THE FORENSIC MEDICAL CENTER

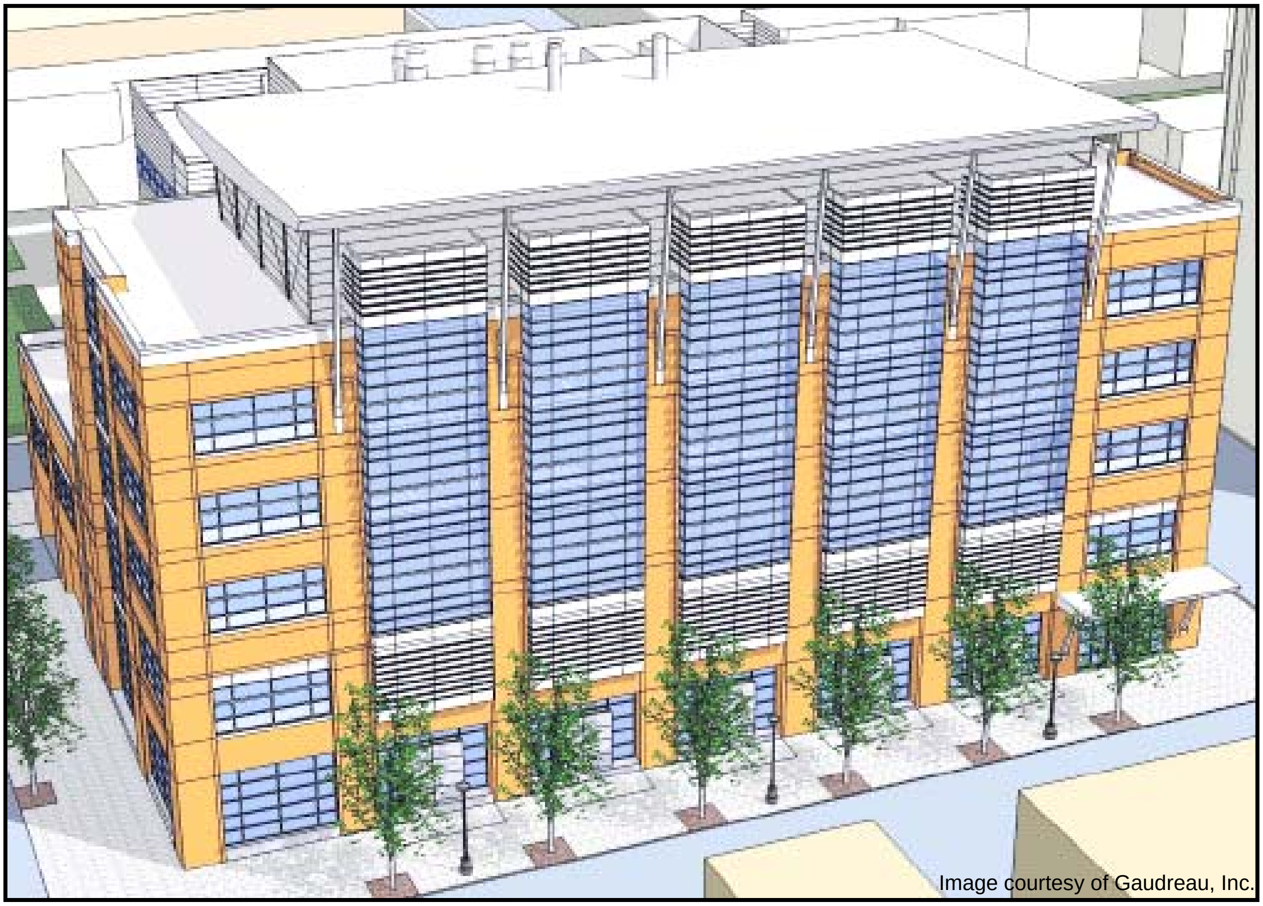

Image courtesy of Gaudreau, Inc.

TECHNICAL REPORT #1
OCTOBER 5, 2007

KEENAN YOHE
STRUCTURAL OPTION

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EXECUTIVE SUMMARY


Image courtesy of Gaudreau, Inc.

Technical Report 1 is a Structural Existing Conditions report. Its purpose is to describe and analyze the structure of the Forensic Medical Center as designed by Gaudreau, Inc. – Architect, and Hope Furrer Associates – Structural Engineer.

Included in this report is a description of the structural systems of the building, including framing, lateral load resisting systems, and material types and strengths. Some plans and elevations are provided for clarification. Loads are calculated according to ASCE 7-05, including wind and seismic loads. The analytical method was used to calculate wind loads, and the equivalent force method was used for seismic loads.

Spot checks of several structural elements are also included in this report. Strength reduction factors and load cases were used according to Load and Resistance Factor Design (LRFD) in ASCE 7-05 and ACI 318-05. Checks include a two-way slab, an interior column, a shearwall, and punching shear at a column-slab connection.

All of the elements checked for this report were found to be adequate. Detailed calculations for each element are included in the Appendix at the end of the report.

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STRUCTURAL SYSTEM DETAILS

Foundation

The foundation of the Forensic Medical Center consists of drilled piers under each column and under the shearwalls, as recommended by the geotechnical report prepared by T.L.B. Associates, dated April 18, 2007. The drilled piers are typically 48” in diameter, with 54” diameter piers under the shearwalls. The concrete strength is 3500 psi, with 12 #11 bars for the top third of the depth. The piers extend from the first level elevation of 76.25’ Mean Sea Level (MSL) to roughly 25’ MSL, where they are socketed into bedrock.

Columns

All of the columns in the building are normal weight concrete with a strength of 5000 psi. Typically, the columns are 24” by 24”, except for ten 34” diameter circular columns in the parking garage area. Exterior columns are reinforced with eight #8 bars and #4 ties at 12” on center. Interior columns are reinforced with eight #10 bars and #4 ties at 12” on center.

Forty columns span from the foundation to the Penthouse level floor, a total of 82 feet. Four columns span 38 feet from the foundation to the low roof at the third level slab. Also at the third level, two columns are shifted 7 feet towards the center of the building (Columns D-2 and D-7). The new columns continue to the Penthouse level floor slab. At the third level, they rest on 36” by 36” transfer beams which span between the two adjacent columns (D-2 to D-3; D-6 to D-7). These transfer beams are reinforced with ten #11 bars on the top and ten #9 bars on the bottom, tied by double #4 closed stirrups at 4’ on center.

Slabs

The ground floor parking garage level of the Forensic Medical Center is a 6” thick, normal weight concrete slab-on-grade, reinforced with #5 bars at 12” on-center each way. Concrete strength is 3500 psi. At the edges of this slab are concrete grade beams that are 30”-36” deep, with concrete strength of 3000 psi. The grade beams are reinforced with four #8 bars, five #9 bars, or five #10 bars, and #4 stirrups at 12” on center, depending on location.

The floor systems of levels two through five are typically 11” thick, two-way, flat-plate, normal weight concrete slabs with 26” wide by 36” deep concrete perimeter beams, reinforced with five #10 bars typically, with #4 stirrups at 8” on center. Slab reinforcement is typically #5 bars at 15” on center, each way, top and bottom at mid-span, with heavier reinforcement at the columns. Typical slab spans range from 22’-6” to 30’-0”.

Level two contains large recessed slab areas for body storage coolers and freezers. The finished floor elevation of these slabs is 10” lower than the typical finished floor elevation. These slabs are 11” thick, one-way slabs, and are supported by monolithically-poured concrete beams with sizes ranging from 18” to 40” wide by 11” to 26” deep.

Aside from the typical 11" two-way slab, level three also has two 9" thick, two-way slab sections that serve as low roofs. A high-density file storage area requires two 24"x18" concrete beams under the mid-span of the slabs, between grid lines 3 and 4.

