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Technical Report 3 – Lateral Systems

Executive Summary

The Hershey Academic Support Center sports a composite floor design on each floor of the building. The main lateral system for this building is varying moment connections located at almost every column. These connections extend to all 5 floors of the buildings and brace the building in both the N-S and the E-W conditions. The moment connections in the building are only for partially restraining the lateral movement, but for the purpose of computer analysis, these connections were assumed to be fully restrained. Wind and Seismic values were transferred from Technical Report 1 and obtained from ASCE 7-02.

This Technical Report contains a complete lateral analysis of the Hershey Academic Support Center. Lateral forces were distributed by finding the individual stiffness of each moment frame in the building. This stiffness was then used to distribute direct and torsional shear forces throughout the building. To find the deflection and story drift, a model of each moment frame was created in SAP2000 and the portal method was used to check the computer deflection values. These drift values were checked against the criteria of H/400 and passed for both the individual members and the structure as a whole. The building was also checked for overturning and strength, both of which passed analysis. Lastly, three members were spot checked to see if the proposed design matched up with the loads calculated. One member was taken from each of the three building sections and the overall design came out exactly the same or very close.

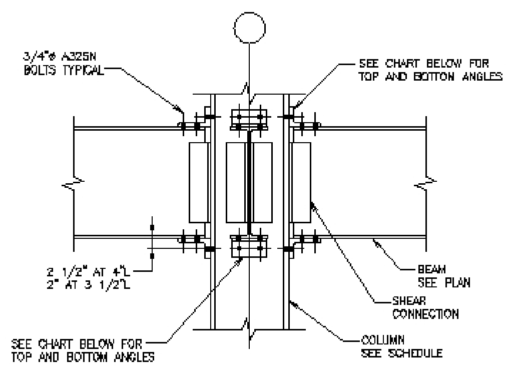
By the values that were calculated, it could be argued that "Type 2 with Wind" design was used because it proposed members that were just slightly smaller than those calculated by the seismic design. It is my opinion that this is indeed the case with my structure, judging by the varying member's sizes per bay and the fact that shear studs were used in the composite floor design. The change in the controlling lateral load is most likely attributed to the wind factor code change from 1.7 when the original design was done and my redesign with the new wind factor of 1.6.

General Information

The Hershey Academic Support Center is part of the Hershey Medical Center complex and is owned by The Pennsylvania State University. Constructed from March 1999 to August 2000, The Penn State Geisinger Health System was designed as the primary occupant, but was dissolved before the building was occupied. Currently the building is used for auxiliary purposes of the Hershey Medical Center and accommodates 680 people. The building itself can be considered in two sections, an East and a West wing. The wings are structurally identical with the only difference between them found in the center section. The building footprint encompasses a total area of 150,000 square feet. The total height of the building over 5 stories is measured as 56'-0" with the height to top of the roof including the Mechanical Penthouse being 69'-0". The building consists of a conventional structural steel system with composite beam floor framing and a precast concrete and glass facade. Moment connections placed at the columns as well as braced steel frames help to resist the wind and lateral loads throughout the building.

Lateral System

The Hershey Academic Support Center sports a composite floor design on each floor of the building. The main lateral system for this building is varying moment connections located at almost every column. These connections extend to all 5 floors of the buildings and brace the building in both the N-S and the E-W conditions. The top floor does not utilize moment connections in the E-W direction, but uses Cross Bracing to help prevent the lateral load instead. There are 3 different moment connections used and with bolt combinations, it comes to 16 total types. The three types of connections used are top & bottom angles, top & bottom plates, and top angles & bottom plates. These connections use different bolt numbers to add strength where needed and the most common connection used in a typical bay is a L6 x 4 x 7/8 x 0'7" steel angle with 4 bolts to a girder and 2 bolts to a column. The moment connections in the building are only for partially restraining the lateral movement, but for the purpose of computer analysis, these connections were assumed to be fully restrained. Moment connections were used in the building most likely to maintain the spaces architecturally and create lateral uniformity throughout the building. The building is also only 5 stories tall, which helped to make the moment connections a more practical design.



WIND MOMENT CONNECTION–
TOP AND BOTTOM ANGLES
MC–1 THRU MC–10

SEE PLAN FOR LOCATIONS

DETAIL A

NOT TO SCALE

Load Cases

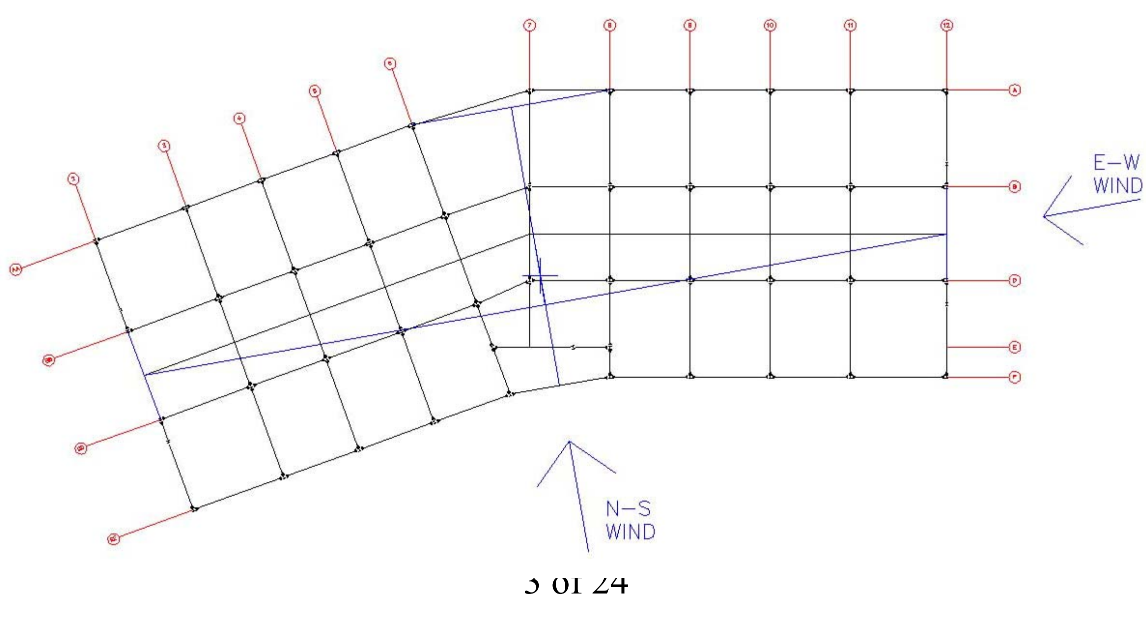
My building was designed by the BOCA 1996 code, but to bring things up to date, as with the past Tech Reports, ASCE 7-02 will be used to analyze the structure. From ASCE 7-02, there are 7 controlling load cases:

~Case #1: 1.4D
~Case #2: 1.2D + 1.6L + 0.5S
~Case #3: 1.2D + 1.6S + 0.8W
~Case #4: 1.2D + 1.6W + 0.5L + 0.5S
~Case #5: 1.2D + 1.0E + 0.5L + 0.2S
~Case #6: 0.9D +1.6W
~Case #7: 0.9D + 1.0E

For this assignment, the moment braced frames were checked for lateral forces by finding the biggest seismic moment and comparing it to the largest wind moment multiplied by the load factor of 1.6. From the comparison, it was found that the Seismic Load force controlled the design in both the North-South and East-West directions. This means that load Case #5 should be used to check the foundations and that load Case #7 would be used to check uplift and overturning. From the initial analysis, it was believed that this building was designed as "Type 2 with Wind" which would account for the differing in member sizes per bay and the number of shear studs that were used. For this reason, both Seismic and "Type 2 with Wind" Designs were considered and compared.

