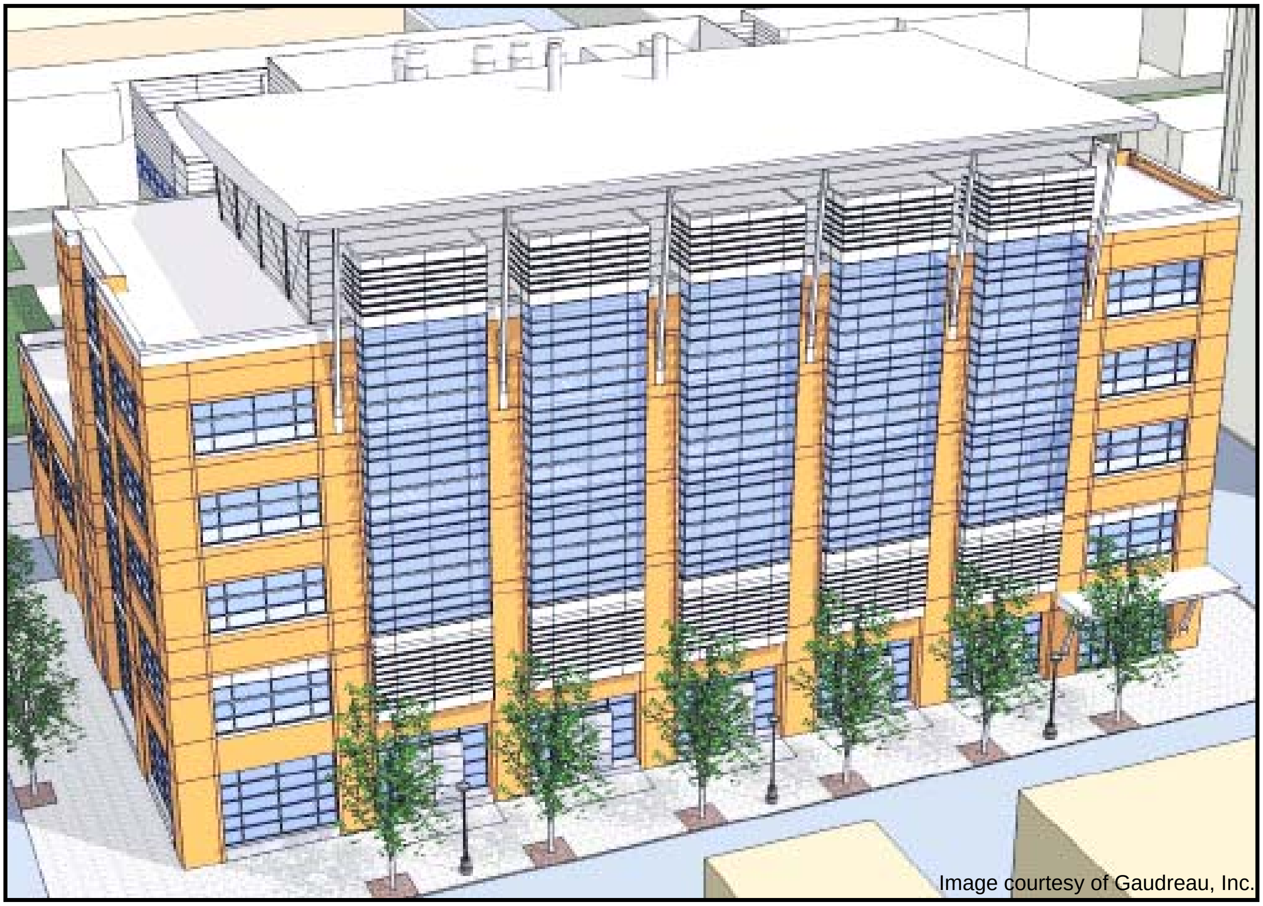
THE FORENSIC MEDICAL CENTER



TECHNICAL REPORT #3
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STRUCTURAL OPTION

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EXECUTIVE SUMMARY

The Forensic Medical Center is a five-story concrete laboratory building with a steel framed penthouse level that brings the total height of the building to 105 ft. above grade. The architects on the project are Gaudreau Inc., and McClaren, Wilson, and Laurie, and the structural engineer is Hope Furrer Associates.

Technical Report 3 is an in-depth investigation of the lateral force resisting system of The Forensic Medical Center, a dual-system consisting of reinforced concrete shearwalls with a reinforced concrete moment frame capable of resisting 25% of the lateral loads.

For this report, computer models were built and analyzed using the computer program ETABS. The results from the program were verified through spot-checks of various members using hand-calculation methods that approximated the actual behavior of the building. The hand calculations were near enough to the ETABS results to justify the use of the program.

This report concludes that the lateral system of The Forensic Medical Center is adequate for the wind and seismic loads applied. In some cases, the lateral system appears to be over-designed, but this could be due to the highly motion-sensitive nature of the high-tech laboratory equipment used in the building.

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BUILDING DESCRIPTION

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.

A steel-framed mechanical penthouse sits on the top of the Penthouse level. The HSS 14"x14"x½" columns are cantilevered from the concrete floor slab and extend 20' to the roof.

Lateral System

The lateral force resisting system of the Forensic Medical Center is a dual-system, consisting of four ordinary reinforced concrete shearwalls with an ordinary reinforced concrete moment frame. To be considered a dual-system and use the increased R-value for a dual-system, the moment frame must be able to resist 25% of the lateral loads.

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 ρ = 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.

Figure 1 shows the location of the shearwalls, as well as the columns considered part of the moment frame. The moment frame is made up of these columns and the concrete floor slab and perimeter beams between them. Reinforcement was added to these slabs and beams where necessary to add the moment capacity required to resist 25% of the lateral loads.

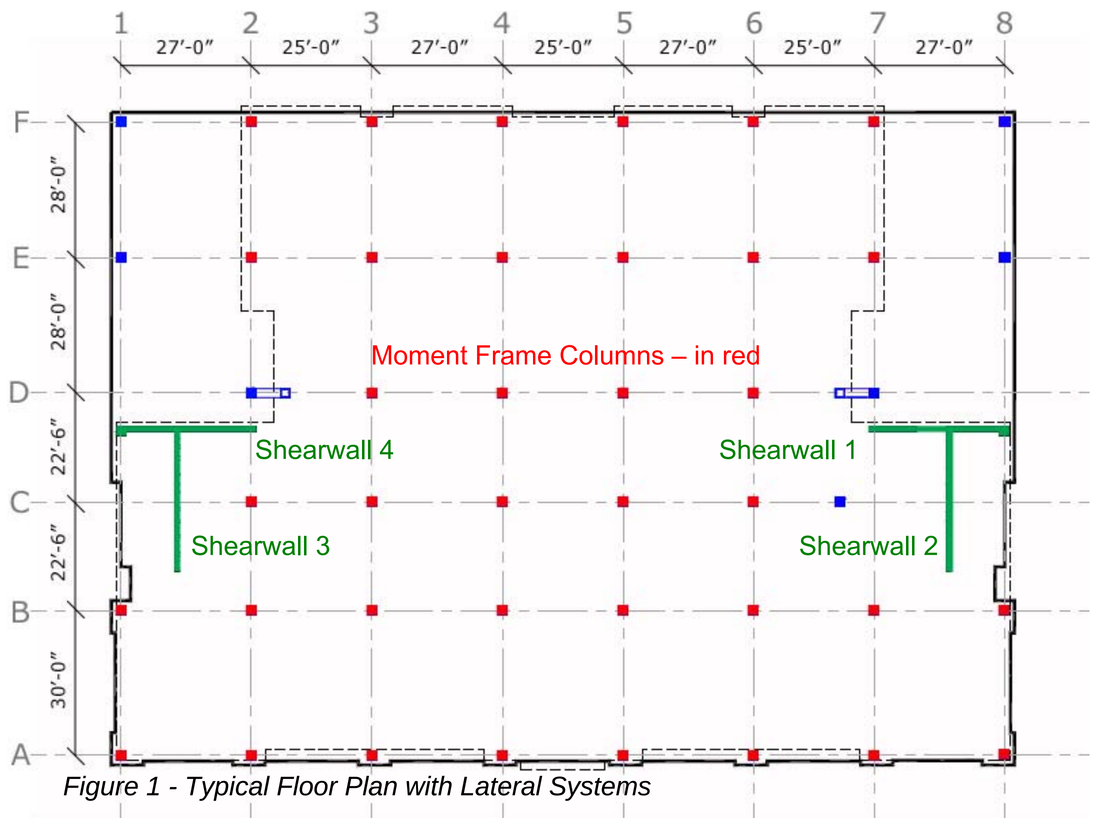


Figure 1 - Typical Floor Plan with Lateral Systems

LOADS AND LOAD DISTRIBUTION

Lateral Loads

In this lateral system, the four shearwalls are much stiffer than the moment frames. Using both systems in one computer model results in nearly all of the lateral load being resisted by the shearwalls. In order to justify the increased R-value of the dual-system, the moment frame must be checked without the shearwalls. Separate models were created for the moment frame, the shearwalls, and the dual-system.

