Daniel Hancock
Structural Option
Dr. Hanagan

S&T Bank
Corporate Headquarters
Indiana, PA

Executive Summary

Construction of S&T Bank Headquarters began in June 2005 and is projected to be completed by August 2006. The building is 4 stories above ground rising to almost 60 feet with a one-story basement underground. Primarily the building is a corporate office for S&T Bank employees, however there is also a bank branch on the first floor.

The following report is meant to describe the building's lateral resisting system and the effects lateral loads have on different aspects of the design. The existing lateral resisting system is a moment connection frame. Frame stiffness was used to distribute lateral loads according to highest stiffness. Once the lateral loads were determined, they were used to perform checks on torsional effects, over-turning moments, building and story drift, as well as member strengths.

All of the checks passed design criteria. One discrepancy occurred in checking the strength of a column. The column that exists is strong enough, however it could be downsized by one size. It may be that live loads may not have been reduced (as was done in this analysis) or since a W12x79 is a common shape found in the building and therefore more economical to order that shape.

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Introduction

S&T Bank Corporate Headquarters is located in Indiana, PA. Construction of the project began in June 2005 and project completion is projected for August 2006. Primarily the building is a corporate office for S&T Bank employees. On the first floor, a bank branch is available for customers. The rest of the floors except the fourth floor comprise of mostly offices, however there are large lobby areas designated for different facilities of the bank (i.e. finance dept., loan dept., etc.). The 4th floor is reserved for future plan layouts, which are dependent on the growth of the company.

The building is 4 stories above ground rising to almost 60 feet with a one-story basement underground. The floor system is non-composite decking is set on 24k4 joists that are spaced at 2′-0″ apart. On top of the decking is a 3″ normal weight concrete topping rated at 3000psi. The slab on grade foundation is a 4 inch concrete slab rated at 3000psi which is supported by spread footings underneath. The footings for the structure can support 6000psf.

Structural steel makes up the framing of the building. The steel girders and columns are made of A992 steel, which has yield strength of 50ski. Framing from floor to floor is grid-like and relatively consistent.

Every beam and girder frames into columns. The connections are either shear connections of moment connections. The moment connections that resist the lateral loads are the focus for this report. Girders running E-W are W16x26 up to W24x76 with a typical girder of W24x55. Beams running N-S are much smaller, W12x16 up to W16x26, with a typical beam of W14x22. Running spans for the girders and beams are typically 28 to 30 feet. The building's columns range from W10x33 to W12x87, while a typical column size used is W12x53.

The sections in the following report are broken down as follows…

- Existing Lateral System
- Load and Load Combinations
- Analysis of Load Effects– Torsion, Over-turning, Drift, and Strength
- Summary/Conclusion

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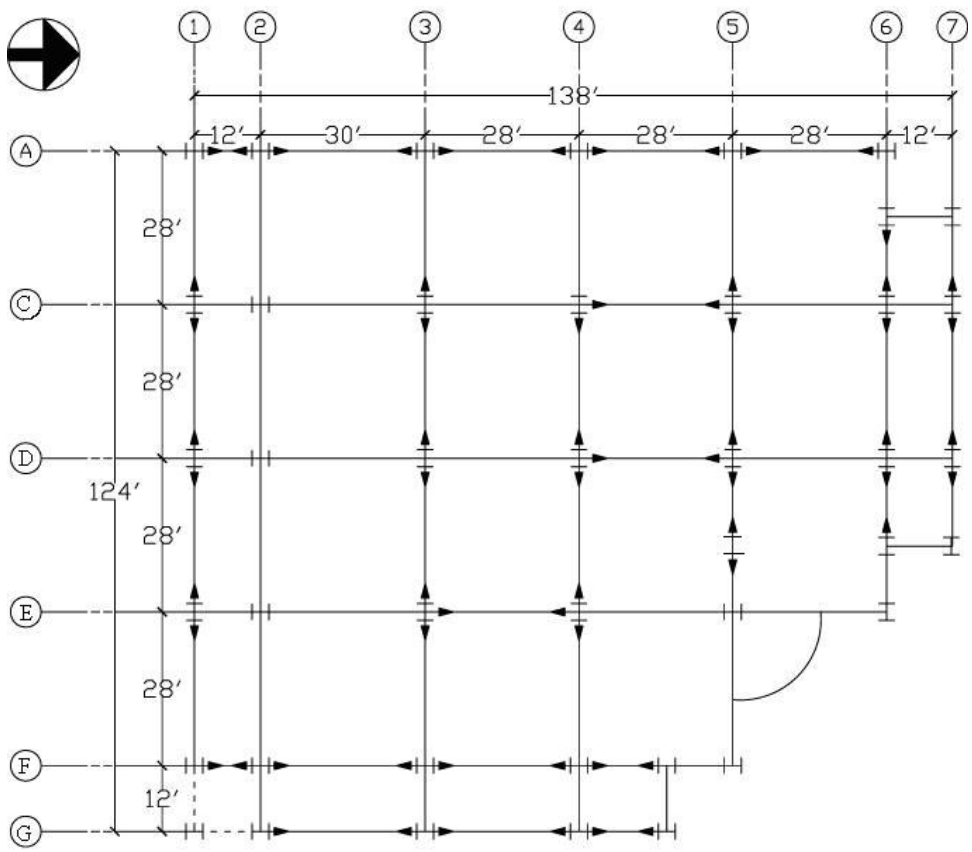
Existing Lateral System

S&T Bank Corporate Headquarters incorporates a moment frame to resist lateral loads. The moment connections are attached via wind clips which act as a partially restrained connection. To simplify this report these wind clips will be assumed to be full moment connections. Below is a representation of the layout of the moment connections. The arrows on Figure#1 indicate which direction the moment connections resist lateral loads.