Lateral Design

For the lateral design, full Wind and Seismic loads calculations were completed and compiled in the first Technical Report as seen in the Appendix. SAP2000 was used to analyze each moment frame individually in the building. Using a 1k force at the top of the each frame structure, story deflections were found and then converted into stiffness values by the equation Stiffness (K) = 1/deflection (Δ). When combined, these stiffnesses give the load distribution for the moment frame, the floor, and the total section as well. The center of rigidity and wind direction are shown below.



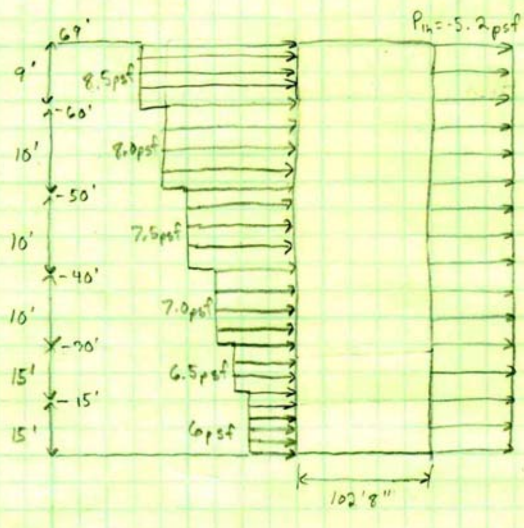
Wind Loads

z (ft)	K_z	q_z	(P_{wz}) N-S	(P_{lh}) N-S	(P_{tot}) N-S	(P_{wz}) E-W	(P_{lh}) E-W	(P_{tot}) E-W
0-15	0.85	9.06304	6.079937	-5.21265	11.29259	6.257873	-3.21912	9.476997
20	0.9	9.59616	6.437581	-5.21265	11.65023	6.625984	-3.21912	9.845107
25	0.94	10.02266	6.723695	-5.21265	11.93635	6.920472	-3.21912	10.1396
30	0.98	10.44915	7.00981	-5.21265	12.22246	7.21496	-3.21912	10.43408
40	1.04	11.0889	7.438982	-5.21265	12.65163	7.656692	-3.21912	10.87582
50	1.09	11.62202	7.796626	-5.21265	13.00928	8.024802	-3.21912	11.24393
60	1.13	12.04851	8.08274	-5.21265	13.29539	8.319291	-3.21912	11.53841
70	1.17	12.47501	8.368855	-5.21265	13.58151	8.613779	-3.21912	11.8329

	N-S	E-W
Story Shear @ 0	21.21098	6.811023
Story Shear @ 1	43.07454	13.87904
Story Shear @ 2	46.18385	15.10357
Story Shear @ 3	48.39108	15.97283
Story Shear @ 4	50.09928	16.64556
Story Shear @ 5	35.30126	10.81774

The charts shown above summarize the results found from my wind calculation analysis. Shown below is the wind loading for a typical building wall as well as story forces. Specific calculations of wind forces are located in the Appendix.

Wind Load Distribution



Find Story Forces

0-15':
(11.3)(28') = 316.4 psf
(316.4)(14') = 4.43 k

15-30:
(12.2)(28') = 341.6 psf
(341.6)(14') = 4.78 k

30-40
(12.7)(28') = 355.6 psf
(355.6)(14') = 4.98 k

40-50
(13.0)(28') = 364.0 psf
(364)(14') = 5.10 k

50-60
(13.3)(28') = 372.4 psf
(372.4)(14') = 5.21 k

60-69
(13.6)(28') = 380.8 psf
(380.8)(13') = 4.95 k

Given on the right is point force on the walls as well as wall distribution.

Seismic Loads

Hershey 5					
Vertical Distribution N-S					
Level	w_x	h_x	$w_x h_x^k$	C_{vx}	F_x
1	2195.5809	14	47409.8578	0.072521	30.8688
2	2195.5809	28	106249.519	0.162527	69.1796
3	2195.5809	42	170346.004	0.260573	110.9131
4	2195.5809	56	238114.2	0.364236	155.0374
5	662.50927	69	91616.8137	0.140143	59.65217
Value Sum			653736.394	1	425.6511
Base Shear					425.6511
Overturning Moment					46910.63

Hershey 5					
Vertical Distribution E-W					
Level	w_x	h_x	$w_x h_x^k$	C_{vx}	F_x
1	2195.5809	14	47409.8578	0.072521	30.8688
2	2195.5809	28	106249.519	0.162527	69.1796
3	2195.5809	42	170346.004	0.260573	110.9131
4	2195.5809	56	238114.2	0.364236	155.0374
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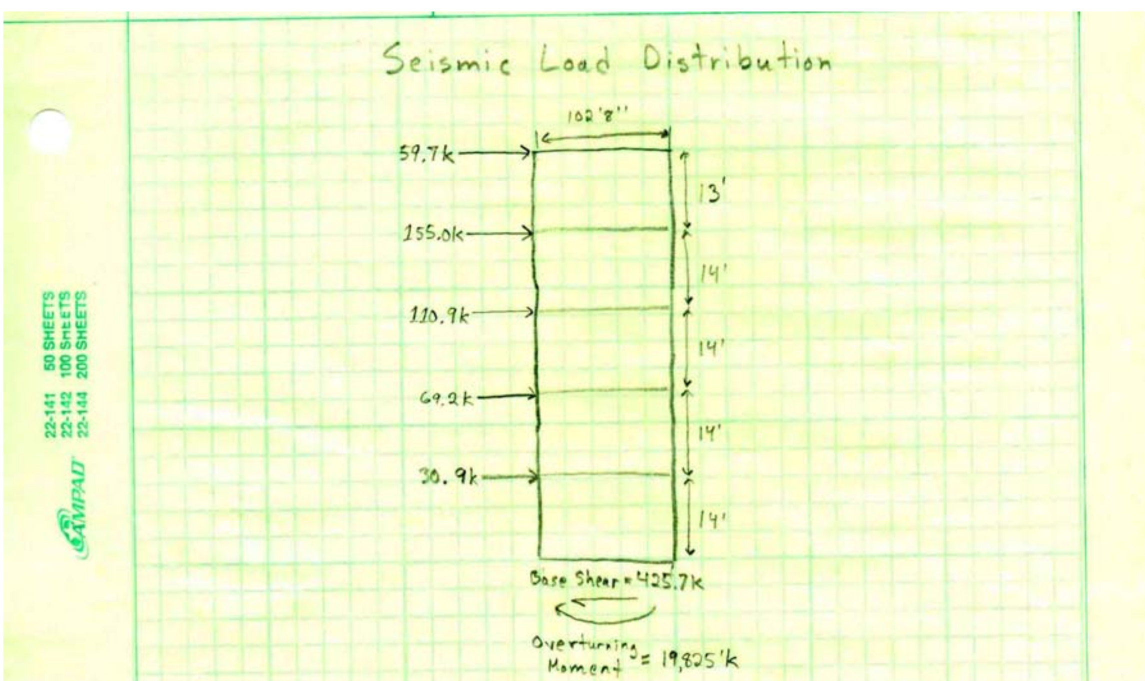
Overturning Moment					46910.63

The charts shown above summarize the results found from my seismic calculation analysis. Shown below is the seismic loading for a typical building as depicted by story forces. Specific calculations of seismic forces are located in the Appendix.