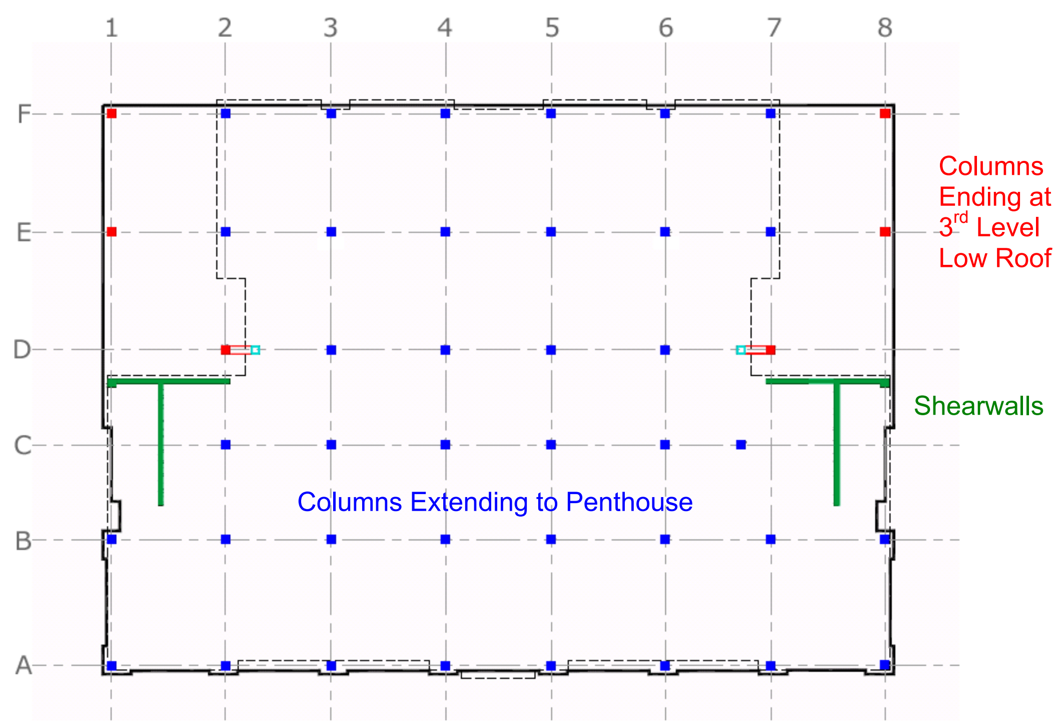
The Penthouse level floor slab consists of two areas. The roof areas are an 8" thick, two-way, flat-plate, normal weight concrete slab with #5 bars, typically spaced at 16", each way, top and bottom for reinforcement. The slab under the mechanical equipment is increased to 15" thick, with #5 bars at 11" each way, top and bottom, for typical reinforcement.

Lateral System

In addition to a moment frame consisting of the columns and slabs, the lateral force resisting system of the Forensic Medical Center is also made up of four 12" thick concrete shearwalls with a strength of 5000 psi, spanning from the foundation to the Penthouse level floor slab. These shearwalls are in two "T" configurations around the main stairwells which are located on the east and west sides of the building.

Shearwalls 1 and 4 are oriented east-west, and are tied to an exterior column. On the interior side of these walls is a 4'-6" boundary element containing 12 #9 bars for vertical reinforcement, with #4 ties at 12" on center. The webs of these walls contain the minimum amount of reinforcement for $\rho = 0.0025$, which is #5 bars at 18" on center each way, in each face.

Shearwalls 2 and 3 are oriented north-south. At both ends of these walls are 6'-0" boundary elements with 14 #9 bars for vertical reinforcement and #4 ties at 12" on center. The webs of these walls also contain the minimum amount of reinforcement, #5 bars at 18" on center each way, in each face.



CODES

Building Code

International Building Code – IBC 2006

Structural Concrete Code

American Concrete Institute – ACI 318-05
Aggregate: ACI 304
Slump: ACI 211.1
Reinforcing Steel: ASTM A615 Grade 60

GRAVITY LOADS

Dead Load

11” N.W.C. Slab:	138	psf
8” N.W.C. Roof Slab:	100	psf
9” N.W.C. Roof Slab:	113	psf
15” N.W.C. Roof Slab:	188	psf
Roofing/Insulation:	20	psf
Mech./Elec/Light:	8	psf

Equipment Loads

	Weight(k)	Size (in.)		9” pad	Total Load
		L	W		
Boilers:	39.6	160	68	113	637 psf
Generator:	25.5	223	60	113	387 psf
Chillers:	20.6	434	89	113	190 psf
AHUs:	12	190	75	113	234 psf
Vents:	10.3	180	72	113	227 psf
Tanks:	20	120	60	113	513 psf

Live Load

Operating Rooms/Labs:	60	psf
Partitions:	20	psf
Corridors above level 1:	80	psf
Stairs:	100	psf
Roof:	30	psf
High Density Storage:	250	psf
Mechanical Mezzanine:	40	psf

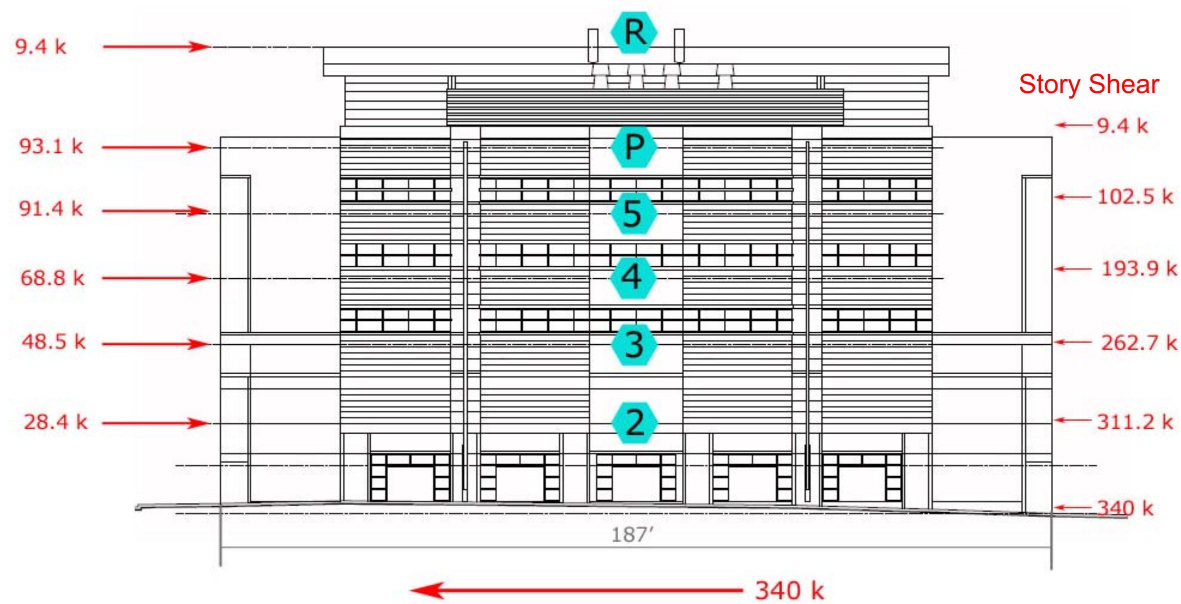
Snow Load

Ground Snow Load:	P_g=	25	psf
Exposure (B) Factor:	C_e=	1.0	
Importance Factor:	I_s=	1.2	psf
Thermal Factor:	C_t=	1.0	psf
$P_f = 0.7\ C_e\ C_t\ I_s\ P_g$ =		21	psf – Flat Roof Load

GOVERNING LATERAL LOADS

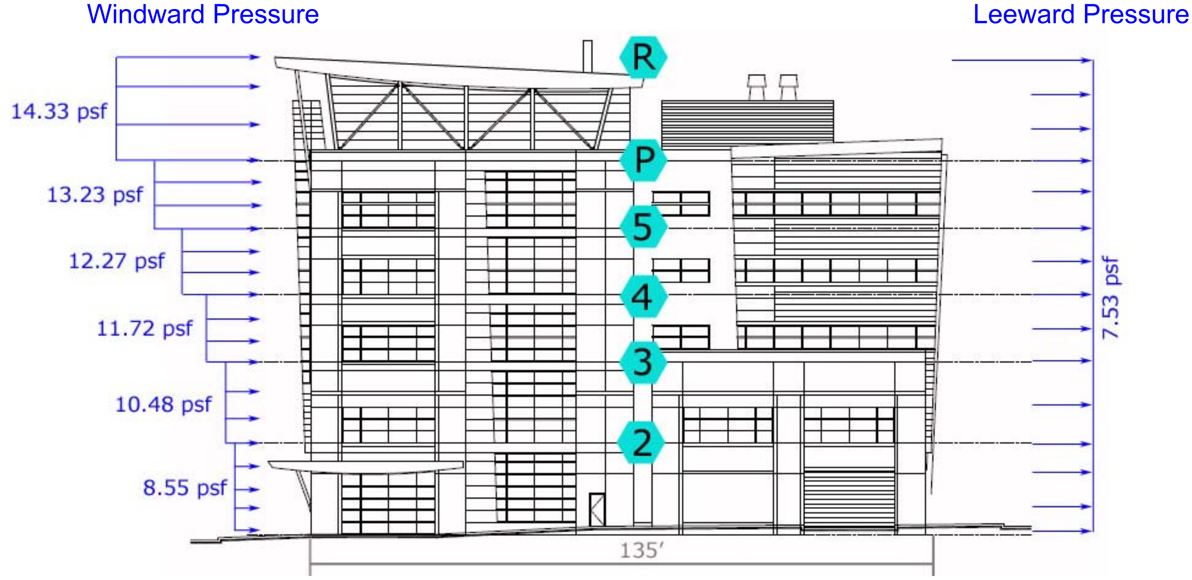
The design seismic and wind forces on the building are similar in magnitude. Seismic forces were found to control the design in the east-west direction. However, because the building has a larger area facing north-south, wind forces govern in that direction.