Figure#1: Moment connection Layout.

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RAM Steel was used to set up a model of S&T Bank. A print out from RAM gave the deflections (as seen in Appendix A-1) of each frame when a 1kip load was applied. From these deflections, frame stiffness was derived by using the equation; 1/deflection= Stiffness. The method of frame stiffness is a logical method of determining how lateral loads would be distributed. Due to the amount of moment connections, distribution by stiffness showed an almost uniform distribution. Calculations of distribution of forces can be seen in Appendix A-2. The results from this work are as follows...

FRAME	% LOAD CARRIED
A	17.46
C	16.8
D	16.8
E	16.5
F	16.16
G	16.03
1	13.04
2	13.23
3	13.73
4	14.24
5	14.77
6	15.36
7	15.63

Once the controlling loads were determined (wind/seismic) they were distributed to each frame depending on the frames stiffness; the higher the frame stiffness, the greater load the frame will see.

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Loads and Load Combinations

Wind and seismic loads were determined using ABC 2003 in accordance with ASCE 7-02. The derivation of the wind and seismic loads can be referenced in Appendix B-1. The summary of those calculations verified that wind loads were greater on the roof and near the base of the building while seismic loads controlled on the 2nd, 3rd, and 4th floors. These particular loads are as follows…

F_{roof}= 14.66 kips
F_{4th} = 43.98 kips
F_{3rd}= 49.42 kips
F_{2nd}= 53.1 kips
F_{1st}= 16.9 kips

From IBC 2003 different load combinations were analyzed to check which would control design. The load combinations looked at are…

1.4D
1.2D+1.6L+ (0.5L or 0.8W)
1.2D+1.6W+0.5L+0.5S
1.2D+1.0E+0.5L+0.5S
0.9D+ (1.6W or 1.0E)

The controlling case is

1.2D+ 1.6W+ 0.5L+ 0.5S

Though it is not readily apparent, after some minor calculations it was determined that 1.6W>1.0E, hence the controlling case is chosen. For

analysis that did not require any factors (i.e. torsion analysis & overturning moment) the largest load was used, as explained above.

Analysis of Load Effects

Torsion effects are due to an eccentricity of applied loads to the building's center of rigidity. A building that is completely symmetric will have no torsion loads induced on the framing members. S&T Bank is not quite symmetric and therefore has some torsion effects. Appendix C-1 shows the exact calculations of the torsion loads as well as the location of the building's center of rigidity. The 4th floor was randomly chosen to analyze the torsion forces. In the end it turned out that the torsional force, F_t= 0.1164 kips. Compared to the applied story force of 43.98 kips, the torsional force is less than .5% of said load. Since the loads are so insignificant, torsional loads can be neglected. Even though the 2nd story has a higher external load they are again so small that they can be neglected.

Overturning of a building is a matter that must be addressed, especially if the building is relatively light. Typically the building's weight is enough to resist the overturning moment induced by the applied external loads. If the building is not heavy enough or is to "skinny" to resist the

moment, additional loads would be seen in the foundation and would have to be designed accordingly. S&T Bank had no problem resisting the overturning moment. The moment due to the external forces is M= 4,888.5ft-kips. The moment due to weight was able to resist about 400,000ft-kips. Since 4,888.5 << 400,000 the building will not be overturned. Specific calculations can be referenced in Appendix C-2.

Another effect of external loading that must be addressed is building and story drift. To study drift, loads on framing members need to be determined first. The real work method is used (as seen in Appendix C-3) to find story drift by calculating the horizontal deflection of columns on a given floor. In this particular case, the third story was analyzed. By combining internal and external work, a story drift of 0.0003 inches was found; this is an exceptionally small drift. It is common practice to limit story drifts to h/240, which is .665 inches for the third story. Since .0003<<.667, the drift is well within the limit. Building drift is usually limited to h/400, which is 1.8 inches for S&T Bank. Acknowledging that the story drifts are very small and well within limit, it is safe to say that the building drift would also be within limits and therefore will not be directly checked.