Load Distribution

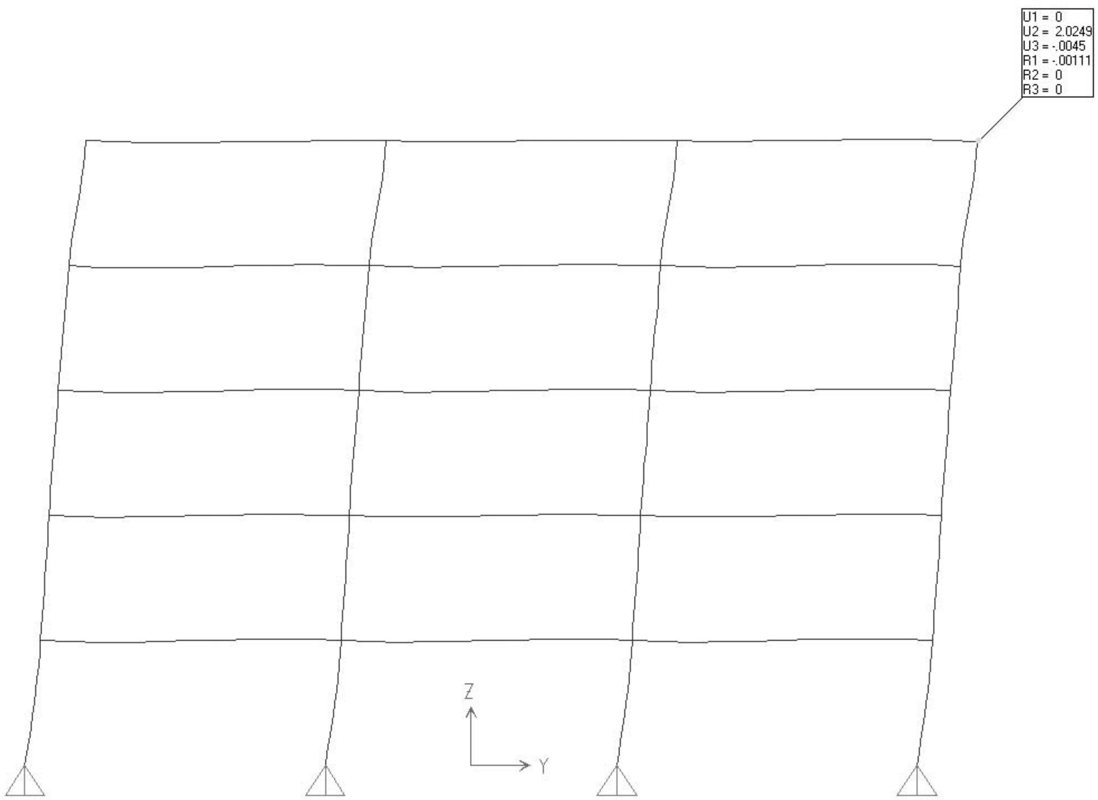
For each moment frame, load is distributed from both shear and torsional components. The direct shear on each moment frame is calculated by relative stiffness of the frame. This stiffness is determined by the equation: relative stiffness (q) = stiffness (k)/ sum of stiffnesses (Σk). This stiffness was the multiplied by the highest story shear in order to confirm that every floor was designed to capacity. The center of rigidity was found using the moment frame system and AutoCAD as seen in the drawing below. Next, an eccentricity for the moment frame relative to the entire building was found and this was applied with the max shear force to create a torsional moment. This torsional moment was distributed over the frame per foot of area and a torsional shear was found. As shown below, the torsional shear was negligible to the direct shear so issues caused by torsion need not be worried about.

Direct Shear & Torsion

Moment Frame	Direct Shear	Torsional Shear N-S	Torsional Shear E-W
East N-S #7	12.50264047	0.001502676	0.007892037
East N-S #8	16.25543606	0.017416909	0.010523465
East N-S #9	13.33761414	0.059019706	0.016543145
East N-S #10	12.73133578	0.130110969	0.023742049
East N-S #11	13.21300535	0.24396674	0.033000768
East N-S #12	11.35352398	0.35897486	0.037035884
East E-W #A	25.14869633	0.165296882	0.161328559
East E-W #B	34.15331921	0.177330101	0.082921837
East E-W #D	44.39125585	0.208860386	0.005361055
East E-W #F	19.1356619	0.178671708	0.178961217
West N-S #2	11.35473044	0.350031356	0.087969572
West N-S #3	13.21300535	0.240540431	0.052386464
West N-S #4	12.73133578	0.130386662	0.022168976
West N-S #5	13.35649357	0.061208016	0.004628227
West N-S #6	20.98830728	0.025782719	0.005086008
West E-W #AA	24.70589385	0.387695017	0.132492287
West E-W #BB	35.33712194	0.408137265	0.007974769
West E-W #DD	33.8579128	0.317460678	0.123148093
West E-W #FF	16.90116658	0.144833282	0.138769961
Center E-W #A	7.09961593	0.017101546	0.04117169
Center E-W #B	14.86178526	0.013738652	0.01916161
Center E-W #D	7.962006001	0.002293879	0.001021465
Center E-W #E	6.55837958	0.000980076	0.006489761
Center E-W #F	4.499756576	0.001859453	0.010539642

Deflection and Drift

To calculate the story drift and total drift of the building, a SAP2000 model was used. First, a force of 1k was placed at the top of every moment frame to determine which one would have the highest deflection in each of the three sections (East, West, and Center). One moment frame was also checked to verify that the computer results were accurate. The value obtained was almost exactly the same as the computer deflection, so this method of analysis can be assumed to be accurate. After this was determined, the appropriate Direct Shear and Torsional force was applied to the frame with the biggest deflection and compared to the allowable H/400 drift criteria. The results are below along with a visual example of the East Section:



Deflection Calculation H/400: ((69')*(12in/ft))/400 = 2.07in
East Section Frame #12: Story Drift = 2.02in < 2.07in ALLOW
West Section Frame #2: Story Drift = 1.91in < 2.07in ALLOW
Center Section Frame #D: Story Drift = 1.83in < 2.07in ALLOW

As you can see above, each frame section passed the drift comparison and is adequately designed for drift. Since the worse case in all three sections passed the H/400 drift requirement, it is safe to say that the overall section would also pass this requirement. The building is therefore acceptable in both total drift and story drift values.

