Ingleside at King Farm

Errata for Technical Assignment # 3

Lateral System Analysis and Confirmation Design Report

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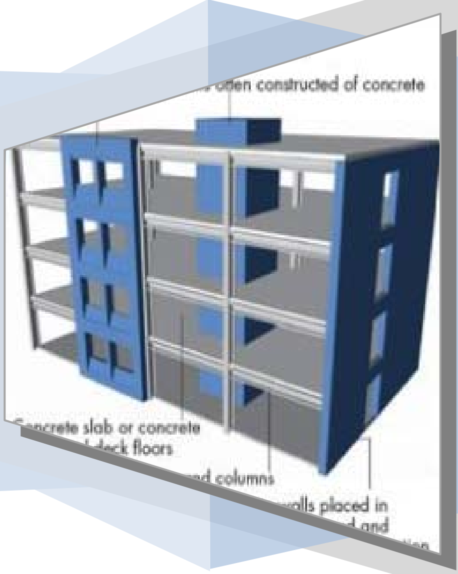

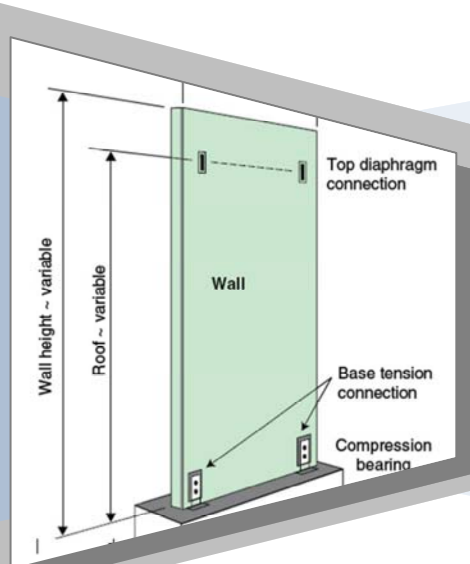

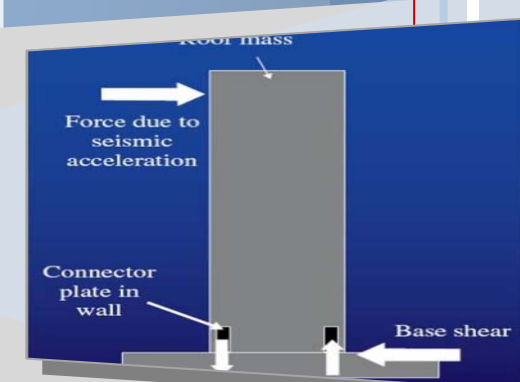

The Pennsylvania State University
Department of Architectural Engineering

Senior Thesis 2008-2009

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Executive Summary

This report is an erratum for Technical Report 3. There were mistakes found in the spreadsheet calculations that are corrected for this report. They include lateral drift calculations for wind and seismic, overturning moments, and detailed explanation of the load path, drift limitations used in the previous report, expansion joints, and the columns near the expansion joints. Much of the mistakes were due to miss referencing cells in the spreadsheets.

The controlling load combination this time was found to be 1.2*D* + 1.0*E* + *L* + 0.2*S.* After calculating the overturning moments due to both wind and seismic forces using un-factor loads, it was discovered that seismic forces contributed a greater overturning moment. Only load combination 5 and 7 of ASCE 7-05 section 2.3.2 includes seismic loads. Since there is no other noticeable lateral pressures such as earth, ground water, or bulk materials acting on the building, H is neglected. Thus, load combination 5 governs the strength design of this building. As the approximate square footage of the building is 790,000 square feet, dead load was the dominating factor.

After re-analyzing Ingleside at King Farm's lateral force system, it was found that the structure is adequate to resist the imposed wind and seismic forces. Serviceability and strength criteria of the most critical shear wall were analyzed. Despite shear wall 1 having to resist about 90% of the direct distributed seismic shear in the South/North direction; it met the drift limitations and strength requirements. This was mainly due to the assumed or self calculated displacement of the building section's center of mass, as this will greatly affect the torsional shear on the shear walls if the eccentricity is large enough. The distributed torsional shear for Shear wall 3 was large enough to counter much of the direct shear it had to resist. A computer model will later be used to see if the calculated displacement of center of mass was accurate.

The overturning moment was recalculated for the shear wall and was found to be adequate to resist the overturning moment. However, the safety factor was calculated to be 1.3, which is slightly below 1.5. In this case, some anchorage is recommended. This unexpected high amount of torsional shear will indeed affect a goal in the thesis proposal of achieving a central shear core lateral resisting system to reduce pre-stress losses due to concrete shortening. However, the placement of shear walls can still be optimized in other ways and layouts.

Structural Plans Clarification

Where there exist a 2” true expansion joint in the building, there is a row of double 12” x 30” columns as oppose to the typical 24” x 30” columns on each side of the joint.