The lateral loads calculated in Technical Report #1 were used in this report, with an adjustment to the seismic forces. The R-value was increased from 5 to the dual-system value of 5.5. The loads used in this analysis are shown in Figure 2.

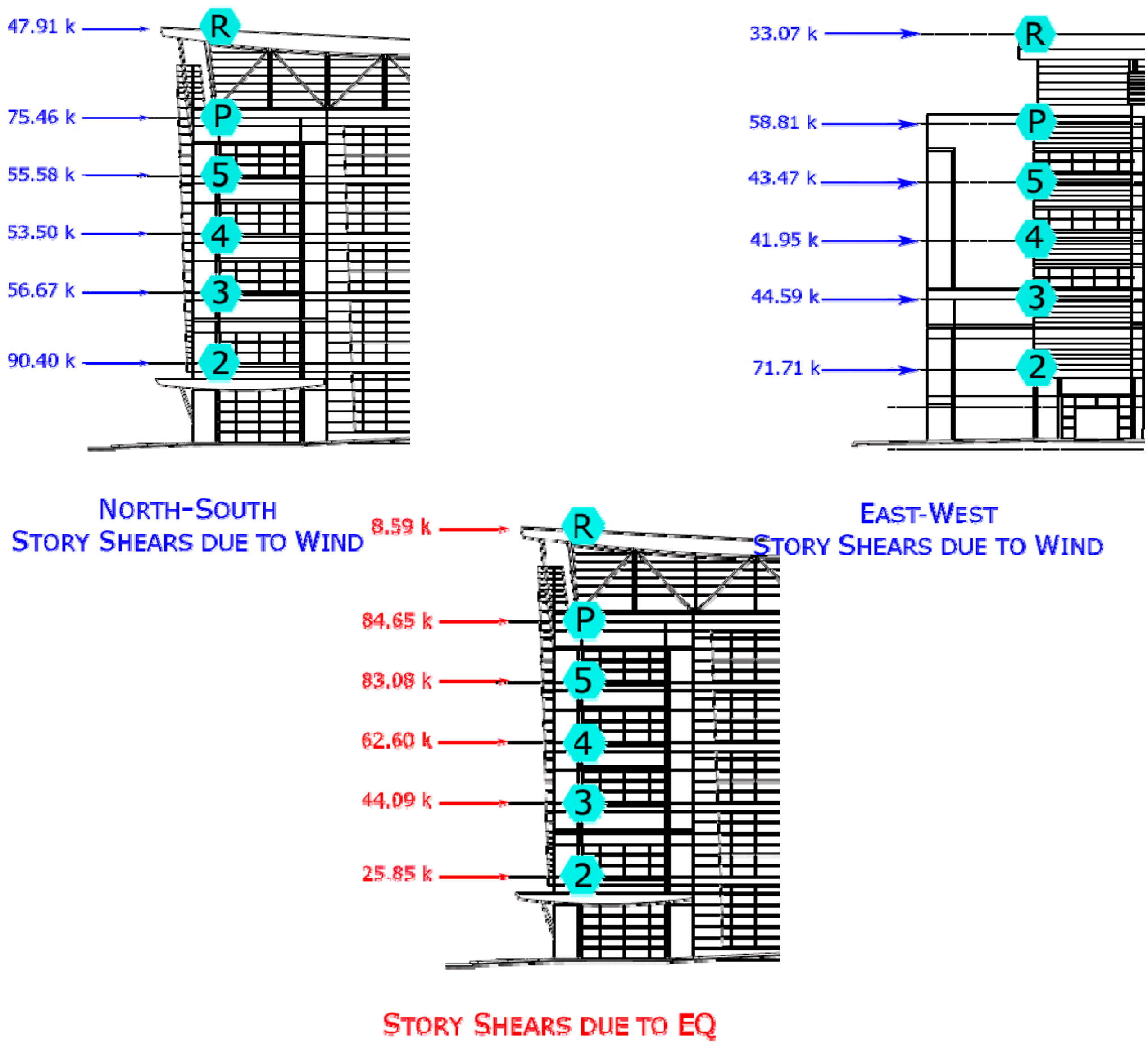


Figure 2 - Lateral Loads

The following load combinations from ACI 318-05 were used for this analysis:

- 1.4D
- 1.2D + 1.6L
- 1.2D + 1.6W + 1.0L
- 1.2D + 1.0E + 1.0L
- 0.9D + 1.6W
- 0.9D + 1.0E

- See Appendix A for Load Calculations -

Distribution

Before computer analysis of the entire building, several individual lateral system elements were spot-checked by hand, with some computer assistance for load distribution.

First, one quarter of the controlling lateral loads for each story were to be applied to the moment frame. The section of the frame along gridline 6 (Fig. 3) was selected for analysis by hand in the north-south (Y-axis) direction. The load distribution was estimated proportionally according to the frame length in the north-south direction (Fig. 4). A portal method analysis was used to estimate the moments in the beams and columns of the frame. These results were compared to the results obtained from the frame-only ETABS model. The two methods yielded similar results (Fig. 5), so the ETABS frame model is justified. The discrepancy in the column moments is most likely due to the assumption that the moment is zero at the mid-height point of the columns, and also due to the assumption that the interior columns resist twice the shear of the outside columns.