Overturning and Uplift

To calculate the overturning moment, seismic forces were considered because they control the design. Each floor was analyzed by the appropriate seismic force multiplied by the distance of the base. The total moment resulting from this calculation is considered to be the Overturning Moment of the structure. This moment is then compared to the weight of the structure (found in the Seismic Calculations, W = 9444k). The overturning moment is taken against the short side of building since that value is critical between the two. The calculation produced this table and result:

Overturning (Seismic)			
Story #	Story Shear (k)	Distance From Base (ft)	Story Moment (ft-k)
Floor 1	425.65	14	5959.1
Floor 2	397.78	28	11137.84
Floor 3	325.6	42	13675.2
Floor 4	214.69	56	12022.64
Floor 5	59.65	69	4115.85
Overturning Moment			46910.63
Overturning Force			469.1063

E-W Controlling Section: 469k < 9444k ALLOW

Clearly, the weight of the structure is more than enough to hold down the structure from uplift. The N-S Section has a considerably bigger surface area, so it can be deduced that it will not experience uplift from the overturning moment either.

Strength Check

To check the strength requirement in the lateral systems, two columns in critical sections were chosen to represent the structure. The first column is used at connection B between the East section and the Center section. The second column is used at connection D between the West section and the Center section. To compare the values, the equation Pu/b + Mu/m < 1 was used. Both columns are located on the first floor and stiffness values were used to obtain the specific axial and moment force exerted on the column. Table 6-2 from the Steel Manual was used to obtain the b and m values for each column.

Section B:
W14x193, Pu/b + Mu/m = (166.37)/(0.47) + (387.34)/(0.668) = 0.934 < 1 ALLOW

Section D:
W14x175, Pu/b + Mu/m = (173.08)/0.516) + (403.23)/(0.741) = 0.880 < 1 ALLOW

Both columns possess enough strength to overcome the forces that are placed on them.

Member Spot Check

Member spot checks were performed to ensure that the lateral system was designed to hold the controlling lateral force. All the checks were done for the 2nd story of the building, and the lateral force was distributed appropriately by stiffness and tributary area. Three checks in all were performed to represent the building.

Check #1
For the first member spot check, I chose a typical bay in the Moment Frame East #10 to try and confirm if in fact the Seismic loads controlled the design or if "Type 2 with Wind" controlled. As a quick side note, the basic principal for Type 2 with Wind design is to take the negative moment value from the wind force and use this when designing the lateral force member. This method ensures that the lateral force will be adequately resisted with the structure, but can often result in varied member types throughout the building. Another factor that could be attributed to Type 2 with Wind is that shear studs are used to help adjust the balance between the positive moment in the center of a normal gravity load distribution and the negative moment located at the ends. This creates an issue where economy must be considered to pick a member that has an optimum girder size to shear stud ratio.

Seismic Force on one member: Mu = 331.06'k
Projected Member Design: W21x44 where øMn = 358'k
Wind Force on one member: Mu = 277.56'k
Projected Member Design: W18x40 where øMn = 294'k
Actual Member Design: W21x44

This member comparison shows that the controlling seismic load case was right on target. The wind force was close to being accurate and if the old design factor of 1.7 had been applied, a closer member would have been chosen.

Check #2
For the second spot check, I chose the Moment Frame West #A to represent an E-W member in design. All appropriate tributary areas were applied, giving the results:

Seismic Force on one member: Mu = 248.30'k
Projected Member Design: W18x35 where øMn = 249'k
Wind Force on one member: Mu = 180.22'k
Projected Member Design: W16x31 where øMn = 203'k
Actual Member Design: W18x35

Again, the controlling case produced the correct member design with the wind slightly lower. Aside from the change in factors from 1.7 to 1.6, another issue for design could be the number of shear studs needed in each member. It is possible a larger member was chosen so that less shear studs needed to be used.

Check #3
The last spot check was chosen from the center section because that is the critical section for the lateral case. When considered alone, this section must take the most lateral force with the least overall tributary area. The specific frame that was chosen is the Moment Frame Center #B and the values are as follows:

Seismice Force on one member: 595.91'k
Projected Member Design: W24x68 where øMn = 664'k
Wind Force on one member: 507.94'k
Projected Member Design: W24x62 where øMn = 578'k
Actual Member Design: W24x76

This time, the member design in the structure was greater than necessary by both the seismic and wind calculations. Both forces produce reasonable results and the bigger member was most likely used because this part of the building rests in a critical section. It is also relevant to note that shear studs were not used for this particular section of the building as a revision. This change could be attributed to a number of reasons, such as convenience to the stairwell construction or possibly a time issue involved in labor, but whatever the case may be, it is likely that the larger member sizes and the lack of shear studs is connected.

Conclusion

The design of the lateral system has met all checks and passed all requirements. Torsion, drift, overturning, and strength were all produced passing values. The design was ultimately controlled by seismic forces from Case #5: 1.2D + 1.0E + 0.5L + 0.2S. By the values calculated, it could be argued that "Type 2 with Wind" design was used and that issues such as old design codes caused the seismic forces to control instead.

Appendix

Wind Loading Calculations

	Hershey 5	Assumptions and Information
(K_{zt}) Topographic Factor	1	Code 6.5.7.2, Figure 6-4, $K_{zt} = (1+(k_1)*(k_2)*(k_3))^2$
(K_d) Directional Wind Factor	0.85	Code 6.5.5.4, Table 6-4
(V) Basic Wind Speed	70	Given by Structural Notes
(I) Importance Factor	1	Code 6.5.5, Table 6-1
(C_t) Peroid Parameter	0.02	Code 9.5.3.2, Table 9.5.5.3.2
(h) Building Height in Feet	69	Height to the 5th Story
(f) Frequency in Hz	2.08849378	Code 9.5.3.2, Table 9.5.5.3.2, f = 1/((C_t)*((h)^0.75))
Exposure Category C	α	Given by Structural Notes
(α)	9.5	Code 6.5, Table 6-2
(zg (ft))	900	Code 6.5, Table 6-2
(^a)	2/19	Code 6.5, Table 6-2
(^b)	1	Code 6.5, Table 6-2
(α bar)	1/6	Code 6.5, Table 6-2
(b bar)	0.65	Code 6.5, Table 6-2
(c) alsdrj	0.2	Code 6.5, Table 6-2
(L (ft))	500	Code 6.5, Table 6-2
(Є bar)	1/5	Code 6.5, Table 6-2
(z min)	15	Code 6.5, Table 6-2
Rigid Structures N-S		
*Exposure C, Table 6-2		
(g_q) Gust Coefficient	3.4	Code 6.5.8.2, Equation 6-8
(g_v) Gust Coefficient	3.4	Code 6.5.8.2, Equation 6-8
(z bar) Wind Coefficient	41.4	Code 6.5.8.2, Table 6-2, z bar = 0.6(h)
(L_z) Turbulence Scale Factor	523.199457	L_z = L*((z bar)/33)^(Є bar)
(I_z) Turbulence Intensity	0.19258196	Code 6.5.8.1, Equation 6-5, Iz = (c)*(33/(z bar))^(1/6)
(B) Perpendicular to Wind	268.33	Code 6.3, Given in Plan
(L) Parallel to Wind	102.67	Code 6.3, Given in Plan
(Q) Background Response	0.82260391	Code 6.5.8.1, Equation 6-6, Q = SQRT(1/(1+(0.63*((B+h)/L_z)^0.63)))
(G) Gust Factor	0.83856209	Code 6.5.8.2, Equation 6-8, G = 0.925*((1+(1.7*(g_q)*(I_z)*(Q)))/(1+(1.7*(g_v)*(I_z))))
Rigid Structures E-W		
(B)	102.67	
(Q)	0.8729702	
(G)	0.86310353	
Flexible Structures N-S		
*Exposure B, Table 6-2		