Figure 1: Building Sections

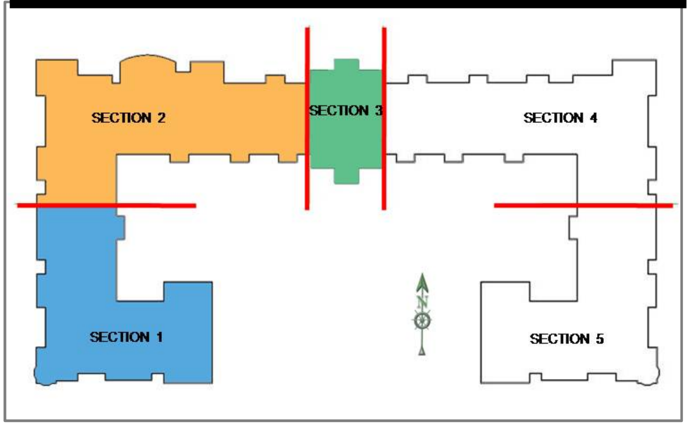


Figure 2: 2-inch continuous expansion joint detail

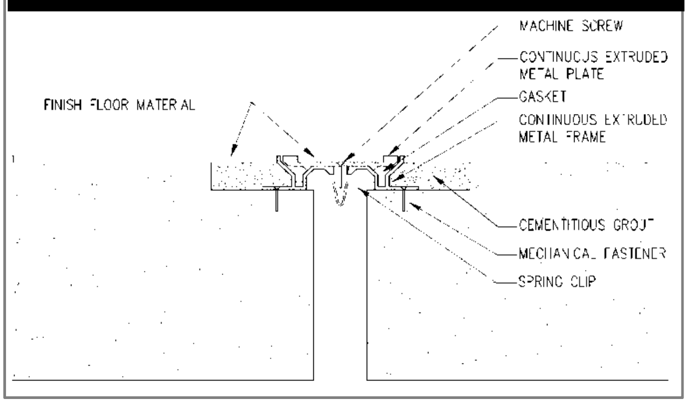


Figure 4: Exp. Joint between building Section 3 and 4

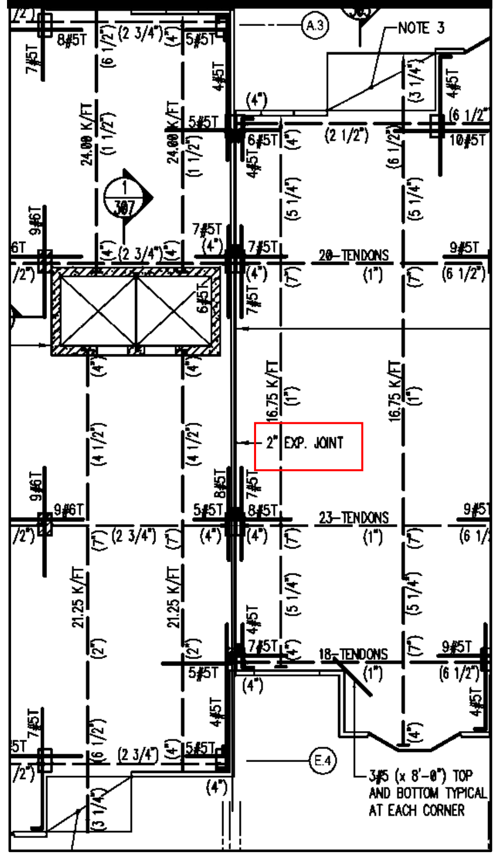
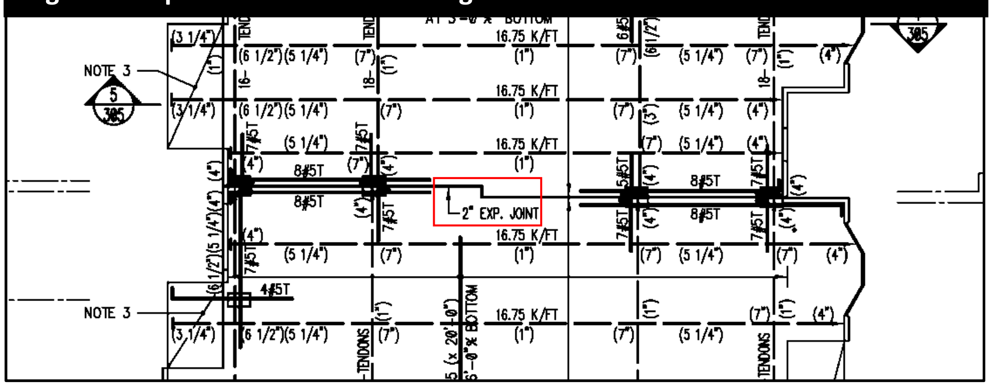


Figure 3: Exp. Joint between building Section 4 and 5



Recently, it was discovered that a double column (12” x 30”) is placed back to back, one on each side of an expansion joint.

Section 3 of the building was selected for lateral analysis due to its maximum building height of over 90 feet (highest parapet), while the other building sections' max heights range from 66 to 82.5 feet. The goal was to analyze the lateral system based on loads and conditions assumed for this thesis to see if it is adequate.

Load Combinations

The list below contains the seven load combinations, per ASCE 7-05 section 2.3.2 for strength design.

1. 1.4(*D* + *F*)
2. 1.2(*D* + *F* + *T*) + 1.6(*L* + *H*) + 0.5(*Lr* or *S* or *R*)
3. 1.2*D* + 1.6(*Lr* or *S* or *R*) + (*L* or 0.8*W*)
4. 1.2*D* + 1.6*W* + *L* + 0.5(*Lr* or *S* or *R*)
5. **1.2*D* + 1.0*E* + *L* + 0.2*S***
6. 6. 0.9*D* + 1.6*W* + 1.6*H*
7. 7. 0.9*D* + 1.0*E* + 1.6*H*

The controlling load combination is 1.2*D* + 1.0*E* + *L* + 0.2*S.* Due to the short nature of the building and over turning moment calculations using un-factor loads, seismic loads controlled in all directions over wind loads. Only load combination 5 and 7 includes seismic loads. Due to the lack of other lateral pressures (earth, ground water, or bulk materials), H is neglected. Thus, load combination 5 governs the strength design of this building.

Load Path and Distribution

As the lateral forces are distributed to each story, the load path is determined by relative stiffness. The forces are transmitted through the diaphragms to the shear walls and down to the footings. Diaphragms are assumed to be rigid. The columns only serve to carry gravity loads. All the shear walls are 12 inches thick and vary in length. Due to their different distances from the building's center of the rigidity and center of mass, the rigidity and stiffness of each shear wall in building section 3 is first calculated, followed by direct and torsional shear due to wind and seismic loads.