Seismic Forces



Overturning Moment = 20810 ft-k

North-South Wind Pressures



Overturning Moment = 21140 ft-k

Load calculations and these and other cases can be found in the Appendix at the end of this report.

DESIGN CHECKS

Two-Way Slab

Because of the regular grid layout of this building, the two-way slab system can be analyzed using the Direct Design method. A section of the fourth level, from gridlines B to E and gridlines 3 to 6, was chosen for analysis. This area of the slab meets the requirements for Direct Design:

- At least three continuous spans in each direction
- Panel aspect ratio less than 2
- Column Offset less than 10% of span length
- Live Load less than twice Dead Load

Another requirement of Direct Design is that the slab must only be analyzed for gravity loading. Therefore, wind and seismic loads are neglected in this calculation.

The results of these calculations are as follows:

Frame X:

		ΦM_n (ft-k/ft)		M_u (ft-k/ft)	
Negative Moment	Column Strip	26.9	>	20.8	√
	Middle Strip	11.1	>	7.2	√
Positive Moment	Column Strip	11.1	>	8.9	√
	Middle Strip	11.1	>	6.2	√

Frame Y:

		ΦM_n (ft-k/ft)		M_u (ft-k/ft)	
Negative Moment	Column Strip	20.7	>	19.8	√
	Middle Strip	10.0	>	6.3	√
Positive Moment	Column Strip	10.0	>	8.9	√
	Middle Strip	10.0	>	5.4	√

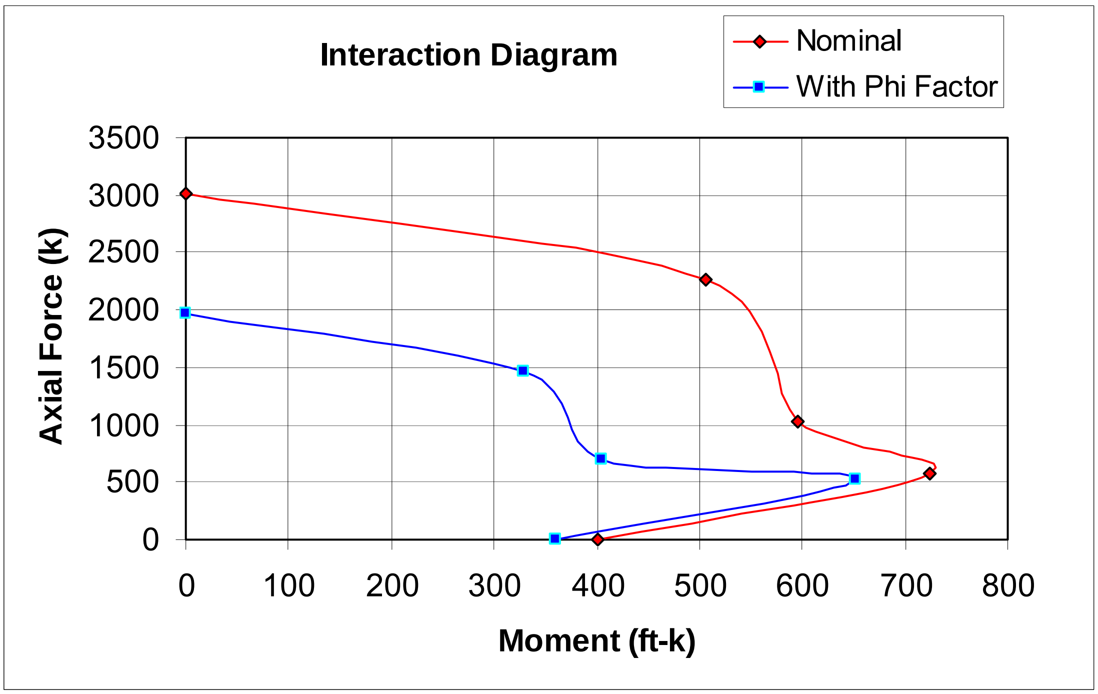
Calculations can be found in the Appendix immediately following this report.

Column

Column E-3 at the first floor level was selected for analysis. The column is 24” by 24” normal weight concrete (f'_c = 5000 psi), with eight #10 bars for reinforcing. Cover is required to be 1 ½” all around, so the distance from the center of a bar to the face of the column was conservatively estimated to be 2 ¾”.

An interaction diagram was constructed from five points:

- Pure Axial Strength
- Balanced Strain
- Pure Bending
- c = h (= 24”)
- $\varepsilon_t = \varepsilon_{s3} = 0.005$



The axial load on the column was calculated from the tributary area on each floor. Even though there are shearwalls to take some of the lateral loads, the column was analyzed by conservatively assuming the concrete moment frames take the entire lateral load. The loads were divided equally among the frames in each direction (eight in the north-south direction, six in the east-west). The frames were then analyzed using the portal method to find the worst-case moment in the column at the first floor level.

The ultimate loads, when plotted on the interaction diagram, fell within the curve. Therefore, the design is satisfactory. Calculations can be found in the Appendix at the end of this report.

Punching Shear

Since all of the slabs in the Forensic Medical Center are flat-plate slabs, there are no drop panels to help counteract punching shear at the columns. The second level floor slab was selected for analysis at column E-3.

The tributary area of column E-3 is 728 ft^2. The slab is 11" thick, with an effective depth of 10", forming a critical section that is 34" square around the column.

The ultimate moment on the column was found using the same method described under the *Column* design check section, while the ultimate shear was found using the tributary area of the column. The polar moment of inertia of the critical section (J_c) was calculated in order to find the ultimate shear stress (v_u). The three equations of ACI 318 11.12.2.1 were used to find the shear capacity of the critical section, which was then divided by the area of the section to get the maximum nominal shear stress (v_n). A Φ-factor of 0.75 was applied.

$$\Phi v_n = 212 \text{ psi} > 176 \text{ psi} = v_u \ \surd$$

Calculations can be found in the Appendix at the end of this report.

Shearwall

Shearwall #3 was selected for lateral load analysis. It is oriented in the north-south direction, and located next to the west stairwell of the building. It is 82' tall and 12" thick. Its 30' length is divided into an 18' web with minimum reinforcement (#5 bars at 18" on center each way, each face), and two more heavily reinforced 6' boundary elements (seven #9 bars on each face).

Since wind forces control in the north-south direction, they were used as the lateral load in this analysis. The shearwalls were conservatively assumed to take the entire lateral load (the concrete moment frame was neglected). Using this assumption, shearwalls #2 and #3 are the only north-south lateral force resisting elements, and they are the same length, so the lateral loads were divided evenly between them. The unfactored moment at the base of the wall was found to be 8073 ft-k, and the unfactored base shear was found to be 151.1 k.

The simplified method was used to analyze the shearwall, with one boundary element in tension and the other in compression, forming a force couple separated by a distance z = 24'. One boundary element was then analyzed for compression, and the other for tension. The ultimate compression force was found to be 981 k, and the ultimate tension force was 538 k. The compression strength was found to be adequate at 2315 k. The tension strength was also adequate at 756 k.

The minimum shear strength of the concrete at the base of the shearwall was calculated to be 331 k. This is adequate for the factored load of 242 k, but is less than $2V_u$, therefore, shear reinforcement is required in accordance with ACI 318 11.10.9. The web reinforcement provided was found to be adequate in both the longitudinal and transverse directions.

Calculations are provided in the Appendix at the end of this report.