(g_q)	3.4	
(g_v)	3.4	
(z bar)	41.4	
(L_z)	523.199457	
(I_z)	0.19258196	
(B) Perpendicular to Wind	268.33	
(L) Parallel to Wind	102.67	
(Q)	0.82260391	
(β) Damping Ratio	0.05	Code 6.3, Section 9
(n_1) Natural Frequency	2.08849378	Code 6.5.8.2
(V_z) Mean Hourly Wind Speed	69.3038272	Code 6.5.8.2, Equation 6-14, V_z = ((b bar)*(B40/33)^(α bar)*(V)*(88/60)
(η_h) R_l Coefficient	9.5649541	Code 6.5.8.2, Equation 6-13, η_h = 4.6*(n_1)*(h)/(V_z)
(η_B) R_l Coefficient	37.1965816	Code 6.5.8.2, Equation 6-13, η_B = 4.6*(n_1)*(B)/(V_z)
(η_L) R_l Coefficient	47.6475145	Code 6.5.8.2, Equation 6-13, η_L = 4.6*(n_1)*(L)/(V_z)
(R_h) R_l Coefficient	0.09908316	Code 6.5.8.2, Equation 6-13, R_h = (1/η_h)-(1/(2*(η_h^2)))*(1-(2.718281828^(-2*η_h)))
(R_B) R_l Coefficient	0.02652281	Code 6.5.8.2, Equation 6-13, R_h = (1/η_B)-(1/(2*(η_B^2)))*(1-(2.718281828^(-2*η_B)))
(R_L) R_l Coefficient	0.02076722	Code 6.5.8.2, Equation 6-13, R_h = (1/η_L)-(1/(2*(η_L^2)))*(1-(2.718281828^(-2*η_L)))
(N_1) Reduced Frequency	15.7667889	Code 6.5.8.2, Equation 6-12, N_1 = (n_1*L_z)/V_z
(R_n) Resonance Coefficient	0.0241168	Code 6.5.8.2, Equation 6-11, R_n = (7.47*N_1)/((1+(10.3*N_1))^(5/3))
(R) Resonance Response Factor	0.02615683	Code 6.8.5.2, Equation 6-10, R = (1/β)*R_n*R_h*R_B*(0.53+(0.47*R_L)))
(g_R) Gust Coefficient	4.36152676	Equation 6-9, g_R = (SQRT((2*(LN(3600*n_1))))+(0.577/(SQRT((2*LN(3600*n_1))))))
(G_f) Gust Factor	0.8388954	Equation 6-8, G_f = 0.925*((1+(1.7*I_z*(SQRT(((g_q)^2)*((Q)^2)+((g_R)^2)*((R)^2)))))/(1+(1.7*g_v*I_z
Flexible Structures E-W		
*Exposure B, Table 6-2		
(B) Perpendicular to Wind	102.67	
(L) Parallel to Wind	268.33	
(Q)	0.8729702	
(η_B)	14.2323744	
(η_L)	124.527686	
(R_B)	0.06779395	
(R_L)	0.0079981	
(R)	0.04158559	
(G_f)	0.863897	
(Cp) Windward	0.8	Code 6.5.11.2, Figure 6-6
(Cp) Leeward N-S	-0.5	Code 6.5.11.2, Figure 6-6, L/B
(Cp) Leeward E-W	-0.3	Code 6.5.11.2, Figure 6-6, L/B
(q_z)*K_z Velocity Pressure	10.6624	Code 6.5.10, Equation 6-15, (qz)*Kz = 0.00256*K_{zt}*K_d*(V^2)*I
(q_h) Velocity Pressure at z	12.4323584	Code 6.5.12.2, Table 6-3, q_h = ((h-C131)/(C132-C131))*(A132-A131)*((qz)*Kz)+(((qz)*Kz)*A
(P_{wz})*q_z N-S	0.69048283	(Pwz)*qz = (Cp Windward)*G
(P_{wz})*q_z E-W	0.67111632	(Pwz)*qz = (Cp Windward)*G_f

Leeward Wind Pressure

(P_{lh}) N-S	-5.2126522	$P_{lh} = q_h$*(Cp Leeward N-S)*G
(P_{lh}) E-W	- 3.21912374	$P_{lh} = q_h$*(Cp Leeward E-W)*G_f

Windward Pressure N-S

(P_{wz}) 0-15	6.07993738
(P_{wz}) 20	6.43758076
(P_{wz}) 25	6.72369546
(P_{wz}) 30	7.00981016
(P_{wz}) 40	7.43898221
(P_{wz}) 50	7.79662559
(P_{wz}) 60	8.08274029
(P_{wz}) 70	8.36885499

Windward Pressure E-W

(P_{wz}) 0-15	6.25787348
(P_{wz}) 20	6.62598369
(P_{wz}) 25	6.92047185
(P_{wz}) 30	7.21496002
(P_{wz}) 40	7.65669226
(P_{wz}) 50	8.02480247
(P_{wz}) 60	8.31929063
(P_{wz}) 70	8.6137788

K_z	q_z	z (ft)
0.85	9.06304	0-15
0.9	9.59616	20
0.94	10.022656	25
0.98	10.449152	30
1.04	11.088896	40
1.09	11.622016	50
1.13	12.048512	60
1.17	12.475008	70

Total Pressure N-S — $P = P_{wz} + P_{lh}$

(P_{tot}) 0-15	11.2925896
(P_{tot}) 20	11.650233
(P_{tot}) 25	11.9363477
(P_{tot}) 30	12.2224624
(P_{tot}) 40	12.6516344
(P_{tot}) 50	13.0092778
(P_{tot}) 60	13.2953925
(P_{tot}) 70	13.5815072