Location of CR (typical floor)

Figure 5: CR and shear walls location

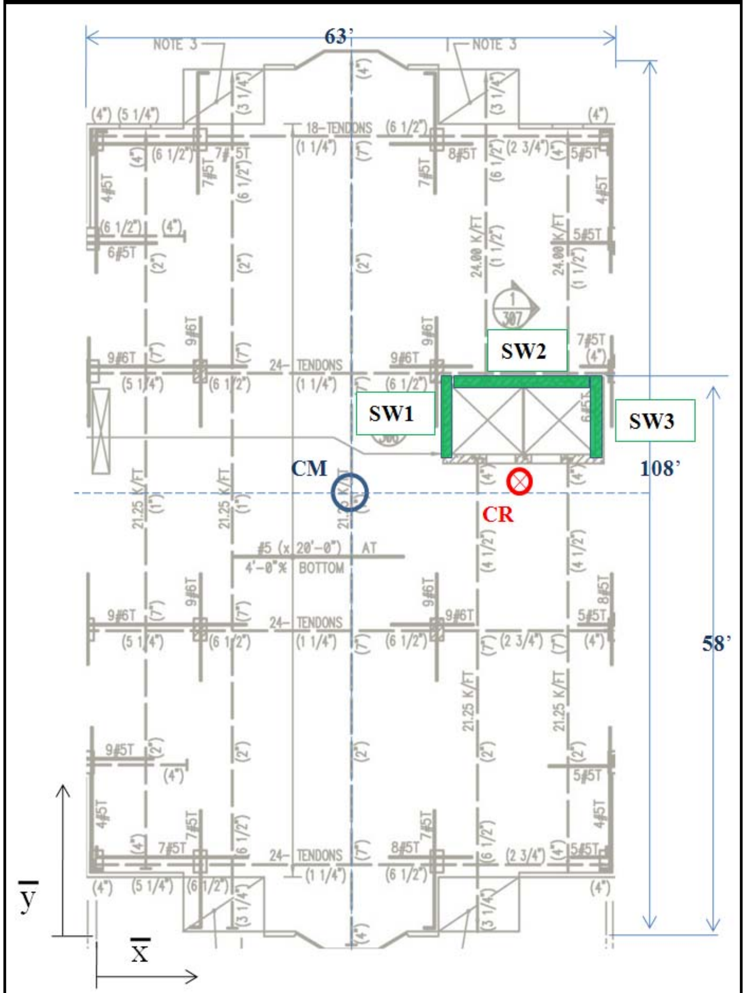
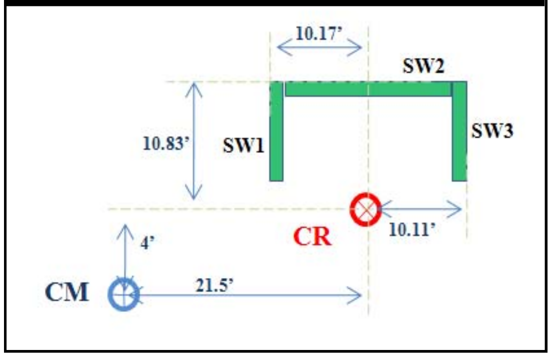


Figure 6: CR and relative distances to elements (not drawn fully to scale)



Equations:

Locate CR	$x=\Sigma x_i L_i/\Sigma L_i$ $y=\Sigma y_i L_i/\Sigma L_i$
Polar Moment of Inertia	$J=\Sigma K_i y_i + \Sigma K_i x_i$
Direct Shear	$V_{sw}=V_{diaph}*k_i/\Sigma K_i$
Torsional Shear	$Fsw=V_{diaph}*k_i*x_i*e_x/J$

See Appendix for detailed calculations.

Torsion

Torsion is found to be the controlling mode for building section 3 due to its center core shear walls and large floor diaphragms. The torsional shear for building section 3 calculated for South/North seismic is much greater than the East/West direction. This is due to the greater eccentricity distance difference between the center of mass and the center of rigidity for the North/South direction (e_x). Since the torsional moment $M_T = V_{Diaph}*e$, the torsional moment is also expected to be the greatest for the South/North direction seismic forces.

WIND: Shear Calculation: Y-Direction (S/N)

Level 2 Wind Story Force = 9.27

Element	Stiffness Coef.	Direct Shear	Torsional Shear	Total Shear
	Ki=Li			
SW1	9.5	4.635	18.935	23.570
SW2	20	-	0.000	0.000
SW3	9.5	4.635	4.452	0.183

Level 3 Wind Story Force = 8.11

Element	Stiffness Coef.	Direct Shear	Torsional Shear	Total Shear
	Ki=Li			
SW1	9.5	4.054	3.917	7.971
SW2	20	-	8.782	8.782
SW3	9.5	4.054	3.894	0.160

Level 4 Wind Story Force = 8.906

Element	Stiffness Coef.	Direct Shear	Torsional Shear	Total Shear
	Ki=Li			
SW1	9.5	4.453	4.303	8.756
SW2	20	-	9.646	9.646
SW3	9.5	4.453	4.277	0.175

Level 5 Wind Story Force = 9.3352

Element	Stiffness Coef.	Direct Shear	Torsional Shear	Total Shear
	Ki=Li			
SW1	9.5	4.668	4.510	9.178
SW2	20	-	10.112	10.112
SW3	9.5	4.668	4.484	0.184

Level 6 Wind Story Force = 10.657

Element	Stiffness Coef.	Direct Shear	Torsional Shear	Total Shear
	Ki=Li			
SW1	9.5	5.328	5.149	10.477
SW2	20	-	11.543	11.543
SW3	9.5	5.328	5.118	0.210

SIESMIC: Shear Calculation: Y-Direction (S/N)

Level 2 Seismic Story Force = 7

Element	Stiffness Coef.	Direct Shear	Torsional Shear	Total Shear
	Ki=Li			
SW1	9.5	3.572	3.451	7.023
SW2	20	-	7.738	7.738
SW3	9.5	3.572	3.431	0.141

Level 3 Seismic Story Force = 14

Element	Stiffness Coef.	Direct Shear	Torsional Shear	Total Shear
	Ki=Li			
SW1	9.5	6.875	6.643	13.519
SW2	20	-	14.894	14.894
SW3	9.5	6.875	6.604	0.271

Level 4 Seismic Story Force = 20.994

Element	Stiffness Coef.	Direct Shear	Torsional Shear	Total Shear
	Ki=Li			
SW1	9.5	10.497	10.143	20.640
SW2	20	-	22.740	22.740
SW3	9.5	10.497	10.083	0.414