APPENDIX

WIND LOAD CALCULATIONS

$V =$ 90 mph $I =$ 1.15
$K_{zt} =$ 1.0 $K_d =$ 0.85

$g_a = g_v = 3.4$ $Q = \sqrt{(1/(1+0.63((B+h)/Lz)^{0.63}))}$ $G = 0.925 \dfrac{(1+1.7 g_a I_z Q)}{(1+1.7 g_v I_z)}$

$I_z = c * (33/z)^{(1/6)} = 0.18$
$L_z = l * (z/33)^{\varepsilon} = 569$
$c = 0.2$ $z = 63$ $\varepsilon = 0.2$

$Q_{N-S} =$	0.841
$Q_{E-W} =$	0.856

$G_{N-S} =$	0.85
$G_{E-W} =$	0.86

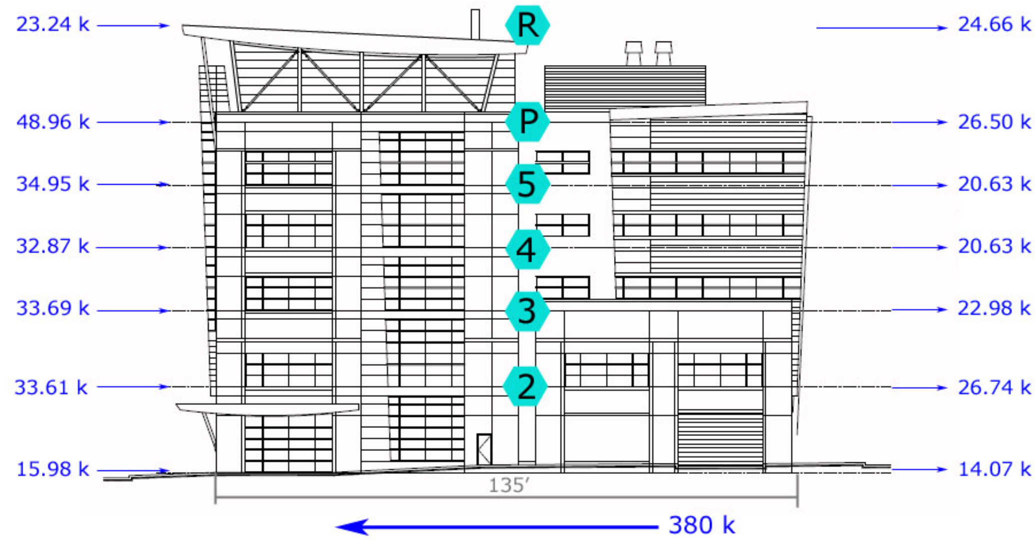
$q = 0.00256 * K_z K_d V^2 I$

$p = qGC_p$

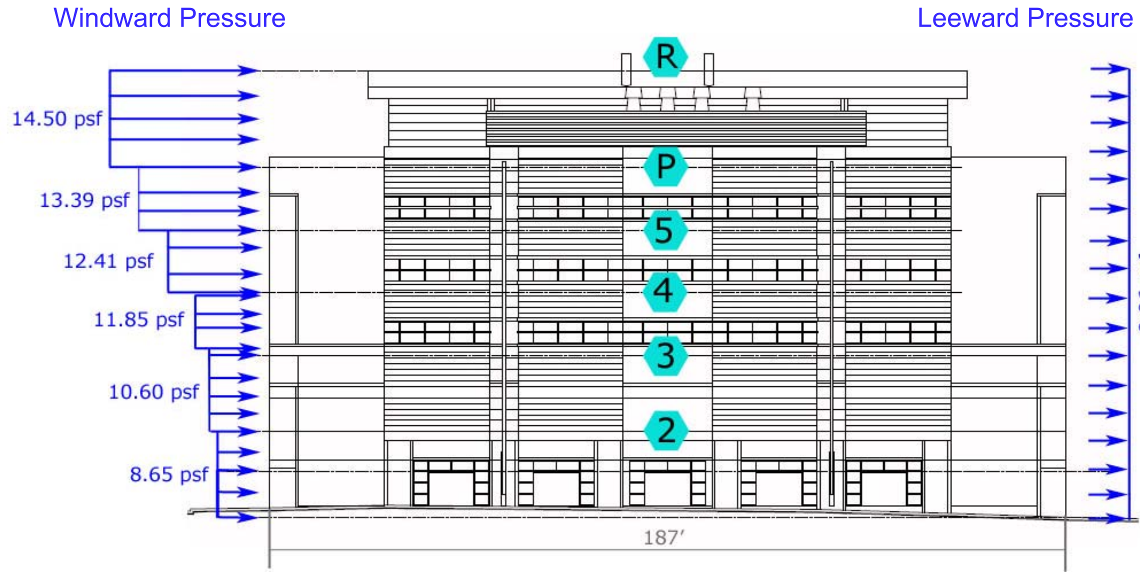
STORY	Start Height (ft.)	End Height (ft.)	Kz (Table 6-3)	q	Wind Pressures (psf) N-S WW	N-S LW	E-W WW	E-W LW
				Cp	0.8	-0.42	0.8	-0.5
Penthouse	82	105	1.04	21.08	14.33	-7.53	14.50	-9.06
5	67.33	82	0.96	19.46	13.23	-7.53	13.39	-9.06
4	52.67	67.33	0.89	18.04	12.27	-7.53	12.41	-9.06
3	38	52.67	0.85	17.23	11.72	-7.53	11.85	-9.06
2	20	38	0.76	15.40	10.48	-7.53	10.60	-9.06
1	0	20	0.62	12.57	8.55	-7.53	8.65	-9.06

LEVEL	Trib. Height (ft.)	Wind Forces (k) N-S Width (ft.)	N-S WW	N-S LW	E-W Width (ft.)	E-W WW	E-W LW
Roof	11.5	141	23.24	-24.66	122	20.35	-12.72
Penthouse	11.5 7.33	187	48.96	-26.50	135	35.76	-23.04
5	7.33 7.33	187	34.95	-20.63	135	25.53	-17.94
4	7.33 7.33	187	32.87	-20.63	135	24.01	-17.94
3	7.33 9	187	33.69	-22.98	135	24.61	-19.98
2	9 10	187	33.61	-26.74	135	24.55	-23.25
1	10	187	15.98	-14.07	135	11.67	-12.24

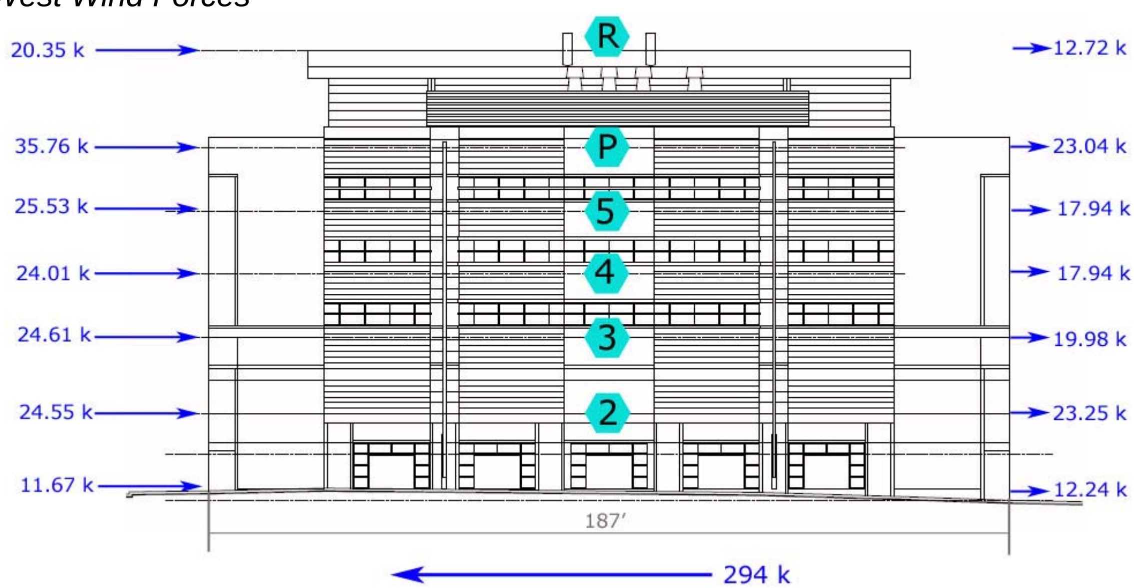
North-South Wind Forces



East-West Wind Pressures



East-West Wind Forces



SEISMIC LOAD CALCULATIONS

Seismic Weight

Level 2

Slab:	150 pcf	*	11 in thick	*	25368 sq ft			=	**3488**	k
Ext. Wall:	44 psf	*	644 ft perimeter	*	19 ft height			=	**538**	k
Partition:	20 psf			*	25368 sq ft			=	**507**	k
Columns:	150 pcf	*	4 sq ft	*	19 ft height	*	44	=	**502**	k
Storage:	250 psf		0 sq ft	*	25 %			=	**0**	k
Roof:	150 pcf	*	in thick	*	0 sq ft			=	**0**	k
								TOTAL =	**5035**	**k**