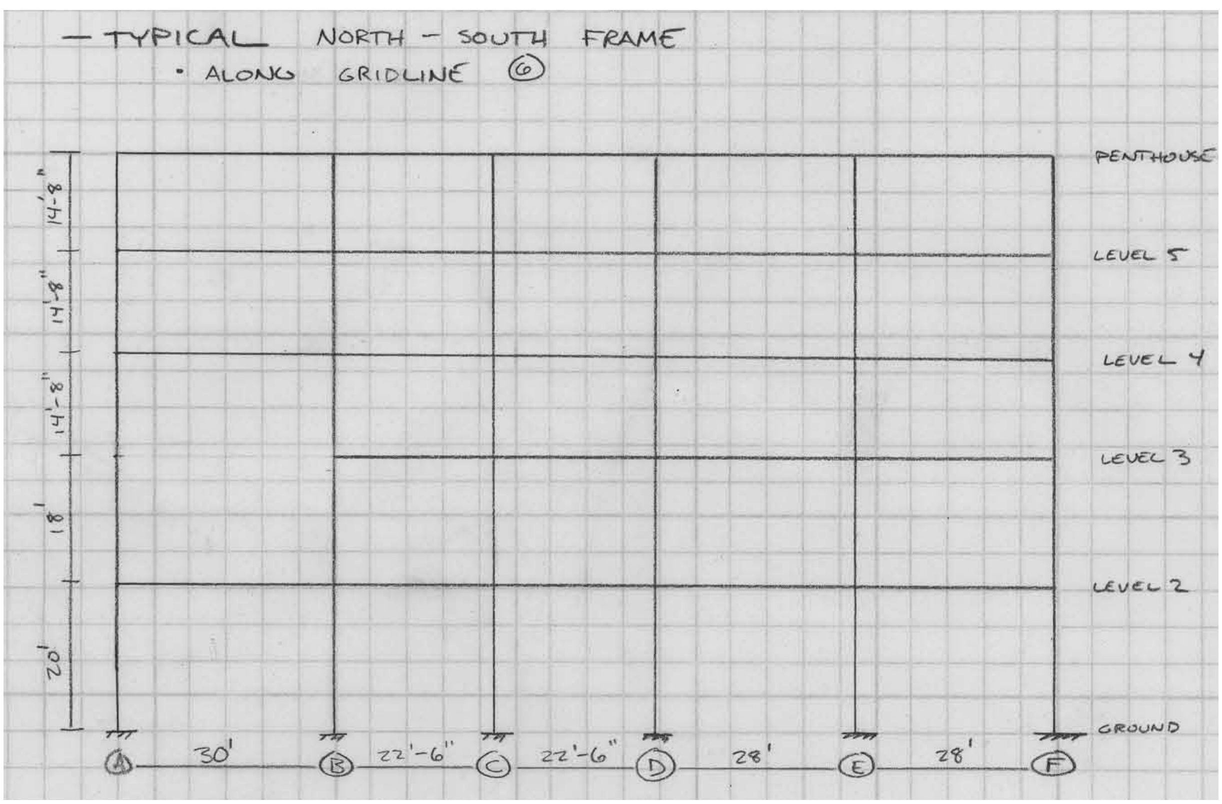


Figure 3 - Moment Frame along gridline 6

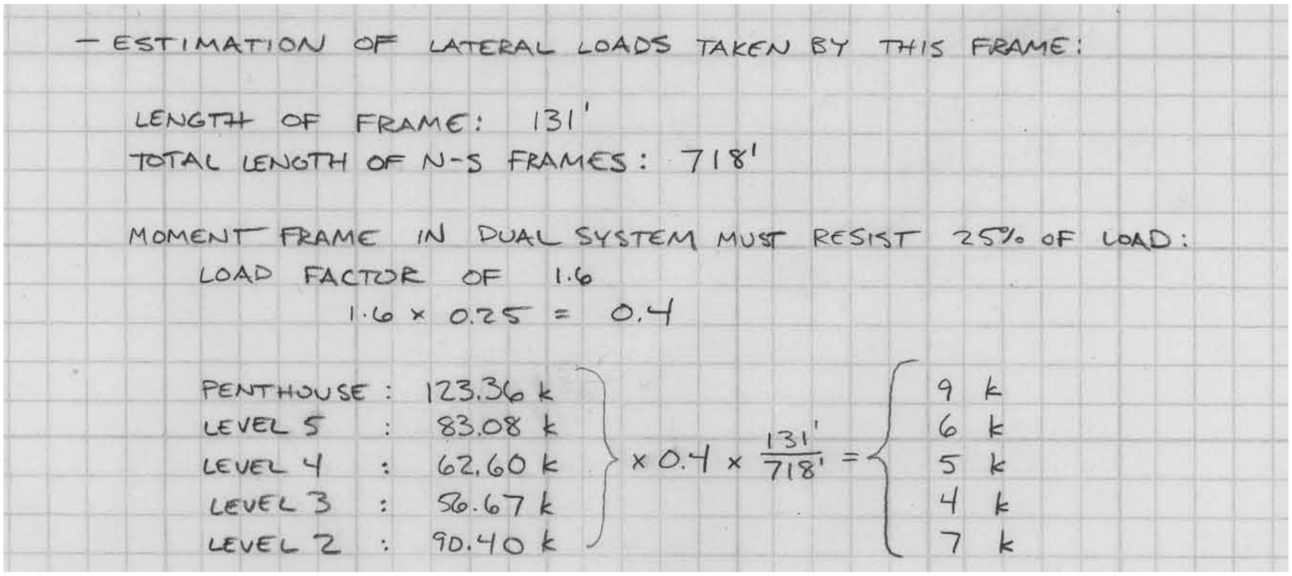
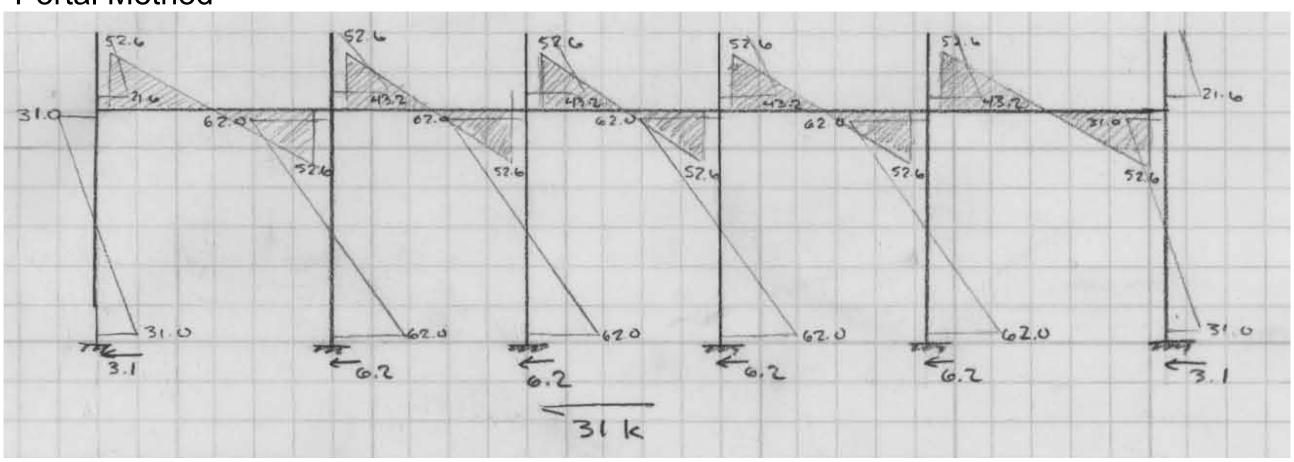


Figure 4 - Distribution to frame 6

-Portal Method



-ETABS

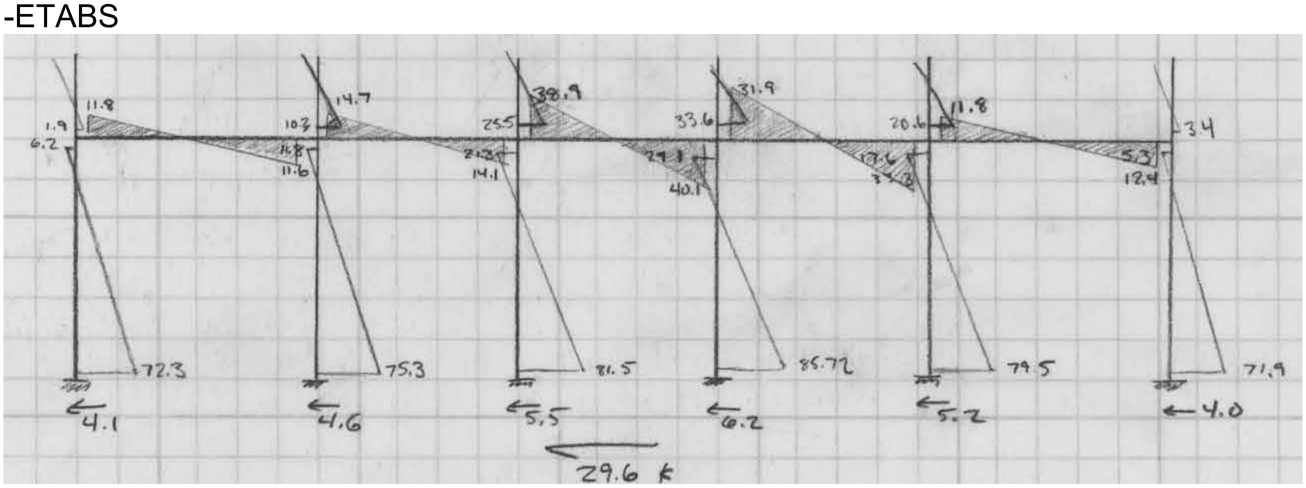


Figure 5 - Portal Method vs. ETABS results

Load distribution to the shearwalls is based on relative stiffness. To find the relative stiffnesses of the walls, they were each modeled individually in ETABS, and a 100 kip load was applied at each story. The relative stiffness of each wall could then be found as the reciprocal of the deflection of the wall at the story where the load was applied.

Example – Relative Shearwall Stiffnesses at Level Four				
Wall	X-Deflection	X-Rel. Stiffness	Y-Deflection	Y-Rel. Stiffness
Shearwall 1	0.894	1.1186		
Shearwall 2			0.587	1.7036
Shearwall 3			0.587	1.7036
Shearwall 4	0.732	1.3661		

These relative stiffness values were then used to calculate the building's center of rigidity at each story.

Example - Level Four Center of Rigidity						
	Relative Stiffness		Dist. from origin			
Element	R_x	R_y	x	y	$R_x y$	$R_y x$
Shearwall 1	1.1186			67.00	74.9462	
Shearwall 2		1.7036	171.38			291.9630
Shearwall 3		1.7036	11.63			19.8129
Shearwall 4	1.3661			67.00	91.5287	
SUM	2.4847	3.4072			166.4749	311.7758

C.O.R.:	x=	91.505	y=	67.000	ft.

The center of mass was also calculated for each story.

Example - Level Four Center of Mass				Dist. from origin			
Element	Area	Height	W	x	y	Wx	Wy
11" Floor	21700	0.92	2994.6	93.50	59.86	279995.1	179256.8
SW1	24	14.67	52.812	168.00	67.00	8872.416	3538.404
SW2	29	14.67	63.8145	171.38	52.50	10936.53	3350.261
SW3	29	14.67	63.8145	11.63	52.50	742.1626	3350.261
SW4	24	14.67	52.812	15.00	67.00	792.18	3538.404
		SUM	3227.853			301338.4	193034.1

C.O.M.:	x=	93.356	y=	59.803	ft.

The controlling lateral load at each story in each direction was applied at the center of mass, and the shear was distributed to each shearwall according to its stiffness. Shear due to torsion was also calculated.