Level 5 Seismic Story Force = 28.7171

Element	Stiffness Coef.	Direct Shear	Torsional Shear	Total Shear
	Ki=Li			
SW1	9.5	14.359	13.875	28.233
SW2	20	-	31.105	31.105
SW3	9.5	14.359	13.793	0.566

Level 6 Seismic Story Force = 38.769

Element	Stiffness Coef.	Direct Shear	Torsional Shear	Total Shear
	Ki=Li			
SW1	9.5	19.384	18.731	38.115
SW2	20	-	41.993	41.993
SW3	9.5	19.384	18.620	0.764

Level 7 Wind Story Force = 16.090

Element	Stiffness Coef.	Direct Shear	Torsional Shear	Total Shear
	Ki=Li			
SW1	9.5	8.045	7.774	15.819
SW2	20	-	17.428	17.428
SW3	9.5	8.045	7.728	0.317

Level 7 Seismic Story Force = 53

Element	Stiffness Coef.	Direct Shear	Torsional Shear	Total Shear
	Ki=Li			
SW1	9.5	26.2827	25.3968	51.6795
SW2	20	-	56.9368	56.9368
SW3	9.5	26.2827	25.2470	1.0358

Roof Wind Story Force = 8.9229

Element	Stiffness Coef.	Direct Shear	Torsional Shear	Total Shear
	Ki=Li			
SW1	9.5	4.461	4.311	8.773
SW2	20	-	9.665	9.665
SW3	9.5	4.461	4.286	0.176

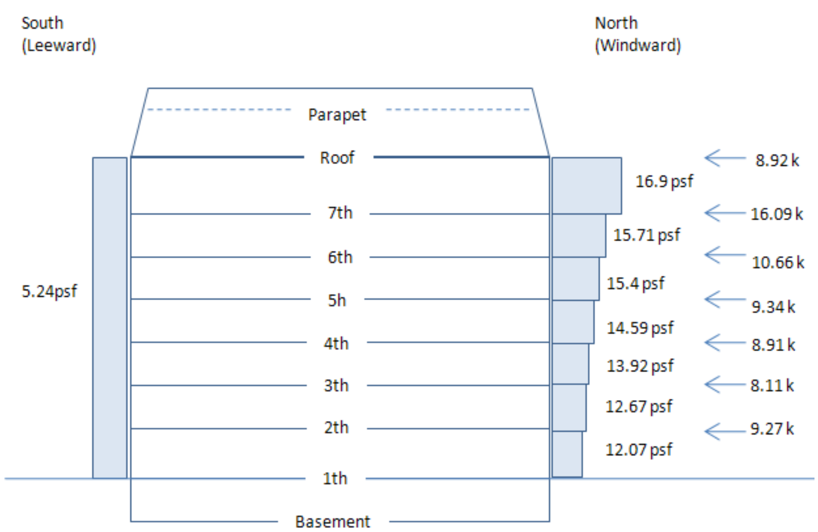
Roof Seismic Story Force = 12

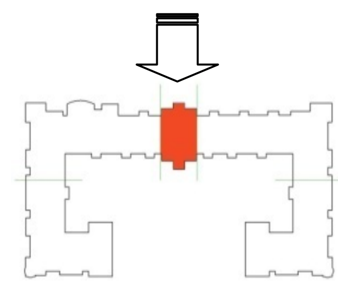
Element	Stiffness Coef.	Direct Shear	Torsional Shear	Total Shear
	Ki=Li			
SW1	9.5	6.055	5.851	11.905
SW2	20	-	13.116	13.116
SW3	9.5	6.055	5.816	0.239

Based on the calculations, it appears that shear wall 1 is required to resist more shear forces than the other shear walls. It will also experience a higher drift and torsional moment. More analysis of shear wall one is needed.

Distribution of Wind Loads and Overturning Moments

Wind: (North-South)									
Floor	Floor Height	Height from Ground (ft)	Tributary Height	Windward (psf)	Leeward (psf)	Total (psf)	Story Force (kip)	Story Shear (kips)	Overturning Moment (K-ft)
2	10.00	14.00	12.00	6.83	-5.24	12.07	9.27	62.36	873.11
3	10.00	24.00	10.00	7.43	-5.24	12.67	8.11	53.10	1274.29
4	10.00	34.00	10.00	8.68	-5.24	13.92	8.91	44.99	1529.58
5	10.00	44.00	10.00	9.35	-5.24	14.59	9.34	36.08	1587.61
6	12.00	54.00	11.00	9.90	-5.24	15.14	10.66	26.75	1444.33
7	14.50	66.00	16.00	10.48	-5.24	15.71	16.09	16.09	1061.94
Roof	16.50	82.50	8.25	11.27	-5.63	16.90	8.92	8.92	736.14
							Total =	248.29	8507.00





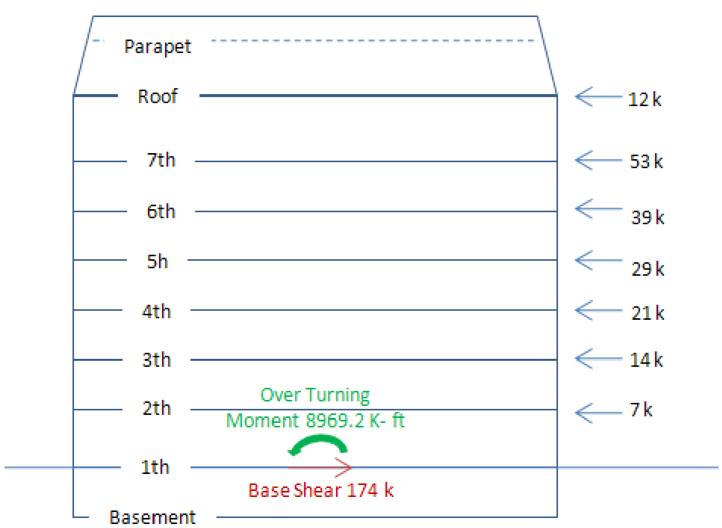
The wind loads were recalculated because of a discrepancy in assigning the floor heights in Technical Report 3.