Level 3

Slab:	150 pcf	*	11 in thick	*	18547 sq ft			=	**2550**	k
Ext. Wall:	44 psf	*	644 ft perimeter	*	16.33 ft height			=	**463**	k
Partition:	20 psf			*	18547 sq ft			=	**371**	k
Columns:	150 pcf	*	4 sq ft	*	16.33 ft height	*	42	=	**412**	k
Storage:	250 psf	*	800 sq ft	*	25 %			=	**50**	k
Roof:	150 pcf	*	9 in thick	*	3500 sq ft			=	**394**	k
								TOTAL =	**4239**	**k**

Level 4

Slab:	150 pcf	*	11 in thick	*	21700 sq ft			=	**2984**	k
Ext. Wall:	44 psf	*	644 ft perimeter	*	14.67 ft height			=	**416**	k
Partition:	20 psf			*	21700 sq ft			=	**434**	k
Columns:	150 pcf	*	4 sq ft	*	14.67 ft height	*	42	=	**370**	k
Storage:	250 psf	*	0 sq ft	*	25 %			=	**0**	k
Roof:	150 pcf	*	in thick	*	0 sq ft			=	**0**	k
								TOTAL =	**4203**	**k**

Level 5

Slab:	150 pcf	*	11 in thick	*	21810 sq ft			=	**2999**	k
Ext. Wall:	44 psf	*	644 ft perimeter	*	14.67 ft height			=	**416**	k
Partition:	20 psf			*	21810 sq ft			=	**436**	k
Columns:	150 pcf	*	4 sq ft	*	14.67 ft height	*	42	=	**370**	k
Storage:	250 psf	*	600 sq ft	*	25 %			=	**38**	k
Roof:	150 pcf	*	in thick	*	0 sq ft			=	**0**	k
								TOTAL =	**4258**	**k**

Penthouse

Slab:	150 pcf	*	15 in thick	*	8400 sq ft			=	**1575**	k
Ext. Wall:	44 psf	*	644 ft perimeter	*	7.33 ft height			=	**208**	k
Partition:	psf			*	sq ft			=	**0**	k
Columns:	150 pcf	*	4 sq ft	*	7.33 ft height	*	42	=	**185**	k
Equip:					165 k			=	**165**	k
Roof:	150 pcf	*	8 in thick	*	13600 sq ft			=	**1360**	k
								TOTAL =	**3492**	**k**

Roof

Framing:	10 psf	*	10000 sq ft	=	**100**	**k**
Roofing:	17 psf	*	10000 sq ft	=	**170**	**k**
				TOTAL =	**270**	**k**

W = 21498 k

Seismic Force

Site Class C

S_s = 0	F_a = 1.2	$S_{DS} = (2/3)F_aS_s$	= 0.14
S_1 = 0	F_v = 1.7	$S_{D1} = (2/3)F_vS_1$	= 0.06

R = 5 For ordinary reinf. conc. shearwalls
I = 1.5 For Occupancy Type IV

T_L	=	6 s	ASCE 7 Fig. 22-15
$T_a = C_th_n\hat{}$	=	0.656 s	ASCE 7 Table 12.8-2
$T = C_uT_a$	=	1.115 s	(Cu = 1.7 - ASCE 7 Table 12.8-1)

C_s = MIN

$S_{DS}/(R/I)$	=	0.04056	
$S_{D1}/[T(R/I)]$	=	0.015549	<- CONTROLS
$S_{D1}T_L/T^2(R/I)$	=	0.083656	

$$C_{vx} = \frac{w_xh_x^k}{\Sigma\, w_ih_i^k}$$

$\boxed{k = 1.1}$
(interpolated)

k = 1 if T<1.0s
k = 2 if T>2.5s

$\boxed{V = C_s * W}$ $\boxed{C_s = 0.0155}$

$\boxed{F_x = C_{vx} * V}$

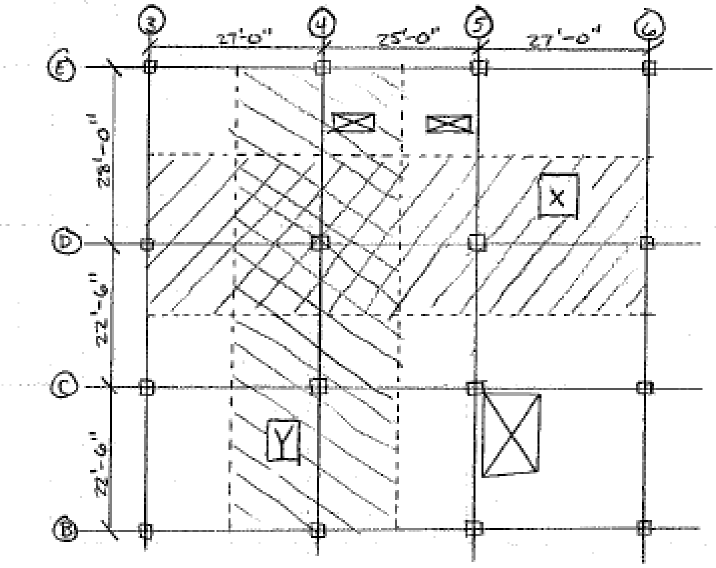
V = 339.7 k

	h_x	$w_xh_x^k$
Level 2	20	135885
Level 3	38	231761
Level 4	53	329073
Level 5	67	436747
Penthouse	82	444961
Roof	105	45151
	Σ =	1623577

	Cvx	Fx
Level 2	0.0837	28.4 k
Level 3	0.1427	48.5 k
Level 4	0.2027	68.8 k
Level 5	0.2690	91.4 k
Penthouse	0.2741	93.1 k
Roof	0.0278	9.4 k
Σ =	1.0000	339.7 k

TWO-WAY SLAB CALCULATIONS

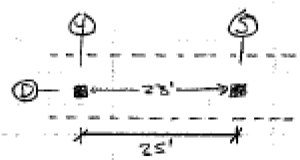
FOURTH FLOOR – TWO-WAY SLAB – DIRECT DESIGN



w_D = 138 psf 11" CONC. SLAB
8 psf MECH/ELEC.
146 psf

w_L = 60 psf LABORATORY
20 psf PARTITION
80 psf

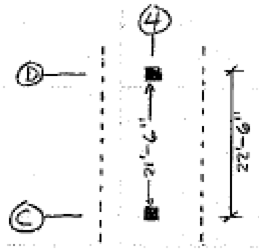
w_u =
1.2D + 1.6L
= 1.2(146) + 1.6(80)
= 303 psf



FRAME $\boxed{X}$ – SPAN 4–5
STATIC MOMENT

$$M_o = \frac{w_u \ell_2 \ell_n^2}{8} = \frac{(0.303\text{ k/ft}^2)\left(\frac{22'-6" + 28'-0"}{2}\right)(23'-0")^2}{8}$$

M_o = 506 ft·k



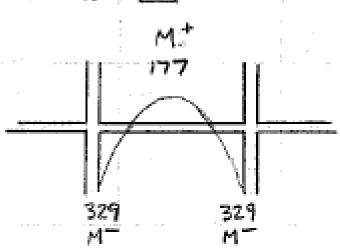
FRAME $\boxed{Y}$ – SPAN C–D
STATIC MOMENT

$$M_o = \frac{w_u \ell_2 \ell_n^2}{8} = \frac{(0.303\text{ k/ft}^2)\left(\frac{27'-0 + 25'-0"}{2}\right)(21'-6)^2}{8}$$

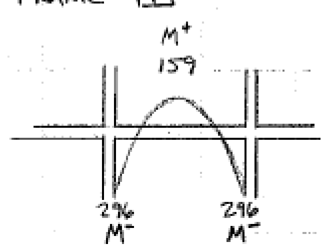
M_o = 455 ft·k

IN AN INTERIOR SPAN, M_o IS DISTRIBUTED: <ACI 318 13.6.3.2>
NEGATIVE FACTORED MOMENT........ $0.65 M_o$
POSITIVE FACTORED MOMENT......... $0.35 M_o$

FRAME $\boxed{X}$:



FRAME $\boxed{Y}$:



– NO BEAMS ⇒ $\alpha_f = 0$, $\alpha_f \frac{\ell_2}{\ell_1} = 0$

∴ COLUMN STRIPS TAKE 75% OF M^-
60% OF M^+

FRAME $\boxed{X}$:

COLUMN STRIP: $0.75M^- = -247$ ft·k
$0.60M^+ = 106$ ft·k

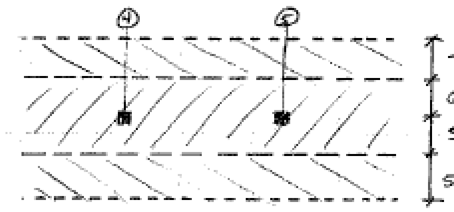
½ MIDDLE STRIP: $0.25M^- = -82$ ft·k
$0.40M^+ = 71$ ft·k

COLUMN STRIP WIDTH = 0.25(22'-6") = 5.6' + 0.25(25'-0") = 6.3'
½ MIDDLE STRIP WIDTH = (22'-6"/2) – 5.6' = 5.7' (28'/2) – 6.3' = 7.7'

TOP ½ MIDDLE

COLUMN

BOTTOM ½ MIDDLE



TOP ½ MIDDLE : M^-: $\left(\frac{-82}{2}\right)/7.7' = -5.3$ ft·k/ft M^+: $\left(\frac{71}{2}\right)/7.7' = 4.6$ ft·k/ft

COLUMN: M^-: $-247/11.9' = -20.8$ ft·k/ft M^+: $106/11.9' = 8.9$ ft·k/ft

BOTTOM ½ MIDDLE: M^-: $\left(\frac{-82}{2}\right)/5.7' = -7.2$ ft·k/ft M^+: $\left(\frac{71}{2}\right)/5.7' = 6.2$ ft·k/ft

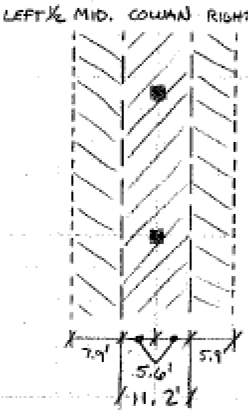
FRAME $\boxed{Y}$:

COLUMN STRIP: $0.75M^- = -222$ ft·k
$0.60M^+ = 95$ ft·k

½ MIDDLE STRIP: $0.25M^- = -74$ ft·k
$0.40M^+ = 64$ ft·k

COLUMN STRIP WIDTH = 0.25(22'-6") = 5.6' + 0.25(22'-6") = 5.6'
½ MIDDLE STRIP WIDTH = (27'/2) – 5.6' = 7.9' (25'/2) – 5.6' = 5.9'

LEFT ½ MID. COLUMN RIGHT ½ MID.

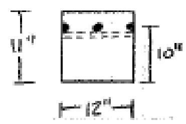


LEFT ½ MIDDLE: M^-: $\left(\frac{-74}{2}\right)/7.9' = -4.7$ ft·k/ft
M^+: $\left(\frac{64}{2}\right)/7.9' = 4.1$ ft·k/ft

COLUMN: M^-: $-222/11.2' = -19.8$ ft·k/ft
M^+: $95/11.2' = 8.5$ ft·k/ft

RIGHT ½ MIDDLE: M^-: $\left(\frac{-74}{2}\right)/5.9' = -6.3$ ft·k/ft
M^+: $\left(\frac{64}{2}\right)/5.9' = 5.4$ ft·k/ft

FRAME $\boxed{X}$: NEGATIVE MOMENT @ COLUMN STRIP : $M_u = -20.8$ ft·k/ft
SPAN ④–⑤ <AS DESIGNED: 11" THICK SLAB, #5 BARS @ 6">



$f'_c = 5000$ psi
$f_y = 60000$ psi

$A_s = 2 \times 0.31 = 0.62$ in²

$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{(0.62)(60000)}{0.85(5000)(12)} = 0.73"$

$c = a/\beta_1 = 0.73/0.8 = 0.91"$

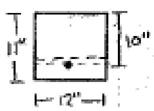
$\varepsilon_t = 0.003 \frac{d-c}{c} = 0.03 > 0.005$ ∴ $\phi = 0.9$

$M_n = A_s f_y (d - \frac{a}{2}) = (0.62)(60)(10 - \frac{0.73}{2})$

$M_n = 358$ in·k $= 29.9$ ft·k

$\phi M_n = 26.9$ ft·k $> M_u = 20.8$ ft·k <u>OK</u>

FRAME $\boxed{X}$: POSITIVE MOMENT @ COLUMN STRIP: $M_u = 8.9$ ft·k/ft
SPAN ④–⑤ <AS DESIGNED: 11" THICK SLAB, #5 BARS @ 15">



$f'_c = 5000$ psi
$f_y = 60000$ psi

$A_s = \frac{12"}{15"} \times 0.31 = 0.25$ in²

$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{(0.25)(60000)}{0.85(5000)(12)} = 0.29"$

$c = a/\beta_1 = 0.29/0.8 = 0.36"$

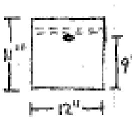
$\varepsilon_t = 0.003 \frac{d-c}{c} = 0.080 > 0.005$ <u>OK</u> $\phi = 0.9$

$M_n = A_s f_y (d - \frac{a}{2}) = (0.25)(60)(10 - \frac{0.29}{2})$

$M_n = 148$ in·k $= 12.3$ ft·k

$\phi M_n = 11.1$ ft·k $> M_u = 8.9$ ft·k <u>OK</u>

FRAME $\boxed{Y}$: NEGATIVE MOMENT @ WORST CASE MIDDLE STRIP: $M_u = -6.3$ ft·k/ft
<AS DESIGNED: 11" THICK SLAB, #5 BARS @ 15">



$f'_c = 5000$ psi
$f_y = 60000$ psi

$A_s = \frac{12"}{15"} \times 0.31 = 0.25$ in²

$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{(0.25)(60000)}{0.85(5000)(12)} = 0.29"$

$c = a/\beta_1 = 0.29/0.8 = 0.36"$

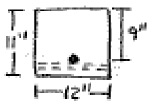
$\varepsilon_t = 0.003 \frac{d-c}{c} = 0.072 > 0.005$ <u>OK</u> $\phi = 0.9$

$M_n = A_s f_y (d - \frac{a}{2}) = (0.25)(60)(9 - \frac{0.29}{2})$

$M_n = 133$ in·k $= 11.1$ ft·k

$\phi M_n = 10.0$ ft·k $> M_u = 6.3$ ft·k <u>OK</u>

FRAME $\boxed{Y}$: POSITIVE MOMENT @ WORST CASE MIDDLE STRIP: $M_u = 5.4$ ft·k/ft
<AS DESIGNED: 11" THICK SLAB, #5 BARS @ 15">



$f'_c = 5000$ psi
$f_y = 60000$ psi

$A_s = \frac{12"}{15"} \times 0.31 = 0.25$ in²

$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{(0.25)(60000)}{0.85(5000)(12)} = 0.29"$

$c = a/\beta_1 = 0.29/0.8 = 0.36"$

$\varepsilon_t = 0.003 \frac{d-c}{c} = 0.072 > 0.005$ <u>OK</u> $\phi = 0.9$

$M_n = A_s f_y (d - \frac{a}{2}) = (0.25)(60)(9 - \frac{0.29}{2})$

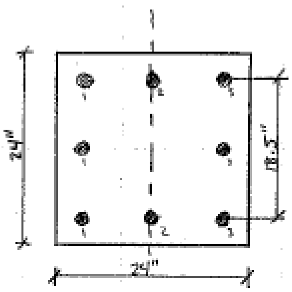
$M_n = 133$ in·k $= 11.1$ ft·k

$\phi M_n = 10.0$ ft·k $> M_u = 5.4$ ft·k <u>OK</u>

COLUMN CALCULATIONS

COLUMN E-3 — FIRST FLOOR

AS DESIGNED: TYPICAL INTERIOR COLUMN



$f'_c = 5000$ psi

(8) #10 BARS
#4 TIES @ 12" o.c.
$f_y = 60000$ psi

INTERACTION CURVE CALCULATION:

— PURE AXIAL STRENGTH:

$P_o = 0.85 f'_c A_c + A_s f_y = 0.85(5)(24^2 - 8(1.27)) + 60(8 \times 1.27)$

$P_o = 3014$ k

— BALANCED STRAIN:

$\varepsilon_y = 60 \text{ ksi} / 29000 \text{ ksi} = 0.0021 \qquad c = \frac{0.003}{0.003 + 0.0021}(21.25) = 12.5''$

$\varepsilon_{s_1} = \frac{0.003}{12.5}(12.5 - 2.75) = 0.00234 \qquad f_{s_1} = 60$ ksi

$\varepsilon_{s_2} = \frac{0.003}{12.5}(12.5 - 12) = 0.00012 \qquad f_{s_2} = \varepsilon_{s_2} \times E_s = 3.5$ ksi