Total Pressure E-W		$P = P_{wz} + P_{lh}$
(P_{tot}) 0-15	9.47699723	
(P_{tot}) 20	9.84510743	
(P_{tot}) 25	10.1395956	
(P_{tot}) 30	10.4340838	
(P_{tot}) 40	10.875816	
(P_{tot}) 50	11.2439262	
(P_{tot}) 60	11.5384144	
(P_{tot}) 70	11.8329025	
Leeward Shear N-S		
(B) Perpendicular to Wind	268.33	
	-	
Shear @ Ground	9790.97675	
	-	
Shear @ Floors	19581.9535	
	-	
Shear @ Roof	9091.62127	
Leeward Shear E-W		
(B) Perpendicular to Wind	102.67	
	-	
Shear @ Ground	2313.55204	
	-	
Shear @ Floors	4627.10408	
	-	
Shear @ Roof	2148.29833	
Winward Shear N-S		
(B) Perpendicular to Wind	268.33	
Shear @ 0	11420.0072	
Shear @ 1	23492.5862	
Shear @ 2	26601.8991	
Shear @ 3	28809.1274	
Shear @ 4	30517.3301	
Shear @ 5	15719.304	
Windward Shear E-W		
(B)Perpendicular to Wind	102.67	
Shear @ 0	4497.47109	
Shear @ 1	9251.94054	
Shear @ 2	10476.4621	
Shear @ 3	11345.7212	
Shear @ 4	12018.4522	
Shear @ 5	6190.63668	

Seismic Loading Calculations

	Hershey 5	
Seismic Use Group I		Table 9.1.3
Site Classification D		Table 9.4.1.2
Seismic Design Category B		Given in Structural Notes
Number of Stories	5	Mechanical Penthouse is considered as 6th floor
(h) Height	69	From Plans
(L) Building Length	268.33	From Plans
(B) Building Width	102.67	From Plans
Roof Dead Load	15	See Calculations
Snow Load	21	See Calculations
Floor Dead Load	70	See Calculations
Exterior Wall Load	30	Assumed
(S_s)	0.23	Figure 9.4.1.1(a)
(S_1)	0.07	Figure 9.4.1.1(b)
(F_a)	1.6	Table 9.4.1.2.4(a) Site Classification D
(F_v)	2.4	Table 9.4.1.2.4(b) Site Classification D
(S_{MS})	0.368	$S_{MS} = S_s*F_a$
(S_{M1})	0.168	$S_{M1} = S_1*F_v$
(S_{DS})	0.2453333	$S_{DS} = (2/3)*S_{MS}$
(S_{D1})	0.112	$S_{D1} = (2/3)*S_{M1}$
(w_{roof})	662.50927	$w_{roof} = ((L*B*DL_{roof})+(6*WL_{ext}*((2*L)+(2*B)))+(0.2*L*B*SL))/1000$
(w_{floors})	2195.5809	$w_{floors} = ((L*B*DL_{floor})+(12*WL_{ext}*((2*L)+(2*B))))/1000$
(W)	9444.8328	$W = w_{roof} +((5 -1)*w_{floors})$
(R)	3	Table 9.5.2.2, Ordinary Composite Moment Frame
(I)	1	Table 9.1.4, Seismic Use Group I
(T_a) N-S	0.8283947	Table 9.5.5.3.2, T_a (N-S) = $C_t*((h)^{\wedge}(x))$, Steel Moment Frame
(T_a) E-W	0.8283947	Table 9.5.5.3.2, T_a (E-W) = $C_t*((h)^{\wedge}(x))$, Steel Moment Frame
(C_s)	0.0817778	$C_s = (S_{DS})/(R/I)$
($C_{s\ max}$) N-S	0.0450671	$C_{s\ max}$ (N-S) = $(S_{D1})/(T*(R/I))$
($C_{s\ max}$) E-W	0.0450671	$C_{s\ max}$ (E-W) = $(S_{D1})/(T*(R/I))$
($C_{s\ min}$)	0.0107947	$C_{s\ min} = 0.044*(I)*(S_{DS})$
(V) N-S	425.65108	V (N-S) = C_s*W
(V) E-W	425.65108	V (E-W) = C_s*W
(k) N-S	1.1641973	k (N-S) = 1+((T-0.5)/2)
(k) E-W	1.1641973	k (N-S) = 1+((T-0.5)/2)

Hershey 5					
Vertical Distribution N-S					
Level	w_x	h_x	$w_x h_x^k$	C_{vx}	F_x
1	2195.5809	14	47409.8578	0.072521	30.8688
2	2195.5809	28	106249.519	0.162527	69.1796
3	2195.5809	42	170346.004	0.260573	110.9131
4	2195.5809	56	238114.2	0.364236	155.0374
5	662.50927	69	91616.8137	0.140143	59.65217
Value Sum			653736.394	1	425.6511
Base Shear					425.6511
Overturning Moment					19825.64

Hershey 5					
Vertical Distribution E-W					
Level	w_x	h_x	$w_x h_x^k$	C_{vx}	F_x
1	2195.5809	14	47409.8578	0.072521	30.8688
2	2195.5809	28	106249.519	0.162527	69.1796
3	2195.5809	42	170346.004	0.260573	110.9131
4	2195.5809	56	238114.2	0.364236	155.0374
5	662.50927	69	91616.8137	0.140143	59.65217
Value Sum			653736.394	1	425.6511
Base Shear					425.6511
Overturning Moment					19825.64

Stiffness Calculations – East Section

East N-S #7		
Floor	Displacement	Stiffness
5	0.1389	7.199424
4	0.113	8.849558
3	0.0897	11.14827
2	0.0665	15.03759
1	0.0435	22.98851
East N-S #8	Displacement	Stiffness
Floor		
5	0.1222	8.183306
4	0.0947	10.55966
3	0.0716	13.96648
2	0.0499	20.04008
1	0.0312	32.05128
East N-S #9	Displacement	Stiffness
Floor		
5	0.1481	6.752194
4	0.1111	9.0009
3	0.0835	11.97605
2	0.0612	16.33987
1	0.0392	25.5102
East N-S #10		
Floor	Displacement	Stiffness
5	0.1591	6.285355
4	0.1152	8.680556
3	0.0884	11.31222
2	0.0635	15.74803
1	0.041	24.39024
East N-S #11	Displacement	Stiffness
Floor		
5	0.1557	6.422608
4	0.1116	8.960573
3	0.0855	11.69591
2	0.0612	16.33987
1	0.0392	25.5102
East N-S #12	Displacement	Stiffness
Floor		
5	0.1684	5.938242
4	0.1261	7.930214
3	0.0982	10.1833
2	0.0708	14.12429
1	0.0475	21.05263

East E-W #A		
Floor	Displacement	Stiffness
5	0.0901	11.09878
4	0.0604	16.55629
3	0.0463	21.59827
2	0.0329	30.39514
1	0.0194	51.54639
East E-W #B		
Floor	Displacement	Stiffness
5	0.0675	14.81481
4	0.0438	22.83105
3	0.0338	29.5858
2	0.0241	41.49378
1	0.0144	69.44444
East E-W #D		
Floor	Displacement	Stiffness
5	0.0548	18.24818
4	0.0333	30.03003
3	0.0254	39.37008
2	0.0183	54.64481
1	0.0112	89.28571
East E-W #F		
Floor	Displacement	Stiffness
5	0.1199	8.340284
4	0.0793	12.61034
3	0.0602	16.6113
2	0.0431	23.20186
1	0.0256	39.0625