Distribution of Seismic Loads and Overturning Moments

Ss	S1	Fa	Fv	Sms	Sm1	Sds	Sd1	I	R
0.16	0.05	1.00	1.00	0.16	0.05	0.10	0.03	1.00	4.00

Ta	TL	Cu	T	Ts	SDC	Cs	W total (kips)	Base Shear V (kips)	k
0.55	8.00	1.70	0.93	0.33	A	0.026	6694.19	174.05	1.21

Seismic Load								
Floor	h (ft)	Story Height per Floor	Tributary Height (ft)	w_x	w_xh^k	Cv_x	Story Force Fx (kips)	Overturning Moment (ft-kips)
2	14.00	10.00	12.00	895.80	22115.63	0.04	7	100
3	24.00	10.00	10.00	895.80	42569.52	0.08	14	330
4	34.00	10.00	10.00	895.80	64995.21	0.12	21	714
5	44.00	10.00	10.00	895.80	88904.53	0.16	29	1264
6	54.00	12.00	11.00	942.96	120023.16	0.22	39	2094
7	66.00	14.50	16.00	1001.91	162735.99	0.30	53	3469
Roof	82.50	16.50	8.25	176.00	37489.39	0.07	12	999
			Total =	5704.07	538833.43			8969.24



Comparing the over turning moments for both wind and seismic, seismic caused a greater over turning moment than wind. Thus, **1.2*D* + 1.0*E* + *L* + 0.2*S*** is the governing load combination for strength design as compared with 0.9*D* + 1.0*E* + 1.6*H*.

Deflection and Story Drift Calculations

Since story drift and displacement is a serviceability issue, factored loads were not used in any story drift calculations. Seismic drift will be limited by the allowable story drift in Table 12.12-1 from ASCE7-05 while story drift for wind will be limited to L/400 for wind.

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

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

[a] h_{sx} is the story height below Level x.
[b] For seismic force–resisting systems comprised solely of moment frames in Seismic Design Categories D, E, and F, the allowable story drift shall comply with the requirements of Section 12.12.1.1.
[c] There shall be no drift limit for single-story structures with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the story drifts. The structure separation requirement of Section 12.12.3 is not waived.

Drift calculations will be calculated using the equation:

$$\Delta\ cant = (Ph3/3EI) + (1.2Ph/ErA)$$

Drift due to Flexure   Drift due to Shear

Wind Drift (North/South – forces resisted by SW1 and SW3)

d (in)	I (in^4)	E_c (ksi)	E_r (ksi)	A (in^2)
114	1481544	4030	1612	1368

SW 1 Drift N/S Direction

Floor	SW 1 Distributed Shear	Story Height	Story Height (in)	Story Drift Due to Flexure (in)	Story Drift Due to Shear (in)	Total Story Drift (in)	Building Drift (in)	L/400 (in)
2	23.57	14.0	168	0.006	0.002	0.008	0.008	-
3	7.97	24.0	288	0.011	0.001	0.012	0.020	-
4	8.76	34.0	408	0.033	0.002	0.035	0.055	-
5	9.18	44.0	528	0.075	0.003	0.078	0.133	-
6	10.48	54.0	648	0.159	0.004	0.163	0.296	-
7	15.82	66.0	792	0.439	0.007	0.446	0.742	-
Roof	8.77	82.5	990	0.475	0.005	0.480	1.222	2.475

SW 3 Drift N/S Direction

Floor	SW 3 Distributed Shear	Story Height	Story Height (in)	Story Drift Due to Flexure (in)	Story Drift Due to Shear (in)	Total Story Drift (in)	Building Drift (in)	L/400 (in)
2	0.18	14.0	168	0.000	0.000	0.000	0.000	-
3	0.16	24.0	288	0.000	0.000	0.000	0.000	-
4	0.18	34.0	408	0.001	0.000	0.001	0.001	-
5	0.18	44.0	528	0.002	0.000	0.002	0.003	-
6	0.21	54.0	648	0.003	0.000	0.003	0.006	-
7	0.32	66.0	792	0.009	0.000	0.009	0.015	-
Roof	0.18	82.5	990	0.010	0.000	0.010	0.024	2.475

Shear wall 1 experienced the largest total drift of 1.2 inches at the roof level. The drift limitation is L/400 = 2.475 inches. Shear Wall 1 is adequate to resist N/S directional winds. As the other two building sections adjacent to building section 3 are separated by 2-inch expansion joints, no drift analysis for wind in the E/W is required. Because of the torsion shear, it had reduced the distributed direct shear to shear wall 3. This leads to shear wall 1 resisting almost all the direct shear in the South/North direction. Manual spot check calculations were done for the direct and torsional shear for level 2 and it supports the data in the spread sheets.

Seismic Drift

d (in)	I (in^4)	E_c (ksi)	E_r (ksi)	A (in^2)
114	1481544	4030	1612	1368

SW 1 Drift S/N Direction

Floor	**SW 1 Distributed Shear**	**Story Height**	**Story Height (in)**	**Story Drift Due to Flexure (in)**	**Story Drift Due to Shear (in)**	**Total Story Drift (in)**	**Building Drift (in)**	**$0.02h_{sx}$ (in)**
2	7.02	14.00	168	0.002	0.001	0.003	0.003	2.4
3	13.52	24.00	288	0.018	0.002	0.020	0.023	2.4
4	20.64	34.00	408	0.078	0.005	0.083	0.105	2.4
5	28.23	44.00	528	0.232	0.008	0.240	0.346	2.4
6	38.12	54.00	648	0.579	0.013	0.592	0.938	2.4
7	51.68	66.00	792	1.433	0.022	1.456	2.394	2.88
Roof	11.91	82.50	990	0.645	0.006	0.651	3.045	3.48