The final effect that must be checked is a member strength check. A column on an interior bay will be checked to determine if it is adequate to

hold a combined lateral and gravity load. Since design of structural members require safety factors, the governing load combination 1.2D+ 1.6W+ 0.5L+ 0.5S will be used during the check of the column. Live load reduction affects column loads significantly; therefore live loads will be reduced. Spot-checking calculations can be referenced in Appendix C-4. The particular column being checked needs to be able to carry P_{eff}=689.68 kips. Table 4 of the AISC manual provides that a W12x72 can carry a load of 717kips. Since 717k > 689.68k a W12x72 will work. The actual design member used is a W12x79 which is only one size bigger than the W12x79 and can carry a factored load of 790kips. The discrepancy between sizes may be due to a variation in load combinations or more likely live load differences (reduced vs. unreduced). The difference in design may also be because a W12x79 is used throughout the rest of the building and it is more efficient to order one more of the same size than it is to get one column that is W12x72.

Conclusion

This report has shown that even though seismic lateral forces are larger than those caused by wind, wind loads control design when safety factors are applied. Therefore, the controlling load combination was found

to be 1.2D + 1.6W + 0.5L + 0.5S. A method of frame stiffness was used to direct distribution of lateral forces. This is reasonable since a stiffer member will take more load than a non-stiff member. All checks were found acceptable in terms of drift, torsion, and overturning. The only exception was found when completing a spot check of a column. Finding that a smaller column than is used was acceptable could be due to lateral force distribution or possibly live load reductions.

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Appendix A-1: Deflections (RAM Output)

Nodal Displacements

RAM Frame v8.1 N-S Frame Page 2/
DataBase: S&TBank E-W Force 11/20/05 17:08:2

Node	LdC	Disp X	Disp Y	Disp Z	Theta X	Theta Y	Theta Z
32	N1	0.00284	-0.09852	-0.00527	-0.00000	0.00000	0.00001
33	N1	0.00120	-0.09852	-0.00107	0.00007	0.00000	0.00001
34	N1	-0.00098	-0.09852	0.00013	0.00003	-0.00000	0.00001
35	N1	-0.00479	-0.09852	0.00058	0.00003	-0.00000	0.00001
36	N1	-0.00860	-0.09852	0.00028	0.00016	-0.00000	0.00001
37	N1	0.00284	-0.09634	0.00000	0.00015	0.00000	0.00001
38	N1	0.00284	-0.09471	0.00000	0.00015	0.00000	0.00001
39	N1	0.00120	-0.09471	-0.00087	0.00005	0.00000	0.00001
40	N1	-0.00098	-0.09471	0.00001	0.00003	-0.00000	0.00001
41	N1	-0.00479	-0.09471	0.00006	0.00003	-0.00001	0.00001
42	N1	-0.00697	-0.09471	0.00079	0.00007	-0.00001	0.00001
43	N1	-0.00860	-0.09471	0.00006	0.00015	-0.00000	0.00001
44	N1	0.00120	-0.09307	-0.00016	0.00015	0.00000	0.00001
45	N1	-0.00098	-0.09307	-0.00049	0.00003	-0.00000	0.00001
46	N1	-0.00479	-0.09307	0.00051	0.00003	-0.00001	0.00001
47	N1	-0.00697	-0.09307	0.00016	0.00015	-0.00001	0.00001

Level: Fourth Floor

	Node	LdC	Disp X in	Disp Y in	Disp Z in	Theta X (rad)	Theta Y (rad)	Theta Z (rad)
	48	N1	0.00484	-0.08216	-0.00014	0.00020	0.00001	0.00001
	49	N1	0.00208	-0.08216	-0.00061	0.00010	0.00000	0.00001
	50	N1	-0.00069	-0.08216	0.00022	0.00008	-0.00000	0.00001
	51	N1	-0.00345	-0.08216	0.00036	0.00013	-0.00001	0.00001
1	52	N1	-0.00621	-0.08216	0.00011	0.00020	-0.00001	0.00001
	53	N1	0.00602	-0.08098	0.00005	0.00019	0.00001	0.00001
	54	N1	0.00484	-0.08098	-0.00001	0.00019	0.00001	0.00001
	55	N1	0.00208	-0.08098	0.00000	0.00019	0.00000	0.00001
	56	N1	-0.00069	-0.08098	0.00000	0.00019	-0.00000	0.00001
	57	N1	-0.00345	-0.08098	0.00000	0.00019	-0.00001	0.00001
2	58	N1	-0.00621	-0.08098	0.00005	0.00019	-0.00001	0.00001
	59	N1	0.00602	-0.07802	0.00000	0.00019	0.00001	0.00001
	60	N1	0.00484	-0.07802	-0.00030	0.00019	0.00001	0.00001
	61	N1	0.00208	-0.07802	-0.00049	0.00011	0.00000	0.00001
	62	N1	-0.00069	-0.07802	0.00006	0.00009	-0.00000	0.00001
	63	N1	-0.00345	-0.07802	0.00049	0.00011	-0.00001	0.00001
3	64	N1	-0.00621	-0.07802	0.00029	0.00019	-0.00001	0.00001
	65	N1	0.00602	-0.07526	0.00001	0.00018	0.00001	0.00001
	66	N1	0.00484	-0.07526	-0.00025	0.00018	0.00001	0.00001
	67	N1	0.00208	-0.07526	-0.00048	0.00011	0.00000	0.00001
	68	N1	-0.00069	-0.07526	-0.00005	0.00008	-0.00000	0.00001
	69	N1	-0.00345	-0.07526	0.00078	0.00013	-0.00001	0.00001
4	70	N1	-0.00621	-0.07526	0.00001	0.00018	-0.00001	0.00001
	71	N1	0.00602	-0.07368	-0.00006	0.00017	0.00001	0.00001
	72	N1	0.00484	-0.07368	-0.00005	0.00017	0.00001	0.00001