$\varepsilon_{s_3} = \frac{0.003}{12.5}(12.5 - 21.25) = -0.0021 \qquad f_{s_3} = -60$ ksi

$P_b = 0.85 f'_c b \beta_1 c + 3f_{s_1} + 2f_{s_2} + 3f_{s_3}$
$= 0.85(5)(24)(0.8)(12.5) + 3(60) + 2(3.5) + 3(-60) = 1027$ k

$M_b = 0.85 f'_c b \beta_1 c \left(\frac{w}{2} - \frac{\beta_1 c}{2}\right)$
$= 0.85(5)(24)(0.8)(12.5)\left(12 - \frac{0.8 \times 12.5}{2}\right) = 595$ ft·k

— PURE BENDING:

$f_{s_1} = \frac{0.003}{c}(c - 2.75)(29000) \qquad f_{s_2} = -60$ ksi $\qquad f_{s_3} = -60$ ksi

$\Sigma F = 0 = 0.85 f'_c b \beta_1 c + 3f_{s_1} + 2f_{s_2} + 3f_{s_3}$
$0 = 0.85(5)(24)(0.8)c + 3\left[\frac{0.003}{c}(c - 2.75)(29000)\right] + 2(-60) + 3(-60)$

$c = 3.21''$ $\to f_{s_1} = 12$ ksi OK $\qquad \varepsilon_{s_2} = \frac{0.003}{3.21}(3.21 - 12) = -0.008$ OK

$\varepsilon_{s_3} = \frac{0.003}{3.21}(3.21 - 21.25) = -0.017$ OK

$M_o = 0.85 f'_c b \beta_1 c \left(\frac{w}{2} - \frac{\beta_1 c}{2}\right) + 3f_{s_1}(12 - 2.75) + 2f_{s_2}(12 - 12) + 3f_{s_3}(12 - 21.25)$
$= 0.85(5)(24)(0.8)(3.21)\left(\frac{24}{2} - \frac{0.8 \times 3.21}{2}\right) + 3(12)(9.25) + 0 + 3(-60)(-9.25)$
$= 400$ ft·k

— $c = w \quad (= 24'')$

$\varepsilon_{s_1} = \frac{0.003}{24}(24 - 2.75) = 0.0027 \qquad f_{s_1} = E_s \varepsilon_{s_1} \Rightarrow 60 \text{ ksi}$

$\varepsilon_{s_2} = \frac{0.003}{24}(24 - 12) = 0.0015 \qquad f_{s_2} = E_s \varepsilon_{s_2} = 44 \text{ ksi}$

$\varepsilon_{s_3} = \frac{0.003}{24}(24 - 21.25) = 0.00084 \qquad f_{s_3} = E_s \varepsilon_{s_3} = 10 \text{ ksi}$

$P_n = 0.85 f'_c b \beta_1 c + 3f_{s_1} + 2f_{s_2} + 3f_{s_3}$
$= 0.85(5)(24)(0.8)(24) + 3(60) + 2(44) + 3(10) = 2256 \text{ k}$

$M_n = 0.85 f'_c b \beta_1 c \left(\frac{w}{2} - \frac{\beta_1 c}{2}\right) + 3f_{s_1}(12 - 2.75) + 2f_{s_2}(12-12) + 3f_{s_3}(12 - 21.25)$
$= 0.85(5)(24)(0.8)(24)\left(12 - \frac{0.8 \times 24}{2}\right) + 3(60)(9.25) + 0 + 3(10)(-9.25)$
$= 507 \text{ ft}\cdot\text{k}$

— $\varepsilon_t = 0.005 \quad (\varepsilon_{s_3} = 0.005)$

$c = \frac{0.003}{0.003 + 0.005}(21.25) = 7.97''$

$\varepsilon_{s_1} = \frac{0.003}{7.97}(7.97 - 2.75) = 0.00196 \qquad f_{s_1} = E_s \varepsilon_{s_1} = 57 \text{ ksi}$

$\varepsilon_{s_2} = \frac{0.003}{7.97}(7.97 - 12) = -0.00152 \qquad f_{s_2} = E_s \varepsilon_{s_2} = -44 \text{ ksi}$

$P_n = 0.85(5)(24)(0.8)(7.97) + 3(57) + 2(-44) + 3(-50)$
$= 583 \text{ k}$

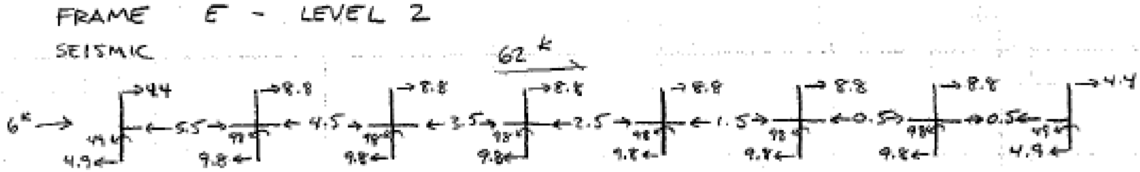
$M_n = 0.85(5)(24)(0.8)(7.97)\left(12 - \frac{0.8 \times 7.97}{2}\right) + 3(57)(12 - 2.75) + 0 + 3(-50)(12 - 21.25)$
$= 725 \text{ ft}\cdot\text{k}$

— INTERACTION:

P_n (k)	M_n (ft·k)	c (in)	$\varepsilon_t = \varepsilon_{s_3}$	ϕ
3014	0	∞		0.65
2256	507	24	0.00034	0.65
1027	595	12.5	−0.0021	0.68
583	725	7.97	−0.005	0.9
0	400	3.21	−0.017	0.9

FRAME E - LEVEL 2

SEISMIC



<98 ft·k MOMENT ON COLUMN 3>

Column E-3 - Loads

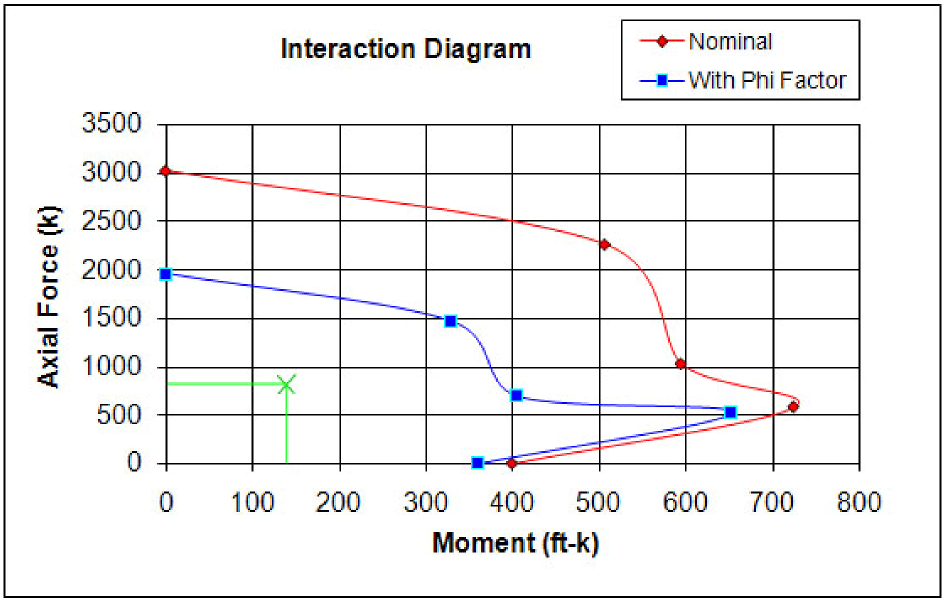
Load Case		
	1	1.2D + 1.6L + 0.5S
	2	1.2D + 1.6W + 1.0L + 0.5S
	3	1.2D + 1.0E + 1.0L + 0.2S

Gravity

Floor	Dead (psf)	Live (psf)	Snow (psf)	Trib. Width (ft) N-S	Trib. Width (ft) E-W	Trib. Area (ft^2)	Dead (k)	Live (k)	Snow (k)
Penthouse	150	30	21	28	26	728	109	9	15
5	146	80		28	26	728	106	23	
4	146	80		28	26	728	106	23	
3	146	80		28	26	728	106	23	
2	146	80		28	26	728	106	23	
					Total A_T =	3640	534	102	15
					K_{LL} =	4			
					LL Reduction Factor =	0.40			