Stiffness Calculations – West Section

West N-S #2		
Floor	Displacement	Stiffness
5	0.1684	5.938242
4	0.126	7.936508
3	0.0982	10.1833
2	0.0708	14.12429
1	0.0475	21.05263
West N-S #3		
Floor	Displacement	Stiffness
5	0.1557	6.422608
4	0.1116	8.960573
3	0.0855	11.69591
2	0.0612	16.33987
1	0.0392	25.5102
West N-S #4		
Floor	Displacement	Stiffness
5	0.1591	6.285355
4	0.1152	8.680556
3	0.0884	11.31222
2	0.0635	15.74803
1	0.041	24.39024
West N-S #5		
Floor	Displacement	Stiffness
5	0.149	6.711409
4	0.1107	9.033424
3	0.0849	11.77856
2	0.0608	16.44737
1	0.0389	25.70694
West N-S #6		
Floor	Displacement	Stiffness
5	0.0853	11.72333
4	0.0661	15.12859
3	0.0527	18.97533
2	0.039	25.64103
1	0.0263	38.02281

West E-W #AA		
Floor	Displacement	Stiffness
5	0.0889	11.24859
4	0.065	15.38462
3	0.049	20.40816
2	0.0333	30.03003
1	0.0193	51.81347
West E-W #BB		
Floor	Displacement	Stiffness
5	0.0589	16.97793
4	0.0424	23.58491
3	0.0328	30.4878
2	0.0236	42.37288
1	0.0141	70.92199
West E-W #DD		
Floor	Displacement	Stiffness
5	0.0611	16.36661
4	0.0437	22.8833
3	0.0341	29.32551
2	0.0247	40.48583
1	0.0148	67.56757
West E-W #EE		
Floor	Displacement	Stiffness
5	0.0997	10.03009
4	0.0755	13.24503
3	0.592	1.689189
2	0.0428	23.36449
1	0.0251	39.84064

Stiffness Calculations – Center Section

Center E-W #A		
Floor	Displacement	Stiffness
1	0.027	37.03704
Center E-W #B		
Floor	Displacement	Stiffness
5	0.1556	6.426735
4	0.1019	9.813543
3	0.0773	12.93661
2	0.0554	18.05054
1	0.033	30.30303
Center E-W #D		
Floor	Displacement	Stiffness
5	0.2794	3.579098
4	0.2002	4.995005
3	0.1511	6.618134
2	0.1058	9.451796
1	0.0592	16.89189
Center E-W #E		
Floor	Displacement	Stiffness
5	0.2642	3.785011
4	0.1914	5.22466
3	0.1637	6.108735
2	0.1354	7.385524
1	0.0854	11.7096
Center E-W #F		
Floor	Displacement	Stiffness
1	0.0426	23.47418

Total Stiffness per Floor

Moment Frames	Floor 5	Floor 4	Floor 3	Floor 2	Floor 1	Total Stiffness
East N-S #7	7.199424046	8.849557522	11.14827202	15.03759	22.98851	65.22335332
East N-S #8	8.183306056	10.55966209	13.96648045	20.04008	32.05128	84.8008108
East N-S #9	6.752194463	9.00090009	11.9760479	16.33987	25.5102	69.57921582
East N-S #10	6.285355123	8.680555556	11.31221719	15.74803	24.39024	66.41640327
East N-S #11	6.422607579	8.960573477	11.69590643	16.33987	25.5102	68.92916085
East N-S #12	5.93824228	7.930214116	10.18329939	14.12429	21.05263	59.22868115
East E-W #A	11.09877913	16.55629139	21.59827214	30.39514	51.54639	131.1948712
East E-W #B	14.81481481	22.83105023	29.58579882	41.49378	69.44444	178.1698842
East E-W #D	18.24817518	30.03003003	39.37007874	54.64481	89.28571	231.578807
East E-W #F	8.34028357	12.61034048	16.61129568	23.20186	39.0625	99.82627588
West N-S #2	5.93824228	7.936507937	10.18329939	14.12429	21.05263	59.23497497
West N-S #3	6.422607579	8.960573477	11.69590643	16.33987	25.5102	68.92916085
West N-S #4	6.285355123	8.680555556	11.31221719	15.74803	24.39024	66.41640327
West N-S #5	6.711409396	9.033423668	11.77856302	16.44737	25.70694	69.67770537
West N-S #6	11.72332943	15.12859304	18.97533207	25.64103	38.02281	109.4910939
West E-W #AA	11.24859393	15.38461538	20.40816327	30.03003	51.81347	128.8848741
West E-W #BB	16.97792869	23.58490566	30.48780488	42.37288	70.92199	184.3455064
West E-W #DD	16.36661211	22.88329519	29.3255132	40.48583	67.56757	176.628818
West E-W #FF	10.03009027	13.24503311	1.689189189	23.36449	39.84064	88.169436
Center E-W #A	37.03703704	0	0	0	0	37.03703704
Center E-W #B	6.426735219	9.813542689	12.93661061	18.05054	30.30303	77.53046033
Center E-W #D	3.579098067	4.995004995	6.618133686	9.451796	16.89189	41.53592448
Center E-W #E	3.785011355	5.224660397	6.108735492	7.385524	11.7096	34.21353349
Center E-W #F	23.4741784	0	0	0	0	23.4741784
Total Stiffness Per Floor	259.2894111	280.8798861	348.9671372	506.807	824.5731	2220.51657

Direct Shear

Direct Shear	Stiffness	Relative Stiffness	Max Story Shear	Direct Shear
East N-S #7	65.22335	0.029373054	425.65	12.5026405
East N-S #8	84.80081	0.038189677	425.65	16.2554361
East N-S #9	69.57922	0.031334698	425.65	13.3376141
East N-S #10	66.4164	0.029910339	425.65	12.7313358
East N-S #11	68.92916	0.031041948	425.65	13.2130053
East N-S #12	59.22868	0.026673379	425.65	11.353524
East E-W #A	131.1949	0.059083041	425.65	25.1486963
East E-W #B	178.1699	0.080238034	425.65	34.1533192
East E-W #D	231.5788	0.104290511	425.65	44.3912559
East E-W #F	99.82628	0.04495633	425.65	19.1356619
West N-S #2	59.23497	0.026676214	425.65	11.3547304
West N-S #3	68.92916	0.031041948	425.65	13.2130053
West N-S #4	66.4164	0.029910339	425.65	12.7313358
West N-S #5	69.67771	0.031379052	425.65	13.3564936
West N-S #6	109.4911	0.049308839	425.65	20.9883073
West E-W #AA	128.8849	0.058042744	425.65	24.7058938
West E-W #BB	184.3455	0.083019199	425.65	35.3371219
West E-W #DD	176.6288	0.079544022	425.65	33.8579128
West E-W #FF	88.16944	0.039706723	425.65	16.9011666
Center E-W #A	37.03704	0.016679469	425.65	7.09961593
Center E-W #B	77.53046	0.034915506	425.65	14.8617853
Center E-W #D	41.53592	0.018705523	425.65	7.962006
Center E-W #E	34.21353	0.015407916	425.65	6.55837958
Center E-W #F	23.47418	0.010571494	425.65	4.49975658