Nodal Displacements

RAM Frame v8.1
DataBase: S&TBank

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Node	LdC	Disp X	Disp Y	Disp Z	Theta X	Theta Y	Theta Z
73	N1	0.00365	-0.07368	-0.00003	0.00000	0.00001	0.00001
74	N1	0.00208	-0.07368	0.00000	0.00017	0.00000	0.00001
75	N1	0.00484	-0.07250	0.00000	0.00017	0.00001	0.00001
76	N1	0.00365	-0.07250	-0.00017	0.00017	0.00001	0.00001
77	N1	0.00208	-0.07250	-0.00588	-0.00000	0.00000	0.00001
78	N1	0.00089	-0.07250	-0.00099	0.00008	0.00000	0.00001
79	N1	-0.00069	-0.07250	0.00012	0.00009	-0.00000	0.00001
80	N1	-0.00345	-0.07250	0.00053	0.00010	-0.00001	0.00001
5 81	N1	-0.00621	-0.07250	0.00026	0.00017	-0.00001	0.00001
82	N1	0.00208	-0.07092	0.00000	0.00017	0.00000	0.00001
83	N1	0.00208	-0.06973	0.00000	0.00016	0.00000	0.00001
84	N1	0.00089	-0.06973	-0.00079	0.00013	0.00000	0.00001
85	N1	-0.00069	-0.06973	-0.00000	0.00009	-0.00000	0.00001
86	N1	-0.00345	-0.06973	0.00006	0.00009	-0.00001	0.00001
87	N1	-0.00503	-0.06973	0.00072	0.00013	-0.00001	0.00001
6 88	N1	-0.00621	-0.06973	0.00006	0.00016	-0.00001	0.00001
89	N1	0.00089	-0.06855	-0.00015	0.00016	0.00000	0.00001
90	N1	-0.00069	-0.06855	-0.00045	0.00011	-0.00000	0.00001
91	N1	-0.00345	-0.06855	0.00047	0.00011	-0.00001	0.00001
7 92	N1	-0.00503	-0.06855	0.00015	0.00016	-0.00001	0.00001

Level: Third Floor

Node	LdC	Disp X in	Disp Y in	Disp Z in	Theta X (rad)	Theta Y (rad)	Theta Z (rad)
93	N1	0.00299	-0.05043	-0.00011	0.00019	0.00001	0.00001
94	N1	0.00129	-0.05043	-0.00048	0.00006	0.00000	0.00001
95	N1	-0.00041	-0.05043	0.00017	0.00005	-0.00000	0.00001
96	N1	-0.00210	-0.05043	0.00029	0.00008	-0.00001	0.00001
97	N1	-0.00380	-0.05043	0.00009	0.00019	-0.00001	0.00001
98	N1	0.00372	-0.04970	0.00004	0.00019	0.00001	0.00001
99	N1	0.00299	-0.04970	-0.00001	0.00019	0.00001	0.00001
100	N1	0.00129	-0.04970	0.00000	0.00019	0.00000	0.00001
101	N1	-0.00041	-0.04970	0.00000	0.00019	-0.00000	0.00001
102	N1	-0.00210	-0.04970	0.00000	0.00019	-0.00001	0.00001
103	N1	-0.00380	-0.04970	0.00004	0.00019	-0.00001	0.00001
104	N1	0.00372	-0.04788	0.00000	0.00018	0.00001	0.00001
105	N1	0.00299	-0.04788	-0.00023	0.00018	0.00001	0.00001
106	N1	0.00129	-0.04788	-0.00039	0.00006	0.00000	0.00001
107	N1	0.00093	-0.04788	0.00190	0.00001	0.00000	0.00001
108	N1	-0.00041	-0.04788	0.00006	0.00004	-0.00000	0.00001
109	N1	-0.00210	-0.04788	0.00038	0.00007	-0.00001	0.00001
110	N1	-0.00380	-0.04788	0.00023	0.00018	-0.00001	0.00001
111	N1	0.00372	-0.04618	0.00000	0.00017	0.00001	0.00001
112	N1	0.00299	-0.04618	-0.00020	0.00017	0.00001	0.00001
113	N1	0.00129	-0.04618	-0.00038	0.00006	0.00000	0.00001