SW 3 Drift S/N Direction

Floor	SW 3 Distributed Shear	Story Height (ft)	Story Height (in)	Story Drift Due to Flexure (in)	Story Drift Due to Shear (in)	Total Story Drift (in)	Building Drift (in)	$0.02h_{sx}$ (in)
2	0.14	14.00	168.0	0.000	0.000	0.000	0.000	2.4
3	0.27	24.00	288.0	0.000	0.000	0.000	0.000	2.4
4	0.41	34.00	408.0	0.002	0.000	0.002	0.002	2.4
5	0.57	44.00	528.0	0.005	0.000	0.005	0.007	2.4
6	0.76	54.00	648.0	0.012	0.000	0.012	0.019	2.4
7	1.04	66.00	792.0	0.029	0.000	0.029	0.048	2.88
Roof	0.24	82.50	990.0	0.013	0.000	0.013	0.061	3.48

d (in)	I (in^4)	E_c (ksi)	E_r (ksi)	A (in^2)
240	13824000	4030	1612	2880

SW 2 Drift E/W Direction

Floor	SW 2 Distributed Shear	Story Height (ft)	Story Height (in)	Story Drift Due to Flexure (in)	Story Drift Due to Shear (in)	Total Story Drift (in)	Building Drift (in)	$0.02h_{sx}$ (in)
2	8.583144983	14.00	168.00	0.00024	0.00037	0.00062	0.00062	2.4
3	16.52136172	24.00	288.00	0.00236	0.00123	0.00359	0.00421	2.4
4	25.22484474	34.00	408.00	0.01025	0.00266	0.01291	0.01712	2.4
5	34.50412519	44.00	528.00	0.03039	0.00471	0.03510	0.05222	2.4
6	46.58136411	54.00	648.00	0.07584	0.00780	0.08364	0.13585	2.4
7	63.15834616	66.00	792.00	0.18774	0.01293	0.20066	0.33652	2.88
Roof	14.54974983	82.50	990.00	0.08447	0.00372	0.08819	0.42471	3.48

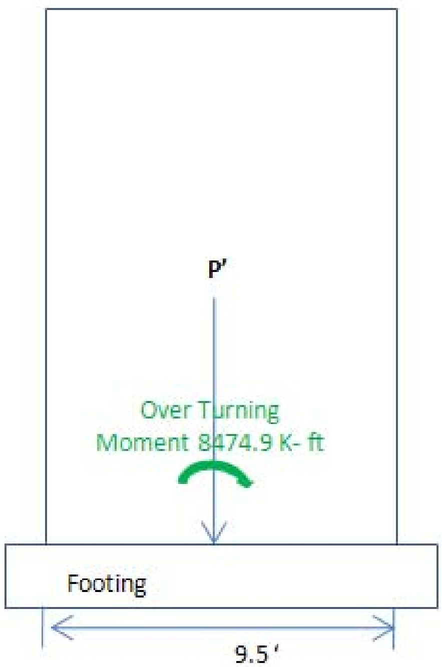
Comparing the drift values for each shear wall and with the 0.02 h_{sx} limitation per story drift for seismic, all the shear walls met their drift criteria. Shear wall 2, which resist shear in the E/W direction had a max drift at the roof level of 0.424 inches, which is acceptable compared with the 2-inch expansion joints that separates the building section. In the future, the other building sections will be analyzed using a computer model for time convenience.

Over Turning Moment

The resisting moment of shear wall 1 was calculated to be greater than the over turning moment. However, the safety factor is less than 1.5 and thus requires tie down or some type of anchorage.

Investigation of Shear Wall 1

$P' = 1.2D + 1.0E + L + 0.2S = 2351.5k$
(includes footing self weight)



Floor	SW 1 Distributed Shear	Story Height	Overturning Moment (ft-k)
2	7.02	14.00	98.322
3	13.50	24.00	324.000
4	10.60	34.00	360.400
5	28.20	44.00	1240.800
6	38.10	54.00	2057.400
7	51.70	66.00	3412.200
Roof	11.90	82.50	981.750
		Total=	8474.872

$M_{resisting} = 2351.5(9.5/2) = 11169.6$ k-ft
$M_{resisting} = 11169.63 > 8474.87$ ok

SF = 11169.63/8474.87 = 1.32 < 1.5

Requires tie down

See Appendix for tie down requirement

Conclusion

The analysis of Ingleside at King Farm lateral force system reveals that the structure is adequate to resist the imposed wind and seismic forces. Serviceability and strength criteria of the most critical shear wall were analyzed, and had proved to be efficient. There is a slight variation between the hand calculated torsional forces distributed to shear wall 1 and the spread sheet by 10%. However, it does support the results of shear wall 1 having to resist approximately 90% of the seismic shear forces in the south/north direction, and still be under the drift limitation of $0.02h_s$. Torsional shear was a huge impact on the lateral system. Perhaps redesigning the system as a semi flexible diaphragm will help mitigate the torsion impact. A computer model will later be used to see if the calculated displacement of center of mass was accurate.

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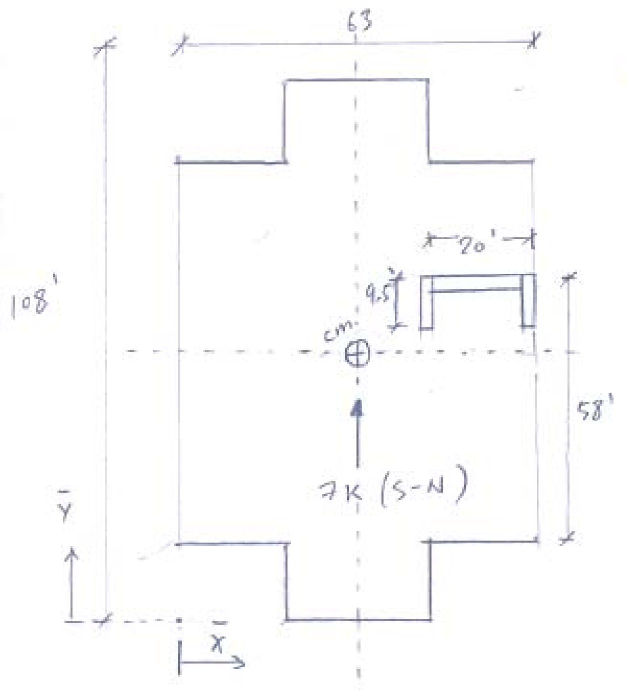
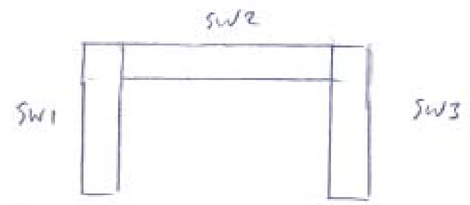
CR SPOT CHECK