Example - Level Four Y-Dist.	Story Shear =			62.60	k, Moment=	498	ft-k
	Relative Stiffness		Dist. from		Direct	Torsional	Total Shear
Element	Rx	Ry	COR	(Rel. Stiff.)x(COR)^2	Shear	Shear	Vn (k)
SW1	1.1186		0.00	0.0000	0.0000	0.0000	0.00
SW2		1.7036	79.88	10868.9946	134.5200	3.1168	137.64
SW3		1.7036	-79.88	10868.9946	134.5200	-3.1168	131.40
SW4	1.3661		0.00	0.0000	0.0000	0.0000	0.00
SUM	2.4847	3.4072		21737.98924			269.04

Because both east-west running shearwalls are in the same plane, the center of rigidity also lies in that plane. For this reason, there is no shear due to torsion in these walls (Shearwalls 1 and 4).

-See Appendix B for complete Shearwall Distribution spreadsheets-

MOMENT FRAME ANALYSIS

According to both ETABS and hand calculation estimates, the worst case for a column in the typical section of the moment frame is an interior column between the ground floor and level two. Hand calculations show a worst-case moment of 62.0 ft-k, while ETABS shows a higher moment of 85.7 ft-k, which was used for this analysis. The axial load on the column was calculated based on tributary area, and was found to be 691 k.

An interaction graph (Fig. 6) for a typical interior column was created by plotting five points: pure axial strength, balanced strain, pure bending strength, c = column width, and ε_t = 0.005. The applied loads were found to be well within the interaction curve. The columns seem to be oversized for the loads they are taking. This could be a topic for further investigation.

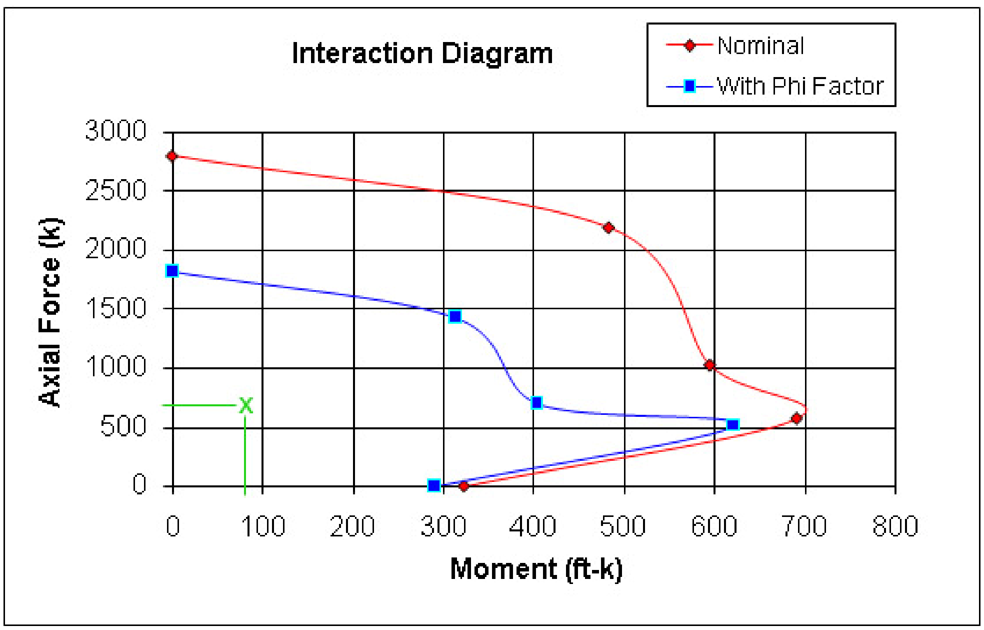


Figure 6 - Exterior Column Interaction Diagram

The “beams” of the moment frame, which are made up of the 11”-thick concrete floor slab, were also analyzed by hand calculation. The worst-case moment due to lateral loads was 52.6 ft-k, found in the portal-method analysis. This moment was added to 214 ft-k, the moment due to gravity loads, which was found using coefficients from ACI 318-05 8.3.3.

The cross-section of the “beam” was assumed to be the 24” wide segment of the 11” slab located along the column line. Typical negative moment reinforcing in this area is #5 bars at 9” on-center, giving an area of 0.31 in^2 of steel per foot of width, or 0.62 in^2 in this cross-section. The negative moment capacity was calculated based on these assumptions, and was found to be adequate at 433 ft-k.

The ETABS model containing only the moment frame was run using the required 25% of the lateral load. The frame was found to be adequate to resist these loads, justifying the use of the reduced R-value for a dual lateral force resisting system.

-See Appendix C for Moment Frame Check Calculations-

SHEARWALL ANALYSIS

Shearwall 4 was selected for analysis by hand calculations. There are two areas of concern: the base of the wall, as well as Level 3, where a doorway interrupts the wall. The shear at each story calculated according to the relative stiffness of the wall and the shear due to torsion were applied.

The wall was checked for shear capacity, boundary element requirements, and overturning at the base. Boundary elements are present in the wall, but were found to be unnecessary. The shear capacity of the concrete alone is sufficient, but according to ACI 318-05, a minimum ρ_l and ρ_t of 0.0025 is required because the ultimate shear is more than half of the allowable shear capacity of the concrete alone. This is supplied by the #5 bars at 18" on-center, each way, each face that are present in the web of the wall.

The simplified method was used at the opening in the wall at Level 3. The boundary elements were found to be adequate to resist the moment applied by the lateral loads, and the minimum shear reinforcement was adequate to resist the shear applied.

An ETABS model of the shearwalls alone was created and run. All four walls were found to be adequate to resist the lateral loads applied in the computer program as well.

-See Appendix D for Shearwall Check Calculations-

LATERAL DRIFT ANALYSIS

Typical allowances for lateral drift of a building are H/400. In this case, 105 ft. / 400 equals just over three inches. An additional constraint is that the steel-framed penthouse on the roof cannot deflect more than 1” in any direction because of the façade system. The cantilevered columns of the penthouse were found to be adequate according to the ETABS model, with the roof moving about 0.9” under wind loads. The main structure of the building, however, drifts only 0.2” total at the Penthouse level. These results are similar to those from a RAM model created by the structural engineer on the project. This could be due to the nature of the laboratory equipment in The Forensic Medical Center, which is very sensitive to movement, it may be that the lateral system is very conservatively designed, or possibly deflection and displacement of the systems are not the controlling factors.

Story Drifts (in.) due to Seismic Loads				
Story	N-S Direction Story Drift	N-S Direction Total Drift	E-W Direction Story Drift	E-W Direction Total Drift
Penthouse	0.0199	0.0903	0.0314	0.1477
Level 5	0.0194	0.0704	0.0323	0.1163
Level 4	0.0184	0.0510	0.0317	0.0840
Level 3	0.0197	0.0326	0.0337	0.0523
Level 2	0.0129	0.0129	0.0186	0.0186

Story Drifts (in.) due to Wind Loads				
Story	N-S Direction Story Drift	N-S Direction Total Drift	E-W Direction Story Drift	E-W Direction Total Drift
Penthouse	0.0379	0.1581	0.0441	0.1980
Level 5	0.0326	0.1202	0.0424	0.1539
Level 4	0.0309	0.0876	0.0417	0.1115
Level 3	0.0327	0.0567	0.0435	0.0698
Level 2	0.0240	0.0240	0.0263	0.0263

CONCLUSION

After detailed analysis, both by hand and with a computer model in ETABS, the lateral load resisting system of The Forensic Medical Center was found to be adequate, and possibly over-designed.

Being a dual-system consisting of ordinary reinforced concrete shearwalls as the main system with an additional ordinary reinforced concrete moment frame, there are certain requirements that must be met to obtain a better R-value for seismic loads.

The moment frame must be capable of resisting 25% of the lateral loads that will be applied to the building. Since the forces are distributed according to stiffness, almost all of the force in a computer model of the entire dual system would be resisted by the stiffer shearwalls. For this reason, each system must be modeled separately to ensure the system meets these requirements. In this case, the moment frame appears to be able to withstand well over the required 25% of the load. The columns in the moment frame appear to be much larger than required to resist both lateral and gravity loads. These could possibly be designed as much smaller columns to help reduce the building's cost.

The shearwalls in the building were found to be adequate for the loads they are required to resist. In a hand calculation, the shearwall investigated had extra reinforcement in its boundary elements that did not appear to be required. Perhaps the shearwalls could be designed with less reinforcement, or as thinner walls, which could lower the cost of the building.

The lateral drift of the building is very small compared to the allowable H/400. This may be because of the high-tech laboratory equipment in the building, which is sensitive to even very small movements and vibrations. If not, however, there is room for the reduction in size of the members in the lateral force resisting systems to save money.