Torsional Shear

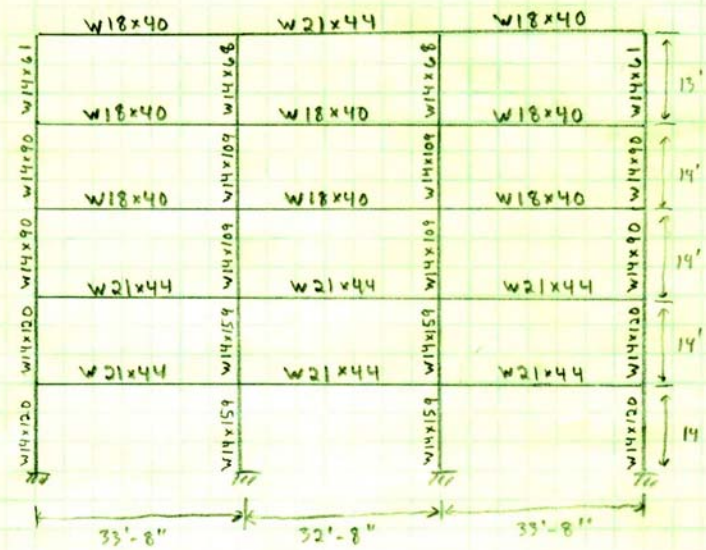
Torsion	k	x (ft)	kx^2	$kx/\Sigma kx^2$
East N-S #7	65.22335332	20.41	27169.97	9.36421E-05
East N-S #8	84.8008108	28.31	67964.13	0.000168875
East N-S #9	69.57921582	54.24	204700.5	0.000265475
East N-S #10	66.41640327	81.55	441695.8	0.000381
East N-S #11	68.92916085	109.22	822256.5	0.000529578
East N-S #12	59.22868115	142.65	1205246	0.000594332
East E-W #A	131.1948712	92.55	1123750	0.000854119
East E-W #B	178.1698842	73.11	952330.9	0.000916297
East E-W #D	231.578807	66.25	1016414	0.00107922
East E-W #F	99.82627588	87.67	767267.6	0.000615632
West N-S #2	59.23497497	142.65	1205374	0.000594395
West N-S #3	68.92916085	109.22	822256.5	0.000529578
West N-S #4	66.41640327	81.55	441695.8	0.000381
West N-S #5	69.67770537	54.24	204990.2	0.000265851
West N-S #6	109.4910939	28.31	87752.31	0.000218044
West E-W #AA	128.8848741	103.04	1368402	0.000934185
West E-W #BB	184.3455064	86	1363419	0.001115209
West E-W #DD	176.628818	80.24	1137217	0.00099696
West E-W #FF	88.169436	86.67	662301.4	0.000537542
Center E-W #A	37.03703704	63.41	148919.6	0.000165203
Center E-W #B	77.53046033	31.87	78747.45	0.000173812
Center E-W #D	41.53592448	14.21	8387.104	4.15186E-05
Center E-W #E	34.21353349	25.31	21917.06	6.09137E-05
Center E-W #F	23.4741784	39.02	35740.85	6.44322E-05

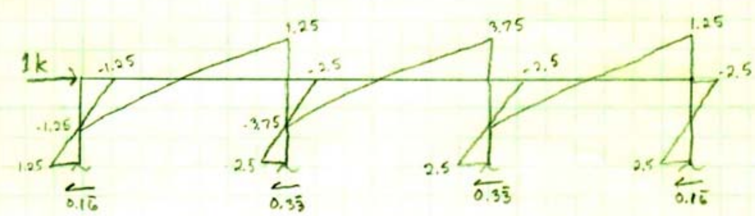
Torsional Shear

Torsion	Max Shear	Ecc. N-S	Torsion N-S	Tor. Shear N-S	Ecc. E-W	Torsion E-W	Tor. Shear E-W
East N-S #7	425.65	3.77	0.150267576	0.001502676	19.8	0.789203713	0.007892037
East N-S #8	425.65	24.23	1.741690928	0.017416909	14.64	1.052346479	0.010523465
East N-S #9	425.65	52.23	5.90197055	0.059019706	14.64	1.654314548	0.016543145
East N-S #10	425.65	80.23	13.01109693	0.130110969	14.64	2.374204899	0.023742049
East N-S #11	425.65	108.23	24.39667404	0.24396674	14.64	3.300076762	0.033000768
East N-S #12	425.65	141.9	35.89748604	0.35897486	14.64	3.703588412	0.037035884
East E-W #A	425.65	66.23	24.07830089	0.165296882	64.64	23.50024716	0.161328559
East E-W #B	425.65	66.23	25.83114388	0.177330101	30.97	12.07897517	0.082921837
East E-W #D	425.65	66.23	30.42406579	0.208860386	1.7	0.780928761	0.005361055
East E-W #F	425.65	80.23	21.02376386	0.178671708	80.36	21.05782954	0.178961217
West N-S #2	425.65	138.35	35.00313559	0.350031356	34.77	8.796957171	0.087969572
West N-S #3	425.65	106.71	24.05404312	0.240540431	23.24	5.238646444	0.052386464
West N-S #4	425.65	80.4	13.03866625	0.130386662	13.67	2.216897607	0.022168976
West N-S #5	425.65	54.09	6.120801621	0.061208016	4.09	0.462822678	0.004628227
West N-S #6	425.65	27.78	2.57827185	0.025782719	5.48	0.508600783	0.005086008
West E-W #AA	425.65	97.5	38.76950169	0.387695017	33.32	13.24922868	0.132492287
West E-W #BB	425.65	85.98	40.81372645	0.408137265	1.68	0.79747686	0.007974769
West E-W #DD	425.65	74.81	31.74606777	0.317460678	29.02	12.31480934	0.123148093
West E-W #FF	425.65	63.3	14.48332816	0.144833282	60.65	13.8769961	0.138769961
Center E-W #A	425.65	24.32	1.710154554	0.017101546	58.55	4.117168961	0.04117169
Center E-W #B	425.65	18.57	1.373865227	0.013738652	25.9	1.916160978	0.01916161
Center E-W #D	425.65	12.98	0.229387872	0.002293879	5.78	0.102146526	0.001021465
Center E-W #E	425.65	3.78	0.098007582	0.000980076	25.03	0.648976134	0.006489761
Center E-W #F	425.65	6.78	0.185945281	0.001859453	38.43	1.053964182	0.010539642

Portal Method

MOMENT FRAME #10





To find the deflection @ floor #5 use:

$$U_K = \frac{M_K L_K^2}{6EI_K} + \frac{L_K L_B}{12E}\left(\frac{M_{BK-1}}{I_{BK-1}} + \frac{M_{BK}}{I_{BK}}\right)$$

where $I_{BEAM} = 843 in^4$ & $I_{COLUMN} = 722 in^4$
W21×44 W14×68

$$U_K = \frac{(292.5"k)(156")^2}{6(29000 ksi)(722 in^4)} + \frac{(156")(312")}{12(29000 ksi)}\left(\frac{(292.5"k)}{(843 in^4)} + \frac{(292.5"k)}{(843 in^4)}\right)$$

$U_K = 0.0567 + 0.1219 = 0.1786 in$, SAP2000 → 0.1591 in

Difference = 0.0195 in, This value is negligable so the computer model is valid