$V_{Diaph} = 7\ k$





① Locate CR

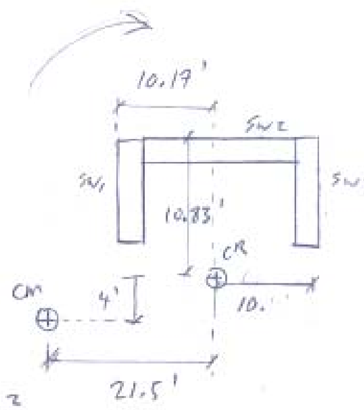
$$\bar{x} = \frac{\Sigma x_i L_i}{\Sigma L_i} = \frac{\overset{SW1}{(63-20)(9.5)} + \overset{SW3}{63(9.5)}}{9.5 + 9.5} = 53'$$

$$\bar{y} = \frac{\Sigma y_i L_i}{\Sigma L_i} = \frac{\overset{SW2}{58(20)}}{20} = 58$$

② Eccentricity

$$e_x = \bar{x} - \frac{63}{2} = 53 - 31.5 = 21.5'$$

$$e_y = \bar{y} - \frac{108}{2} = 58 - 54 = 4'$$



③ Polar moment of Inertia $: J = \Sigma k_i y_i^2 + \Sigma k_i x_i^2$

$$J = 9.5\underbrace{(10.17')^2}_{SW_1\ dist.\ from\ CR} + 20\underbrace{(10.83')^2}_{SW_2\ dist\ from\ CR} + 9.5\underbrace{(10.11')^2}_{SW\,3\ dist\ from\ CR}$$

$$J = 4299.368\ k\text{-}ft$$

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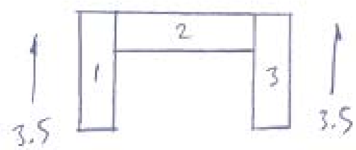
④ Direct shear (goes into walls // to direction of shear)

$$V = V_{Diaph} \cdot \frac{K_1}{K_1 + K_2}$$

$$V_{SW1} = \frac{(7)(9.5)}{9.5+9.5} = 3.5\text{ K}$$

$$V_{W3} = \frac{(7)(9.5)}{9.5+9.5} = 3.5\text{ K}$$



⑤ Torsional shear

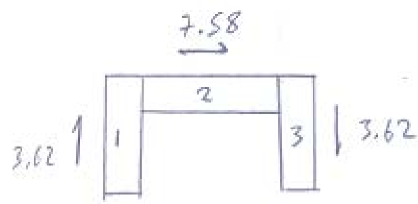
dist to CR

$$F = K_1 x_1 \cdot \frac{V_{Diaph} \cdot e_x}{J} \; (N\text{-}S)$$

$$F_{SW1} = 9.5\frac{(10.17)(7)(21.5)}{4299.37} = 3.62$$

$$F_{SW2} = 20\frac{(10.83)(7)(21.5)}{4299.37} = 7.58$$

$$F_{SW3} = 9.5\frac{(10.11)(7)(21.5)}{4299.37} = 3.62$$



⑥ Total shear (S-N)

$$V_{SW1} = 3.5 + 3.62 = 7.12\text{ K}$$

$$V_{SW2} = 7.58$$

$$V_{SW3} = 3.5 - 3.62 = -0.12$$

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OVERTURNING MOMENTS (SHEAR WALL)

Governing Load Combination: $1.2D + 1.0E + L + 0.2S$

$E = E_h \pm E_v$ (AISC 7-05 eq 12.4.1)

$E_h = \rho Q_E$ (AISC 7-05 Eq 12.4.3)

↳ effect of horz. seismic shear force V, or F_p

$E_v = 0.2S_{DS}D$

$(1.2 + 0.2S_{DS})D + \rho Q_E + L + 0.2S$

$\rho = 1.0$ (AISC 12.3.4.1)
$S_{DS} = 0.10$ (from previous tech report calc.)
$Q_E = 174.05$ (base shear)
$L = 193.75$ kips (from previous tech report calc.)
$S = 625\ sf(10\ psf)\frac{1}{1000} = 6.25\ K$
$DL = 1171.925\ K$

Footing self weight: (size $26' \times 34' \times 41''$)

$FTG\ SW = \left(\frac{41''}{12''}\right)(26)(34)(150)\left(\frac{1}{1000}\right) = 453.05\ K$

$D = 1171.925 + 453.05 = 1624.98\ K$

$(1.2 + 0.2(0.1))\,1624.98 + (1.0)(174.05) + 193.75 + 0.2(6.25)$

$\boxed{P' = 2351.5\ K}$

Total overturning moment for SW1:

$OTM = 7.02(14) + 15.5(24) + 10.6(34) + 27.2(44) + 38.1(54) + 51.7(66) \ldots$
$+ (11.9)(82.5) = 8474.87\ ^{\prime K}$

$\boxed{M' = 8474.87\ ^{\prime K}}$

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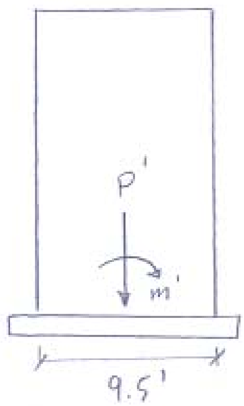