	Case 1 (k)	Case 2 (k)	Case 3 (k)
Load @ Level 1 =	812	751	746

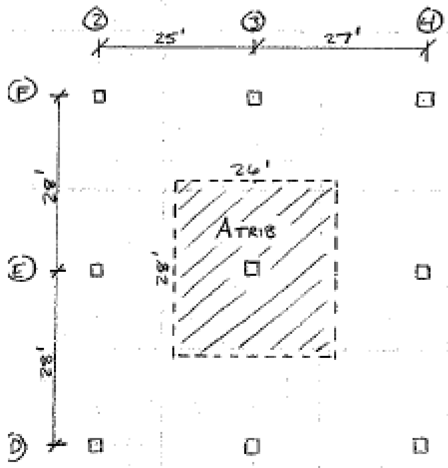
Lateral				Moments (ft-k)		
N-S	Wind	80 ft-k		0	128	0
	Seismic	86 ft-k		0	0	86
E-W	Wind	84 ft-k		0	134	0
	Seismic	98 ft-k		0	0	98

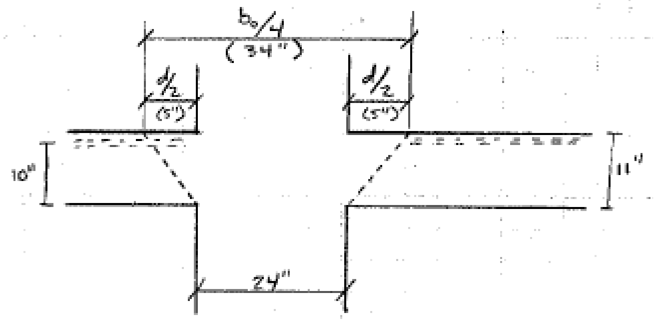


PUNCHING SHEAR CALCULATIONS

PUNCHING SHEAR – COLUMN E-3 – WORST CASE

$$V_c = \text{MIN}\begin{cases} 4\sqrt{f'_c}\, b_o d = 4\sqrt{5000}\,(136)(10) = 385\text{ k} \leftarrow \text{CONTROL} \\ \left(2+\frac{4}{\beta_c}\right)\sqrt{f'_c}\, b_o d = \left(2+\frac{4}{1}\right)\sqrt{5000}\,(136)(10) = 577\text{ k} \\ \left(\frac{\alpha_s d}{b_o}+2\right)\sqrt{f'_c}\, b_o d = \left(\frac{40\times10}{136}+2\right)\sqrt{5000}\,(136)(10) = 475\text{ k} \end{cases}$$





$A_{TRIB} = 26' \times 28' = 728\text{ ft}^2$

$w_u = 1.2D + 1.6W + 1.0L = 1.2(146\text{ psf}) + 1.0(80\text{ psf}) = 255\text{ psf}$

$M_u = 134\text{ ft}\cdot\text{k} \Rightarrow 40\%$ TRANSFERRED BY SHEAR $= M_{uv} = 54\text{ ft}\cdot\text{k}$

$J_c = \frac{2(10)(34)^3}{12} + \frac{2(34)(10)^3}{12} + 2(10)(34)\left(\frac{34}{2}\right)^2 = 267700\text{ in}^4$

$A_c = 2(10)\left[34+34\right] = 1360\text{ in}^2$

SHEAR STRESS: $v_u = \frac{V_u}{A_c} + \frac{M_{uv}c}{J_c}$, $V_u = 255\text{ psf} \times (728 - (34/12)^2)$

$V_u = 184\text{ k}$

$v_u = \frac{184}{1360} + \frac{(54\times12)(34/2)}{267700}$

$v_u = 0.176$

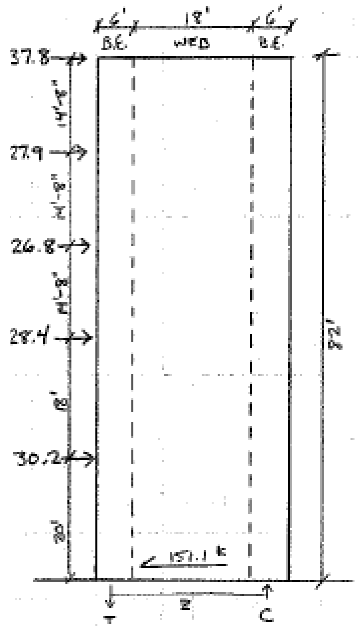
$v_n = V_n/b_o d = 385/(136\times10) = 0.283$

$\phi = 0.75$

$\phi v_n = 0.212\text{ ksi} > v_u = 0.176\text{ ksi}$ OK

SHEARWALL CALCULATIONS

SHEARWALL #3



f'_c = 5000 psi

f_y = 60000 psi

REINFORCEMENT @ FIRST FLOOR

- BOUNDARY ELEMENTS: (14) #9 BARS VERT.
  #4 TIES @ 12" o.c.

- WEB: #5 BARS @ 18" o.c. VERT. EACH FACE
  #5 BARS @ 18" o.c. HORIZ. EACH FACE

LOADS: SELF WEIGHT: (150 pcf)(1' × 82' × 30') = 369 k

SHEAR: 151.1 k

MOMENT @ BASE: 37.8 k × 82'
+ 27.9 k × 67.33'
+ 26.8 k × 52.67'
+ 28.4 k × 38'
+ 30.2 k × 20'
8073 ft·k

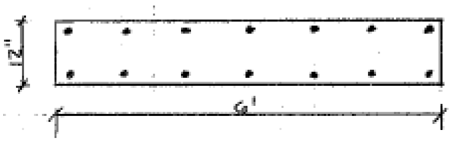
CONTROLLING LOAD CASE: 1.2D + 1.6W

SIMPLIFIED METHOD: $C = W_u + M_u/z$, $T = M_u/z$

$C = 1.2(369) + 1.6(8073)/24' = 981\ k$

$T = 1.6(8073)/24' = 538\ k$

BOUNDARY ELEMENT: A_s = (14) #9 BARS = 14 in²



$\phi P_n = 0.8\phi[0.85f'_c(A_g - A_s) + f_y A_s]$

$\phi P_n = 0.8(0.65)[0.85(5)(72\times12 - 14) + (60)(14)]$

COMPRESSION: $\phi P_n = 2315\ k > 981\ k$ OK

TENSION: $\phi T_n = \phi f_y A_s$

$\phi T_n = (0.9)(60)(14)$

$\phi T_n = 756\ k > 538\ k$ OK

SHEARWALL #3

SHEAR CALCULATIONS

$$V_c = MIN\begin{cases} 3.3\sqrt{f'c}\ hd + \frac{N_u d}{4 l_w} \\ \left[0.6\sqrt{f'c} + \frac{l_w\left(1.25\sqrt{f'c} + 0.2\frac{N_u}{l_w h}\right)}{M_u/V_u - l_w/2}\right]hd \end{cases}$$

$N_u = 1.2(369) = 443\ k$

$M_u = 1.6(8073) = 12920\ ft \cdot k$

$V_u = 1.6(151.1) = 242\ k$

$$V_c = MIN\begin{cases} 3.3\sqrt{5000}\ (12)(288) + \frac{443000 \times 288}{4(360)} = 895\ k \\ \left[0.6\sqrt{5000} + \frac{(360)\left(1.25\sqrt{5000} + 0.2\frac{443000}{(360)(12)}\right)}{12\left(12920/242\right) - 360/2}\right](12)(288) = 441\ k \leftarrow \end{cases}$$

$\phi V_c = 0.75 \times 441 = 331\ k > V_u = 242\ k$ OK

$\phi V_c/2 < V_u$, $\therefore$ PROVIDE SHEAR REINF. IN ACCORDANCE WITH 11.10.9

ρ_l & $\rho_t \geq 0.0025$: $0.0025 \leq \frac{A_{s\,trans}}{A_{cv}}$, USE 12" STRIP

$0.0025 \leq \frac{A_{s\,trans}}{12'' \times 12''}$

$A_{s\,trans} \geq 0.36\ in^2/ft$ OF WALL HEIGHT

#5 BARS @ 18" EACH FACE: $A_{s\,trans} = 2\left[0.31 \times \frac{12''}{18''}\right] = 0.41\ in^2 > 0.36\ in^2$ OK

$0.0025 \leq \frac{A_{s\,long}}{A_{cv}}$, USE 12" STRIP

$0.0025 \leq \frac{A_{s\,long}}{12'' \times 12''}$

$A_{s\,long} \geq 0.36\ in^2/ft$ OF WALL WIDTH

#5 BARS @ 18" EACH FACE: $A_{s\,long} = 2\left[0.31 \times \frac{12''}{18''}\right] = 0.41\ in^2 > 0.36\ in^2$ OK