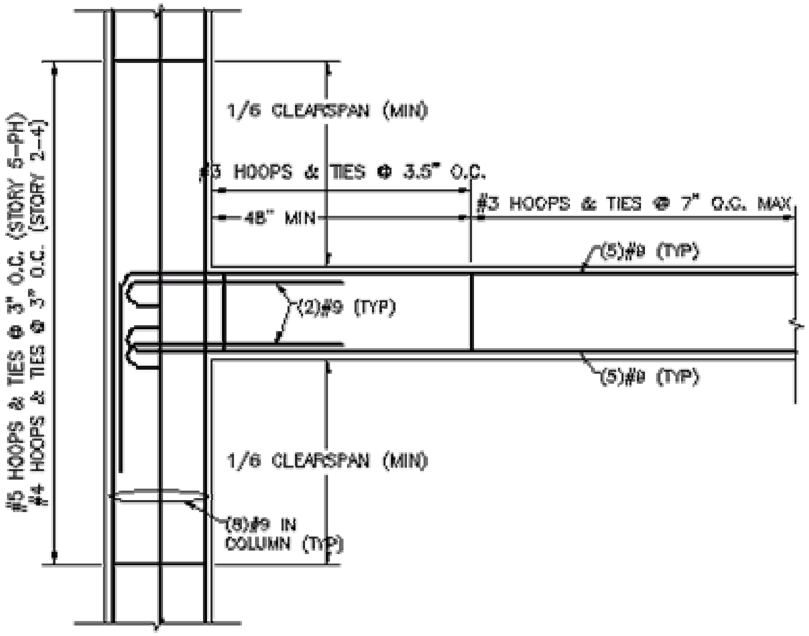
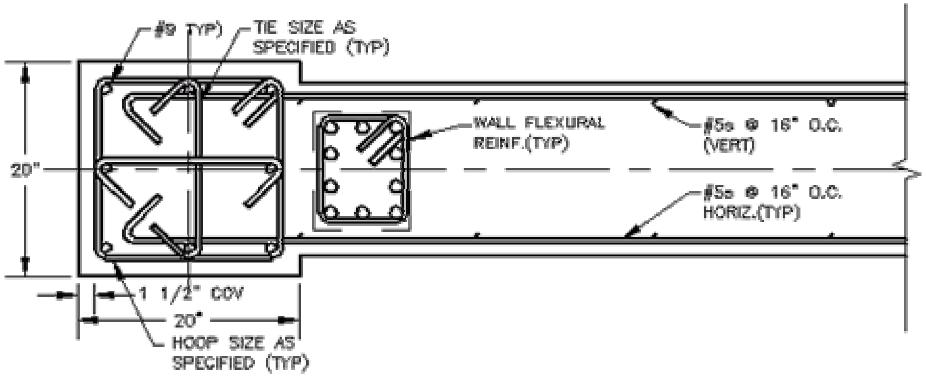
Lateral Structural System Design (Continued):

- Typical Details:





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Drifts:

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 0.020hx (2%), can be found in Appendix H.

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

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.

A New Building

Charleston, SC

Construction Schedule & Cost Impact:

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.

A New Building

Charleston, SC

Construction Schedule & Cost Impact (Continued):

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 New Building

Charleston, SC

Conclusions:

This project was an excellent exercise in structural design.

Original Expectation:

- Design controlled by seismic forces

Incorporated Topics:

- Proper usage of computer modeling software (ETABS)
- Dynamic analysis
- Reinforced concrete design
- Composite steel & concrete design
- Earthquake resistant design

Outcomes:

- Design controlled by drift limitations (Wind)
- Strength controlled by wind forces in lower floors, seismic forces in upper floors.
- Thicknesses of 16” & 20” in each or the orthogonal directions
- Design could be used along much of the east coast
- Concrete reinforcement detailing mostly prescriptive
    (can see reasoning for reinforcement requirements in many pictures from the recent earthquake in Haiti)

A very large Thank you to the people at Columbia Development Companies

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And finally, a HUGE thank you goes to my wife, Courtney. Without her support and perseverance, none of this would have been possible.

Questions