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Nodal Displacements

RAM Frame v8.1
DataBase: S&TBank

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Node	LdC	Disp X	Disp Y	Disp Z	Theta X	Theta Y	Theta Z
32	N1	0.17460	0.00270	0.00018	0.00000	0.00029	0.00001
33	N1	0.17326	0.00270	0.00001	-0.00000	0.00029	0.00001
34	N1	0.17148	0.00270	-0.00035	-0.00000	0.00014	0.00001
35	N1	0.16836	0.00270	-0.00039	-0.00000	0.00010	0.00001
36	N1	0.16523	0.00270	0.00011	-0.00000	0.00003	0.00001
37	N1	0.17460	0.00448	0.00000	-0.00001	0.00029	0.00001
38	N1	0.17460	0.00582	0.00000	-0.00001	0.00029	0.00001
39	N1	0.17326	0.00582	0.00005	-0.00000	0.00029	0.00001
40	N1	0.17148	0.00582	-0.00000	-0.00000	0.00028	0.00001
41	N1	0.16836	0.00582	-0.00000	-0.00000	0.00028	0.00001
42	N1	0.16657	0.00582	-0.00005	-0.00000	0.00028	0.00001
43	N1	0.16523	0.00582	-0.00115	-0.00001	0.00006	0.00001
44	N1	0.17326	0.00716	0.00001	-0.00001	0.00029	0.00001
45	N1	0.17148	0.00716	0.00004	-0.00000	0.00028	0.00001
46	N1	0.16836	0.00716	-0.00004	-0.00000	0.00028	0.00001
47	N1	0.16657	0.00716	-0.00001	-0.00001	0.00028	0.00001

Level: Fourth Floor

	Node	LdC	Disp X in	Disp Y in	Disp Z in	Theta X (rad)	Theta Y (rad)	Theta Z (rad)
F	48	N1	0.12918	-0.00633	0.00108	0.00001	0.00022	0.00001
E	49	N1	0.12678	-0.00633	-0.00003	0.00001	0.00032	0.00001
D	50	N1	0.12438	-0.00633	0.00000	0.00001	0.00031	0.00001
C	51	N1	0.12197	-0.00633	0.00004	0.00001	0.00031	0.00001
A	52	N1	0.11957	-0.00633	0.00177	0.00001	0.00025	0.00001
G	53	N1	0.13021	-0.00530	0.00102	0.00001	0.00026	0.00001
	54	N1	0.12918	-0.00530	-0.00016	0.00001	0.00025	0.00001
	55	N1	0.12678	-0.00530	0.00000	0.00001	0.00032	0.00001
	56	N1	0.12438	-0.00530	0.00000	0.00001	0.00031	0.00001
	57	N1	0.12197	-0.00530	-0.00000	0.00001	0.00031	0.00001
	58	N1	0.11957	-0.00530	-0.00066	0.00001	0.00021	0.00001
	59	N1	0.13021	-0.00273	0.00004	0.00001	0.00021	0.00001
	60	N1	0.12918	-0.00273	-0.00006	0.00001	0.00027	0.00001
	61	N1	0.12678	-0.00273	0.00027	0.00000	0.00013	0.00001
	62	N1	0.12438	-0.00273	0.00001	0.00000	0.00031	0.00001
	63	N1	0.12197	-0.00273	0.00001	0.00000	0.00031	0.00001
	64	N1	0.11957	-0.00273	0.00002	0.00001	0.00020	0.00001
	65	N1	0.13021	-0.00032	0.00014	0.00000	0.00023	0.00001
	66	N1	0.12918	-0.00032	0.00055	0.00000	0.00027	0.00001
	67	N1	0.12678	-0.00032	-0.00027	0.00000	0.00013	0.00001
	68	N1	0.12438	-0.00032	0.00029	0.00000	0.00022	0.00001
	69	N1	0.12197	-0.00032	0.00036	0.00000	0.00022	0.00001
	70	N1	0.11957	-0.00032	-0.00000	0.00000	0.00019	0.00001
	71	N1	0.13021	0.00105	-0.00125	-0.00000	0.00029	0.00001
	72	N1	0.12918	0.00105	-0.00124	-0.00000	0.00029	0.00001

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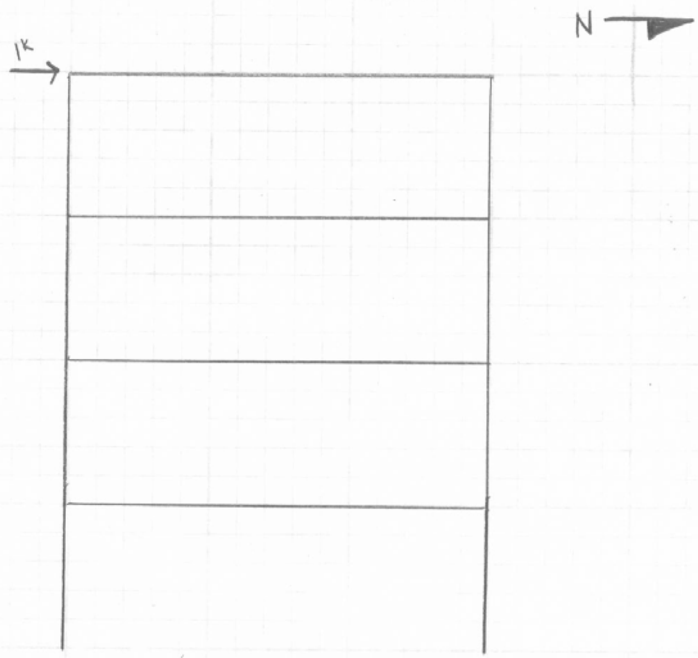
Appendix A-2: Load Distribution