APPENDIX

LOAD CALCULATIONS

Seismic Loads:

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

$$C_{vx} = \frac{w_x h_x^k}{\Sigma\, w_i h_i^k}$$ k = 1.1

S_S =	0.169	S_1 =	0.051
Fa =	1.2	Fv =	1.7
S_{DS} =	0.135	S_{D1} =	0.059

Dual System - Ordinary Conc. Shearwalls w/ Ordinary Conc. Moment Frames

R = 5.5

Occupancy Category IV - I = 1.5

$Ta = C_t h_n^x$ $= 0.02(105)^{0.75} =$ 0.656

T_L = 6

Cu = 1.7 CuTa = 1.12

Cs = MIN

$S_{DS}/(R/I)$ =	0.036818
$S_{D1}/[T(R/I)]$ =	0.014367
$(S_{D1}T_L)/[T^2(R/I)]$ =	0.076965

$V = C_s * W$ C_s = 0.014367

$F_x = C_{vx} * V$

V = 308.9 k

	h_x	$w_x h_x^k$
Level 2	20	135885
Level 3	38	231761
Level 4	52.67	329073
Level 5	67.33	436747
Penthouse	82	444961
Roof	105	45151
	Σ =	1623577

	Cvx	Fx	
Level 2	0.0837	25.85	k
Level 3	0.1427	44.09	k
Level 4	0.2027	62.60	k
Level 5	0.2690	83.08	k
Penthouse	0.2741	84.65	k
Roof	0.0278	8.59	k
Σ =	1.0000	308.86	k

Wind Loads:

V = 90 mph I = 1.15

K_{zt} = 1.0 K_d = 0.85

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

$I_z = c * (33/z)^{(1/6)} = 0.18$

$L_z = l * (z/33)^{\varepsilon} = 569$

c = 0.2 z = 63 ε = 0.2

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

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

$q = 0.00256*K_zK_dV^2I$

$p = qGC_p$

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

Interior Column (Column D-6) Gravity Loads:

Load Case		
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

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	205	30	21	25.25	22.5	568.125	116	7	12
5	146	80		25.25	22.5	568.125	83	18	
4	146	80		25.25	22.5	568.125	83	18	
3	146	80		25.25	22.5	568.125	83	18	
2	146	80		25.25	26	656.5	96	21	
					Total A_T =	2929	461	82	12
					K_{LL} =	4			
					LL Reduction Factor =	0.40			

	Case 1 (k)	Case 2 (k)	Case 3 (k)
Load @ Level 1 =	691	642	638

SHEARWALL DISTRIBUTION CALCULATIONS

CENTER OF MASS CALCULATION	Conc. Wt.:	150	pcf

Penthouse Level				Dist. from origin			
Element	Area	Height	W	x	y	Wx	Wy
15" Floor	9637	1.25	1806.938	93.50	45.82	168948.7	82793.88
8" Floor	10880	0.67	1093.44	93.50	72.29	102236.6	79044.78
SW1	27	14.67	59.4135	169.50	67.00	10070.59	3980.705
SW2	23	14.67	50.6115	171.38	49.50	8673.799	2505.269
SW3	29	14.67	63.8145	11.63	52.50	742.1626	3350.261
SW4	27	14.67	59.4135	13.50	67.00	802.0823	3980.705
		SUM	3133.631			291473.9	175655.6
	C.O.M.:	x=	93.015	y=	56.055	ft.	
Level Five				Dist. from origin			
Element	Area	Height	W	x	y	Wx	Wy
11" Floor	21810	0.92	3009.78	93.50	59.86	281414.4	180165.4
SW1	27	14.67	59.4135	169.50	67.00	10070.59	3980.705
SW2	29	14.67	63.8145	171.38	52.50	10936.53	3350.261
SW3	29	14.67	63.8145	11.63	52.50	742.1626	3350.261
SW4	27	14.67	59.4135	13.50	67.00	802.0823	3980.705
		SUM	3256.236			303965.8	194827.4
	C.O.M.:	x=	93.349	y=	59.832	ft.	
Level Four				Dist. from origin			
Element	Area	Height	W	x	y	Wx	Wy
11" Floor	21700	0.92	2994.6	93.50	59.86	279995.1	179256.8
SW1	24	14.67	52.812	168.00	67.00	8872.416	3538.404
SW2	29	14.67	63.8145	171.38	52.50	10936.53	3350.261
SW3	29	14.67	63.8145	11.63	52.50	742.1626	3350.261
SW4	24	14.67	52.812	15.00	67.00	792.18	3538.404
		SUM	3227.853			301338.4	193034.1
	C.O.M.:	x=	93.356	y=	59.803	ft.	
Level Three				Dist. from origin			
Element	Area	Height	W	x	y	Wx	Wy
11" Floor	18547	0.92	2559.486	93.98	69.02	240540.5	176655.7
9" Roof	3564	0.75	400.95	93.50	99.00	37488.83	39694.05
SW1	18	18	48.6	174.00	67.00	8456.4	3256.2
SW2	29	18	78.3	171.38	52.50	13419.05	4110.75
SW3	29	18	78.3	11.63	52.50	910.629	4110.75
SW4	27	18	72.9	13.50	67.00	984.15	4884.3
		SUM	3238.536			301799.6	232711.8
	C.O.M.:	x=	93.190	y=	71.857	ft.	
Level Two				Dist. from origin			
Element	Area	Height	W	x	y	Wx	Wy
11" Floor	25368	0.92	3500.784	93.50	65.50	327323.3	229301.4
SW1	21	20	63	172.50	67.00	10867.5	4221
SW2	29	20	87	171.38	52.50	14910.06	4567.5
SW3	29	20	87	11.63	52.50	1011.81	4567.5
SW4	27	20	81	13.50	67.00	1093.5	5427
		SUM	3818.784			355206.2	248084.4
	C.O.M.:	x=	93.016	y=	64.964	ft.	

CENTER OF RIGIDITY CALCULATION

Penthouse Level

Element	Relative Stiffness R_x	R_y	Dist. from origin x	y	$R_x y$	$R_y x$
SW1	0.3563			67.00	23.8721	
SW2		0.4950	171.38			84.8331
SW3		0.4988	11.63			5.8010
SW4	0.3987			67.00	26.7129	
SUM	0.7550	0.9938			50.5850	90.6341

C.O.R.: x= 91.200 y= 67.000 ft.

Level Five

Element	Relative Stiffness R_x	R_y	Dist. from origin x	y	$R_x y$	$R_y x$
SW1	0.6035			67.00	40.4345	
SW2		0.8688	171.38			148.8949
SW3		0.8673	11.63			10.0867
SW4	0.6954			67.00	46.5918	
SUM	1.2989	1.7361			87.0263	158.9816

C.O.R.: x= 91.574 y= 67.000 ft.

Level Four

Element	Relative Stiffness R_x	R_y	Dist. from origin x	y	$R_x y$	$R_y x$
SW1	1.1186			67.00	74.9462	
SW2		1.7036	171.38			291.9630
SW3		1.7036	11.63			19.8129
SW4	1.3661			67.00	91.5287	
SUM	2.4847	3.4072			166.4749	311.7758

C.O.R.: x= 91.505 y= 67.000 ft.

Level Three

Element	Relative Stiffness R_x	R_y	Dist. from origin x	y	$R_x y$	$R_y x$
SW1	2.2936			67.00	153.6712	
SW2		4.0000	171.38			685.5200
SW3		4.0000	11.63			46.5200
SW4	3.3670			67.00	225.5890	
SUM	5.6606	8.0000			379.2602	732.0400

C.O.R.: x= 91.505 y= 67.000 ft.

Level Two

Element	Relative Stiffness R_x	R_y	Dist. from origin x	y	$R_x y$	$R_y x$
SW1	11.1111			67.00	744.444	
SW2		17.5439	171.38			3006.674
SW3		17.5439	11.63			204.036
SW4	14.9254			67.00	1000.002	
SUM	26.0365	35.0878			1744.446	3210.709

C.O.R.: x= 91.505 y= 67.000 ft.