Resisting Moment $= P'\left(\frac{9.5}{2}\right)$

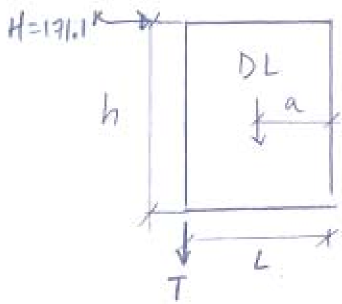
$M_{resisting} = 2351.5^k\left(\frac{9.5'}{2}\right) = 11169.63'^k$

$M_{resisting} = 11169.63 > 8474.87'^k$ ok ✓

$SF = \frac{11169.63}{8474.87} = 1.32 < 1.50$ NG

∴ Although FTG is not subjected to uplift, it requires by UBC that the safety factor is at least 1.5

- Tie down is needed



$DL(a) + T(L) = 1.5\,(H)(h)$

$2351.5\,(4.75) + T(9.5) = 1.5\,(171.1^k)(82.5')$

$T = 1053.1^k$ (tie down requirement)

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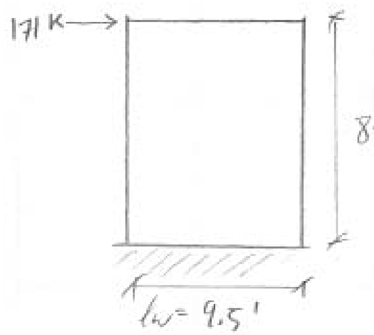
SHEAR WALL 1 SHEAR CHECK



$F'_c = 5{,}000$ psi
$F_y = 60{,}000$ psi
$h = 12''$

$V_u = 171^k$
$N_u = 0$ assume 0

Check max permitted shear strength

$V_u \le \phi V_n = \phi(10)\sqrt{f'_c}\, b\, d$ (ACI 11.10.3)

$d = 0.8\, l_w = 0.8(9.5)(12'') = 91.2''$

$\phi V_n = 0.75(10)\sqrt{5000}(12)(91.2) = 580^k > 171^k$ ok ✓

Shear strength by V_c

$V_{c\,max} = 2\sqrt{f'_c}\, h\, d = 2\sqrt{5000}\,(12'')(91.2'')\frac{1}{1000} = 154.8^k$

$\frac{1}{2}\phi V_c = \frac{1}{2}(0.75)(154.8) = 58.0^k$

$V_u = 171^k > 58.0^k$ ∴ need reinforcements (ACI 11.10.9)

$V_u \le \phi V_n = \phi(V_c + V_s)$

$171 = 0.75(154.8 + V_s)$

$V_s = 73.2^k$

$\frac{A_v}{s} = \frac{V_s}{f_y d} = \frac{73.2}{(60)(91.2)} = 0.0133\ in^2/in$

$A_v \ge 12''(0.0133) = 0.161\ in^2$

$\Rightarrow A_v$ provid $= (2)\#3 = 2 \times (0.11) = .22\ in^2 > A_{v,min}$ OK ✓

Building Section 3							
Ground Floor	(pcf or psf)	Thickness (in)	Height (ft)	Perimeter (ft)	Area (sf)	Amount	Load (k)
Slab	150	8			5500		550
Ext. Wall	40		14	162			90.72
Shear Wall	150		14	39			81.9
Partition	20				5500		110
Columns	150		14		3.75	20	157.5
						Total =	**990.12**
2nd-5th Floor	(pcf or psf)	Thickness (in)	Height (ft)	Perimeter (ft)	Area (sf)	Amount	Load (k)
Slab	150	8			5500		550
Ext. Wall	40		10	162			64.8
Shear Wall	150		10	39			58.5
Partition	20				5500		110
Columns	150		10		3.75	20	112.5
						(per floor)	**895.8**
6th Floor	(pcf or psf)	Thickness (in)	Height (ft)	Perimeter (ft)	Area (sf)	Amount	Load (k)
Slab	150	8			5500		550
Ext. Wall	40		12	162			77.76
Shear Wall	150		12	39			70.2
Partition	20				5500		110
Columns	150		12		3.75	20	135
						Total =	**942.96**
7th Floor	(pcf or psf)	Thickness (in)	Height (ft)	Perimeter (ft)	Area (sf)	Amount	Load (k)
Slab	150	8			5500		550
Ext. Wall	40		14.5	162			93.96
Shear Wall	150		14.5	39			84.825
Partition	20				5500		110
Columns	150		14.5		3.75	20	163.125
						Total =	**1001.91**
Roof	(pcf or psf)	Thickness (in)	Height (ft)	Perimeter (ft)	Area (sf)	Amount	Load (k)
Framing	15				5500		82.5
Roofing	17				5500		93.5
						Total =	176
						Total Weight =	6694.19

Section 3 - Shear Wall 1								
Ground Floor	(pcf or psf)	Thickness (in)	Height (ft)	Perimeter (ft)	Area (sf)	Amount	D Load (k)	L Load (k)
Slab	150	8			625		62.5	
Self wt.	150		14	39			81.9	
Partition	20				625		12.5	
						Total =	156.9	25
2nd-5th Floor	(pcf or psf)	Thickness (in)	Height (ft)	Perimeter (ft)	Area (sf)	Amount	D Load (k)	L Load (k)
Slab	150	8			625		62.5	
Ext. Wall	150		10	39			58.5	
Partition	20				625		12.5	
						(per floor)	133.5	25
6th Floor	(pcf or psf)	Thickness (in)	Height (ft)	Perimeter (ft)	Area (sf)	Amount	D Load (k)	L Load (k)
Slab	150	8			625		62.5	
Self wt.	150		12	39			70.2	
Partition	20				625		12.5	
						Total =	145.2	25
7th Floor	(pcf or psf)	Thickness (in)	Height (ft)	Perimeter (ft)	Area (sf)	Amount	D Load (k)	L Load (k)
Slab	150	8			625		62.5	
Self wt.	150		14.5	39			84.825	
Partition	20				625		12.5	
						Total =	159.825	25
Roof	(pcf or psf)	Thickness (in)	Height (ft)	Perimeter (ft)	Area (sf)	Amount	D Load (k)	L Load (k)
Framing	15				5500		82.5	
Roofing	17				5500		93.5	
						Total =	176	18.75
						Total Weight =	1171.925	193.75