Nodal Displacements — Tech # 3

E-W Frame

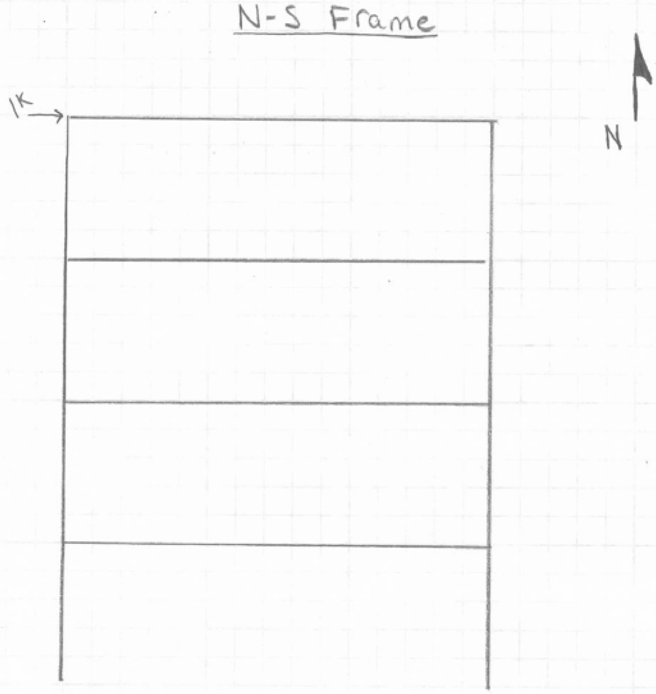


	deflection	Stiffness	% load carried
Frame A :	0.11957 inches	8.363	17.46%
Frame C :	0.12197 inches	8.199	17.1%
Frame D :	0.12438 inches	8.040	16.8%
Frame E :	0.12678 inches	7.888	16.5%
Frame F :	0.12918 inches	7.741	16.16%
Frame G :	0.13021 inches	7.680	16.03%

Loads from N-S direction

Nodal Displacements   Tech #3

N-S Frame



	deflection	Stiffness	% Load Carried
Frame 1:	.08216	12.17	13.04 %
Frame 2:	.08098	12.35	13.23 %
Frame 3:	.07802	12.82	13.73 %
Frame 4:	.07526	13.29	14.24 %
Frame 5:	.07250	13.79	14.77 %
Frame 6:	.06973	14.34	15.36 %
Frame 7:	.06855	14.59	15.63 %

Loads from E-W Direction

Appendix B-1: Wind/Seismic Loads

Wind Load Analysis | S&T Bank

$h \leq 60'$ → use simplified wind load method

$V = 90$ mph (40 m/s)
$I_w = 1.0$
Exposure Category B
$\lambda = 1.22$
$P_{s30} = -6.7$ psf

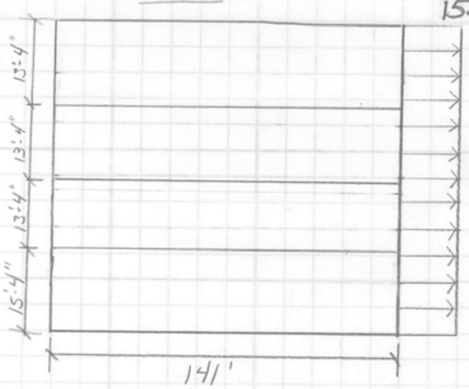
$P_s = \lambda I_w P_{s30}$

$= 1.22(1.0)(-6.7)$

$P_s = -8.174$ psf

Must check wind Loads @ each surface (IBC2003 - 1609.6.2.1)

N-S

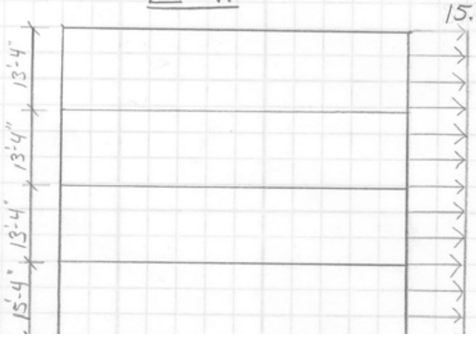


MWFRS

Zone	Adjusted Pressure P_s
A	15.6 psf
B	-8.2 psf
C	10.37 psf
D	-4.5 psf
E	-18.8 psf
F	10.7 psf
G	-13.1 psf
H	-8.3 psf

Zone A is the significant wall loading

E-W



Cladding

Zone	Adjusted Pressure
Wall	17.8 psf -19.3 psf
Corner	17.8 psf -23.4 psf

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Wind Load Analysis

Floor	Tributary Area
1st	(141)(15.33/2) = 1080.76 SF
2nd	(141)((13.33+15.33)/2) = 2020.53 SF
3rd	(141)(13.33) = 1879.53 SF
4th	(141)(13.33) = 1879.53 SF
Roof	(141)(13.33)(.5) = 939.76 SF