SHEAR CALCULATION - X-Direction (East-West)

Penthouse Level	EQ		Story Shear =	93.24	k, Moment=	-1021	ft-k
Element	Relative Stiffness Rx	Ry	Dist. from COR	(Rel. Stiff.)x(COR)^2	Direct Shear	Torsional Shear	Total Shear Vn (k)
SW1	0.3563		0.00	0.0000	44.0019	0.0000	44.00
SW2		0.4950	80.18	3182.3053	0.0000	-6.3882	-6.39
SW3		0.4988	-79.57	3158.0616	0.0000	6.3882	6.39
SW4	0.3987		0.00	0.0000	49.2381	0.0000	49.24
SUM	0.7550	0.9938		6340.3668			93.24

Level Five	EQ		Story Shear =	83.08	k, Moment=	-1264	ft-k
Element	Relative Stiffness Rx	Ry	Dist. from COR	(Rel. Stiff.)x(COR)^2	Direct Shear	Torsional Shear	Total Shear Vn (k)
SW1	0.6035		0.00	0.0000	81.9225	0.0000	81.92
SW2		0.8688	79.81	5533.3834	0.0000	-7.9114	-7.91
SW3		0.8673	-79.94	5542.9534	0.0000	7.9114	7.91
SW4	0.6954		0.00	0.0000	94.3975	0.0000	94.40
SUM	1.2989	1.7361		11076.3369			176.32

Level Four	EQ		Story Shear =	62.60	k, Moment=	-1720	ft-k
Element	Relative Stiffness Rx	Ry	Dist. from COR	(Rel. Stiff.)x(COR)^2	Direct Shear	Torsional Shear	Total Shear Vn (k)
SW1	1.1186		0.00	0.0000	107.5606	0.0000	107.56
SW2		1.7036	79.88	10868.9946	0.0000	-10.7643	-10.76
SW3		1.7036	-79.88	10868.9946	0.0000	10.7643	10.76
SW4	1.3661		0.00	0.0000	131.3594	0.0000	131.36
SUM	2.4847	3.4072		21737.98924			238.92

Level Three	WIND		Story Shear =	44.59	k, Moment=	1377	ft-k
Element	Relative Stiffness Rx	Ry	Dist. from COR	(Rel. Stiff.)x(COR)^2	Direct Shear	Torsional Shear	Total Shear Vn (k)
SW1	2.2936		0.00	0.0000	114.8745	0.0000	114.87
SW2		4.0000	79.88	25520.0625	0.0000	8.6199	8.62
SW3		4.0000	-79.88	25520.0625	0.0000	-8.6199	-8.62
SW4	3.3670		0.00	0.0000	168.6355	0.0000	168.64
SUM	5.6606	8.0000		51040.125			283.51

Level Two	WIND		Story Shear =	71.71	k, Moment=	-723	ft-k
Element	Relative Stiffness Rx	Ry	Dist. from COR	(Rel. Stiff.)x(COR)^2	Direct Shear	Torsional Shear	Total Shear Vn (k)
SW1	11.1111		0.00	0.0000	151.5905	0.0000	151.59
SW2		17.5439	79.88	111930.3561	0.0000	-4.5267	-4.53
SW3		17.5439	-79.88	111930.3561	0.0000	4.5267	4.53
SW4	14.9254		0.00	0.0000	203.6295	0.0000	203.63
SUM	26.0365	35.0878		223860.7122			355.22

SHEAR CALCULATION - Y-Direction

Penthouse Level	WIND		Story Shear =	123.36	k, Moment=	224	ft-k
Element	Relative Stiffness Rx	Ry	Dist. from COR	(Rel. Stiff.)x(COR)^2	Direct Shear	Torsional Shear	Total Shear Vn (k)
SW1	0.3563		0.00	0.0000	0.0000	0.0000	0.00
SW2		0.4950	80.18	3182.3053	61.4442	1.4017	62.85
SW3		0.4988	-79.57	3158.0616	61.9158	-1.4017	60.51
SW4	0.3987		0.00	0.0000	0.0000	0.0000	0.00
SUM	0.7550	0.9938		6340.3668			123.36

Level Five	EQ		Story Shear =	83.08	k, Moment=	366	ft-k
Element	Relative Stiffness Rx	Ry	Dist. from COR	(Rel. Stiff.)x(COR)^2	Direct Shear	Torsional Shear	Total Shear Vn (k)
SW1	0.6035		0.00	0.0000	0.0000	0.0000	0.00
SW2		0.8688	79.81	5533.3834	103.3092	2.2935	105.60
SW3		0.8673	-79.94	5542.9534	103.1308	-2.2935	100.84
SW4	0.6954		0.00	0.0000	0.0000	0.0000	0.00
SUM	1.2989	1.7361		11076.3369			206.44

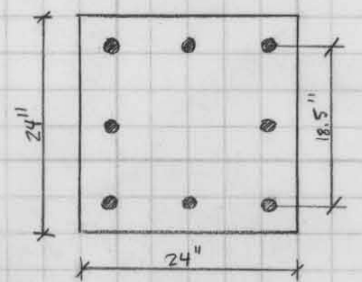
Level Four	EQ		Story Shear =	62.60	k, Moment=	498	ft-k
Element	Relative Stiffness Rx	Ry	Dist. from COR	(Rel. Stiff.)x(COR)^2	Direct Shear	Torsional Shear	Total Shear Vn (k)
SW1	1.1186		0.00	0.0000	0.0000	0.0000	0.00
SW2		1.7036	79.88	10868.9946	134.5200	3.1168	137.64
SW3		1.7036	-79.88	10868.9946	134.5200	-3.1168	131.40
SW4	1.3661		0.00	0.0000	0.0000	0.0000	0.00
SUM	2.4847	3.4072		21737.98924			269.04

Level Three	WIND		Story Shear =	56.67	k, Moment=	549	ft-k
Element	Relative Stiffness Rx	Ry	Dist. from COR	(Rel. Stiff.)x(COR)^2	Direct Shear	Torsional Shear	Total Shear Vn (k)
SW1	2.2936		0.00	0.0000	0.0000	0.0000	0.00
SW2		4.0000	79.88	25520.0625	162.8550	3.4357	166.29
SW3		4.0000	-79.88	25520.0625	162.8550	-3.4357	159.42
SW4	3.3670		0.00	0.0000	0.0000	0.0000	0.00
SUM	5.6606	8.0000		51040.125			325.71

Level Two	WIND		Story Shear =	90.4	k, Moment=	629	ft-k
Element	Relative Stiffness Rx	Ry	Dist. from COR	(Rel. Stiff.)x(COR)^2	Direct Shear	Torsional Shear	Total Shear Vn (k)
SW1	11.1111		0.00	0.0000	0.0000	0.0000	0.00
SW2		17.5439	79.88	111930.3561	208.0550	3.9345	211.99
SW3		17.5439	-79.88	111930.3561	208.0550	-3.9345	204.12
SW4	14.9254		0.00	0.0000	0.0000	0.0000	0.00
SUM	26.0365	35.0878		223860.7122			416.11

MOMENT FRAME CHECK CALCULATIONS

– EXTERIOR COLUMN



$f'_c = 5000$ psi

(8) #8 BARS
#4 TIES @ 12" o.c.

$f_y = 60000$ psi

INTERACTION CURVE CALCULATIONS

– PURE AXIAL STRENGTH:

$P_o = 0.85 f'_c A_c + A_s f_y = 0.85(5)(24^2 - 8(0.79)) + 60(8 \times 0.79)$

$P_o = 2800$ 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

$P_b = 0.85 f'_c b \beta_1 c + f_{s_1} A_{s_1} + f_{s_2} A_{s_2} + f_{s_3} A_{s_3}$
$= 0.85(5)(24)(0.8)(12.5) + 3(60 \times 0.79) + 2(3.5 \times 0.79) + 3(-60 \times 0.79)$
$= 1026$ 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

$\Sigma F = 0 = 0.85 f'_c b \beta_1 c + f_{s_1} A_{s_1} + f_{s_2} A_{s_2} + f_{s_3} A_{s_3}$
$0 = 0.85(5)(24)(0.8)c + 3(0.79)\left(\frac{0.003}{c}(c - 2.75)(29000)\right) + 2(0.79)(-60) + 3(0.79)(-60)$
$c = 2.83$ in $\Rightarrow f_{s_1} = 2.5$ ksi < 60 $\underline{\text{ok}}$

$\varepsilon_{s_2} = \frac{0.003}{2.83}(2.83 - 12) = -0.0097$
$\varepsilon_{s_3} = \frac{0.003}{2.83}(2.83 - 21.25) = -0.0195$ $\underline{\text{OK}}$

$M_o = 0.85 f'_c b \beta_1 c \left(\frac{w}{2} - \frac{\beta_1 c}{2}\right) + f_{s_1} A_{s_1}(12 - 2.75) + f_{s_2} A_{s_2}(12 - 12) + f_{s_3} A_{s_3}(12 - 21.25)$
$= 0.85(5)(24)(0.8)(2.83)\left(12 - \frac{0.8 \times 2.83}{2}\right) + 3(0.79)(2.5)(9.25) + 0 + 3(0.79)(-60)(-9.25)$

$M_o = 323$ ft·k

EXTERIOR COLUMN

$- c = w = 24''$

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

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

$\varepsilon_{s_3} = \frac{0.003}{24}(24 - 21.25) = 0.00034 \qquad f_{s_3} = E_s \varepsilon_{s_3} = 10 \text{ ksi}$

$P_n = 0.85 f'_c b \beta_1 c + f_{s_1} A_{s_1} + f_{s_2} A_{s_2} + f_{s_3} A_{s_3}$
$= 0.85(5)(24)(0.8)(24) + 3(0.79)(60) + 2(0.79)(44) + 3(0.79)(10)$
$= 2194 \text{ k}$