Floor	Wind Load
1st	1080.76(15.6)/1000 = 16.9 Kips
2nd	2020.53(15.6)/1000 = 31.5 Kips
3rd	1879.53(15.6)/1000 = 29.3 Kips
4th	1879.53(15.6)/1000 = 29.3 Kips
Roof	939.76(15.6)/1000 = 14.66 Kips

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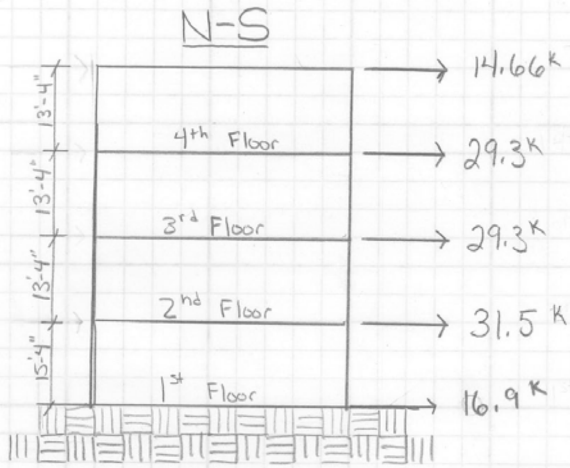
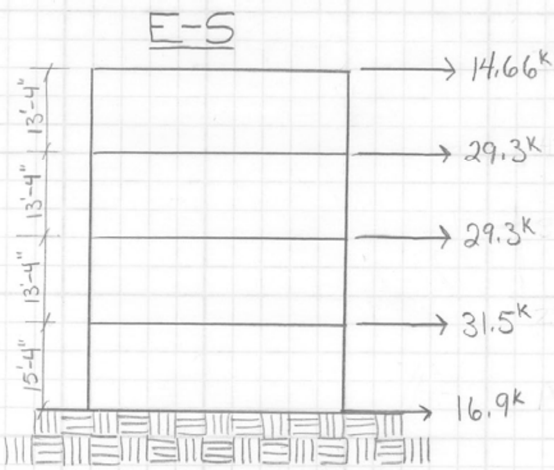
Wind Load Analysis | S&T Bank





* Simplified Analysis method Assumes Building is approximately Symetric.

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Seismic Loads

Seismic use group = 1
Importance = 1.0
$S_S = 12.7$
$S_1 = 5.4$
Site Class C

$F_a = 1.2$ $F_v = 1.7$

$S_{ms} = F_a S_s$
$= 1.2(12.7) = 15.24\%$

$S_{m1} = F_v S_1$
$= 1.7(5.4) = 9.18\%$

$S_{ds} = \frac{2}{3} S_{ms} = 10.16\%$

$S_{D1} = \frac{2}{3} S_{m1} = 6.12\%$

$S_a = 10.16$

$T_a = .1$ (# stories) $= .1(4) = .4$ (approximate)

$T_o = .2 \frac{(S_{D1})}{(S_{DS})} = 0.121$

$T_s = \frac{S_{D1}}{S_{DS}} = \frac{6.21}{10.16} = .611$

$T_o < T_a < T_s$ ∴ $S_a = S_{DS}$

Design Category A: $S_{DS} < .167g$

Ordinary Steel moment frames

$R^a = 4$ $C_d^b = 3.5$
$\Omega_o^g = 3$

$C_s = \frac{S_{DS}}{R/I} = \frac{.1016}{4/1.0} = 0.0254$

$V = C_s W$
$6296.81 \times 0.0254 = 159.94^k$

Base Shear = 124.39^k

$T_a = 0.1N = .1 \times 4$ stories
$T_a = .4 \rightarrow k = 1$

Weight @ floors w/ cladding | Seismic Loads

Roof: $696.59^k + [2(3' \times 141 \times 15psf) + 2(3' \times 127' \times 15psf)] = \underline{720.71^k}$

4th: $720.71 + 601.28 + 1007.57 + [2(13.33' \times 141' \times 15) + 2(13.33' \times 127' \times 15)]$
individual floor weight = 1716.02 $= \underline{2436.73^k}$

3rd: $2436.73 + 636.68 + 1007.57 + [2(13.33' \times 141 \times 40) + 2(13.33' \times 127' \times 40)]$
Individual floor weight = 1930.04 $= \underline{4366.77}$

2nd: $4366.77 + 636.68 + 1007.57 + [2(13.33' \times 141 \times 40) + 2(13.33' \times 127' \times 40)]$
Individual floor weight = 1930.04 $= \underline{6296.81^k}$

1st: $6296.81 + 659.84 + 1007.57 + [2(15.33 \times 141 \times 40) + 2(15.33 \times 127 \times 40)]$
Individual floor weight = 1996.08 $= \underline{8292.89^k}$