$M_n = 0.85 f'_c b \beta_1 c \left(\frac{w}{2} - \frac{\beta_1 c}{2}\right) + f_{s_1} A_{s_1}(12 - 2.75) + F_{s_2} A_{s_2}(12 - 12) + f_{s_3} A_{s_3}(12 - 21.25)$
$= 0.85(5)(24)(0.8)(24)\left(12 - \frac{0.8 \times 24}{2}\right) + 3(60)(0.79)(9.25) + 0 + 3(10)(0.79)(-9.25)$
$= 483 \text{ ft} \cdot \text{k}$

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

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

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

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

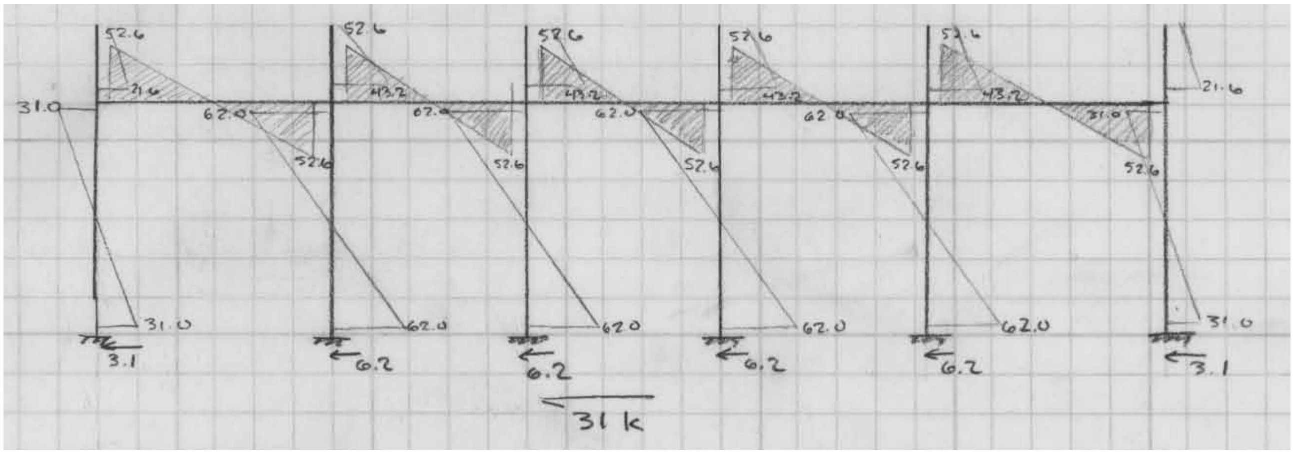
$P_n = 0.85(5)(24)(0.8)(7.97) + 3(0.79)(57) + 2(0.79)(-44) + 3(0.79)(-60)$
$= 574 \text{ k}$

$M_n = 0.85(5)(24)(0.8)(7.97)\left(12 - \frac{0.8 \times 7.97}{2}\right) + 3(0.79)(57)(12 - 2.75) + 0 + 3(-60)(12 - 21.25)(0.79)$
$= 691 \text{ ft} \cdot \text{k}$

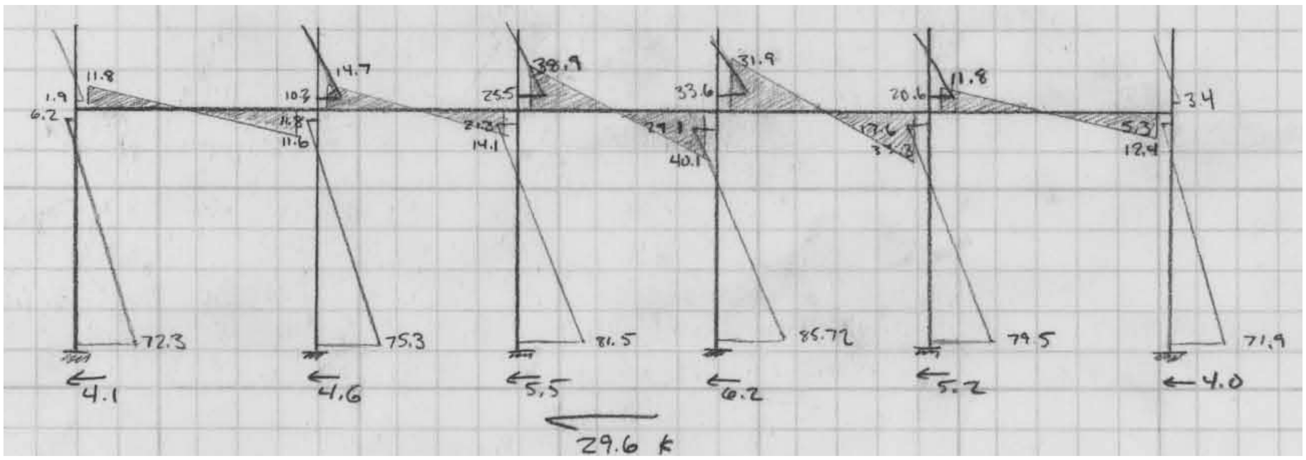
—INTERACTION :

P_n (k)	M_n (ft·k)	c (in)	$\varepsilon_t = \varepsilon_{s_3}$	Ø
2800	0	∞		0.65
2194	483	24	0.00034	0.65
1026	595	12.5	-0.0021	0.68
574	691	7.97	-0.005	0.9
0	323	2.83	-0.0195	0.9

Portal Method



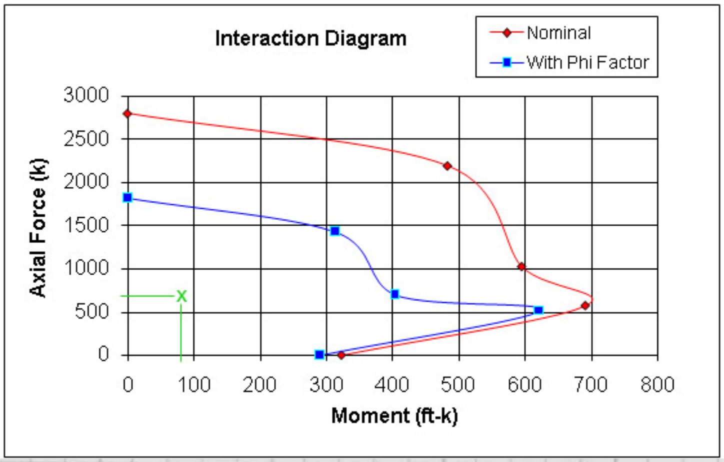
ETABS



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

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	205	30	21	25.25	22.5	568.125	116	7	12
5	146	80		25.25	22.5	568.125	83	18	
4	146	80		25.25	22.5	568.125	83	18	
3	146	80		25.25	22.5	568.125	83	18	
2	146	80		25.25	26	656.5	96	21	
					Total A_T =	2929	461	82	12
					K_{LL} =	4			
					LL Reduction Factor =	0.40			

	Case 1 (k)	Case 2 (k)	Case 3 (k)
Load @ Level 1 =	691	642	638



GRAVITY LOAD – INDUCED MOMENTS IN FRAME 6
– CENTER SPAN – NEGATIVE MOMENT @ SUPPORTS

COEFFICIENT : $\frac{w_u l_n^2}{11}$

$w_u \Rightarrow$ 8"/12" × 150 pcf × 26' TRIB. WIDTH = 2.6 k/ft DEAD
10 psf × 26' = 0.3 k/ft SUPERIMPOSED
80 psf × 26' = 2.1 k/ft LIVE

LOAD CASE: 1.2D + 1.6W + 1.0L

1.2D + 1.0L = 5.6 k/ft

l_n = 22'-6" – (2 × 1'-0") = 20'-6"
COLUMNS

$M = \frac{5.6 \times (20'\text{-}6")^2}{11} = 214$ ft·k

+ 40 ft·k FROM LATERAL LOAD (ETABS)

M_u = 254 ft·k

– AT THIS COLUMN, SLAB TOP REINF. IS #5 BARS @ 9" o.c.
EXISTING SLAB REINF. IS USED AS PART OF MOMENT FRAME

CHECK MOMENT CAPACITY OF 24" STRIP OF SLAB AT COLUMN IN MOMENT FRAME...

$M_n = A_s F_y (d - a/2)$ $A_s = 0.31 \text{ in}^2 \times \frac{12"/ft}{9"} \times 2' = 0.83 \text{ in}^2$ (SPACING)

$a = \frac{A_s F_y}{0.85 f'_c b} = \frac{0.83(60)}{0.85(5)(24)} = 0.49"$

$M_n = (0.83)(60)(9.9 - \frac{0.49}{2})$ $c = a/\beta_1 = \frac{0.49}{0.8} = 0.61"$

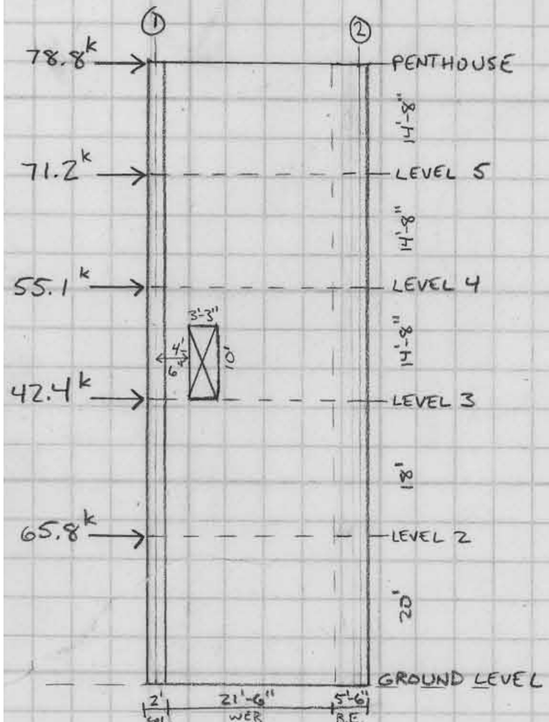
$\varepsilon_s = \frac{0.003}{0.61}(9.9 - 0.61) = 0.046 \Rightarrow \phi = 0.9$

M_n = 481 ft·k

ϕM_n = 433 ft·k > 254 ft·k OK

SHEARWALL CHECK CALCULATIONS

– HAND CALCULATION – SHEARWALL #4



$$93.24^k \times \frac{0.3987}{0.7550} \times [1.6] = 78.8\ k$$

$$83.08^k \times \frac{0.6954}{1.2989} \times [1.6] = 71.2\ k$$

$$62.60k \times \frac{1.3661}{2.4847} \times [1.6] = 55.1\ k$$

$$44.59\ k \times \frac{3.3670}{5.66606} \times [1.6] = 42.4\ k$$

$$71.71k \times \frac{14.9254}{26.0365} \times [1.6] = 65.8\ k$$

SELF-WEIGHT: $(150) \times (1' \times 82' \times 29') = 357\ k$

$[1.2] \times 357 = 428\ k$

$$M_u = (78.8^k \times 82') + (71.2^k \times 67'\text{-}4") + (55.1^k \times 52'\text{-}8") + (42.4^k \times 38') + (65.8^k \times 20')$$

$$M_u = 17085\ ft \cdot k$$

$$V_u = 313\ k$$

$$P_u = 428\ k$$

– CHECK BOUNDARY ELEMENT REQUIREMENTS

$$A_{wall} = \left(29' \times \frac{12"}{ft}\right) \times 12" = 4176\ in^2$$

$$I_{wall} = \frac{12"\,(29' \times 12)^3}{12} = 42144000\ in^4$$

$$f_c = \frac{P_u}{A_{wall}} + \frac{M_u \frac{h_w}{2}}{I_{wall}} = \frac{428}{4176} + \frac{(17085 \times 12)\left(\frac{29 \times 12}{2}\right)}{42144000}$$

$$f_c = 0.949\ ksi \quad < \quad 0.2f'_c = 1\ ksi$$

∴ BOUNDARY ELEMENTS NOT REQ'D

– SHEAR CALCULATIONS

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

$$V_c = \text{MIN}\begin{cases} 3.3\sqrt{5000}\,(12)(303) + \dfrac{428000(303)}{4(348)} = 942\text{ k} \\ \left[0.6\sqrt{5000} + \dfrac{(348)\left(1.25\sqrt{5000} + 0.2\dfrac{428000}{(348)(12)}\right)}{\dfrac{17085\times 12}{313} - \dfrac{348}{2}}\right](12)(303) = 441\text{ k} \leftarrow \end{cases}$$

$\phi V_c = 0.75 \times 441 = 331\text{ k} > V_u = 313\text{ k}$ OK

$0.5\phi V_c < V_u$, ∴ SHEAR REINF. REQ'D PER ACI 318 11.10.9

– REINFORCEMENT – USE $\rho_l = \rho_t = 0.0025$

$0.0025 \le \dfrac{A_{s\,trans}}{A_{cv}}$, USE 12" STRIP

$0.0025 \le \dfrac{A_{s\,trans}}{12\times 12}$

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

#5 @ 18" O.C. EACH FACE: $A_{s\,trans} = 2\left[0.31\times\frac{12''}{18''}\right] = 0.41\ in^2/ft$ OK

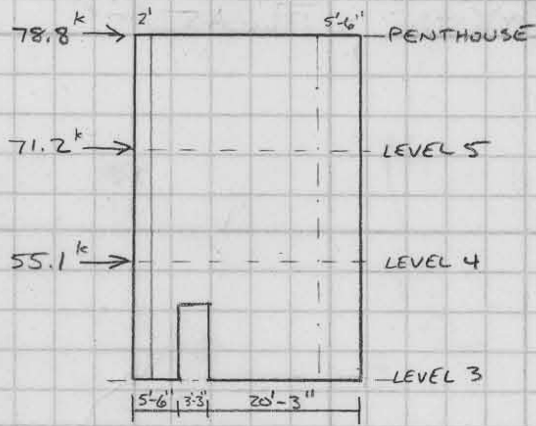
$0.0025 \le \dfrac{A_{s\,long}}{A_{cv}}$, USE 12" STRIP

$0.0025 \le \dfrac{A_{s\,long}}{12\times 12}$

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

#5 @ 18" O.C. EACH FACE: $A_{s\,long} = 2\left[0.31\times\frac{12''}{18''}\right] = 0.41\ in^2/ft$ OK

– AT LEVEL 3 OPENING:



$M_u = 6364$ ft·k

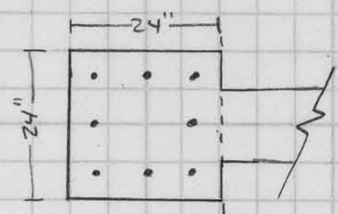
$V_u = 205$ k

$P_u = 229$ k

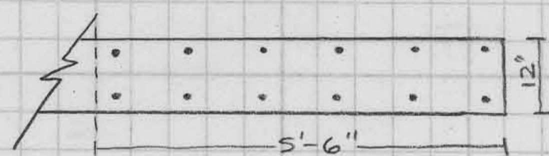
SIMPLIFIED METHOD: $C = 229 + \frac{6364}{25.25'} = 481$ k

$T = \frac{6364}{25.25} = 252$ k

BOUNDARY ELEMENTS:



(8) #8
(6.32 in²)



(12) #9
(12.00 in²)

– COMPRESSION –

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

$\phi P_n = 0.8(0.65)\left[0.85(5)(24\times24 - 6.32) + (60)(6.32)\right]$

$\phi P_n = 1456$ k > 481 k OK

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

$\phi P_n = 0.8(0.65)\left[0.85(5)(66\times12 - 12) + (60)(12)\right]$

$\phi P_n = 2098$ k > 481 k OK

– TENSION –

$\phi T_n = \phi F_y A_s$

$\phi T_n = (0.9)(60)(6.32)$

$\phi T_n = 341$ k > 252 k OK

$\phi T_n = \phi F_y A_s$

$\phi T_n = (0.9)(60)(12)$

$\phi T_n = 648$ k > 252 k OK

– SHEAR CALCULATIONS

• LEFT PIER: $h_w/l_w = 44'/5.5' = 8 \Rightarrow \alpha_c = 2.0$

$$V_n = A_{cv}\left(\alpha_c\sqrt{f'_c} + \rho_t f_y\right) \qquad \left\{\rho_t = \frac{0.41\ in^2}{12'' \times 12''} = 0.0028\right\}$$

$$V_n = (12'' \times 5'\text{-}6'')\left(2.0\sqrt{5000} + 0.0028(60000)\right)$$

$V_n = 245\ k$ MUST BE $\leq 10 A_{cw}\sqrt{f'_c} = 10(12 \times 5'\text{-}6'')\sqrt{5000} = 560\ k$ OK

• RIGHT PIER: $h_w/l_w = 44'/20.25' = 2.17 \Rightarrow \alpha_c = 2.0$

$$V_n = (12'' \times (20.25' \times 12))\left(2.0\sqrt{5000} + 0.0028(60000)\right)$$

$V_n = 902\ k$ MUST BE $\leq 10(12 \times (20.25 \times 12))\sqrt{5000} = 2062\ k$ OK

$V_n = 902 + 245 = 1147\ k$

MUST BE LESS THAN $8A_{cv}\sqrt{f'_c} = 8\left[(12'' \times 66'') + (12'' \times 243'')\right]\sqrt{5000}$
$= 2098\ k$ OK

$\phi V_n = 0.75 \times 1147 = 860\ k > V_u = 205\ k$ OK

$V_u = 205\ k$ DOES NOT EXCEED $A_{cv}\sqrt{f'_c} = 262\ k$

$\therefore$ MIN. SHEAR REINF. OK

#5 @ 18" EACH WAY, EACH FACE IS ADEQUATE.