$F_x = C_{vx} V$ $\quad k = 1$

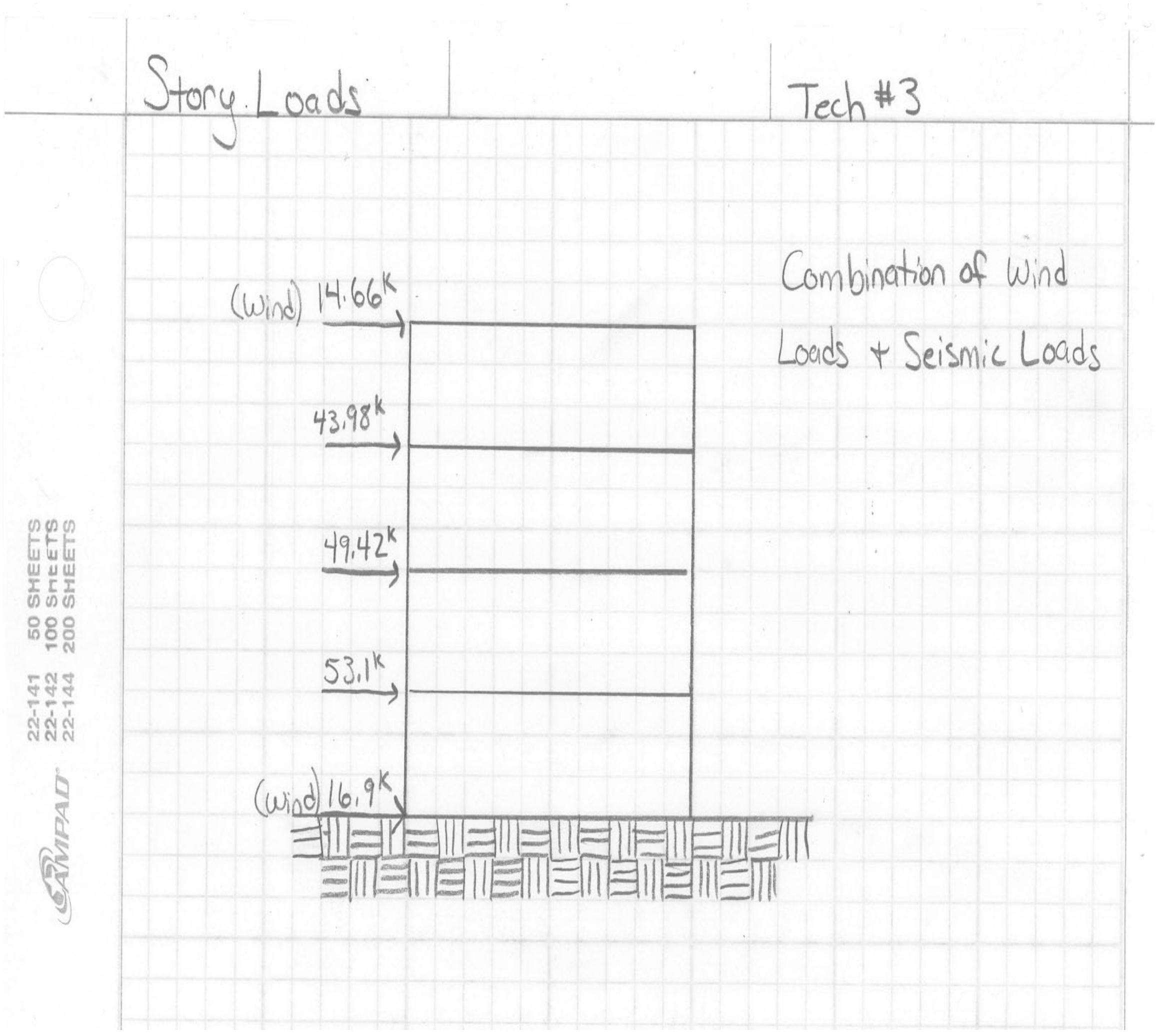
$C_{vx} = \frac{w_x h_x^k}{\sum w_i h_i^k}$

$w_r h_r^k = (720.71)(9.67) = 6969.27$
$w_4 h_4^k = (1716.02)(13.33') = 22874.55$
$w_3 h_3^k = (1930.04)(13.33') = 25727.43$
$w_2 h_2^k = (1930.04)(14.33') = 27657.47$

$\sum w_i h_i^k = 83228.72$

$C_{vr} = .0837$
$C_{v4} = 0.275$
$C_{v3} = 0.309$
$C_{v2} = 0.332$
$C_{v1} = 0$

$F_r = 0.0837 \times 159.94^k = 13.39^k$
$F_4 = 0.275 \times 159.94^k = 43.98^k$
$F_3 = 0.309 \times 159.94^k = 49.42^k$
$F_2 = 0.332 \times 159.94^k = 53.1^k$
$F_1 = 0$


Story Loads
Tech #3
Combination of Wind Loads + Seismic Loads
(wind) 14.66k
43.98k
49.42k
53.1k
(wind) 16.9k

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Appendix C-1: Torsion

Center of Mass — Tech #3

$$\bar{y} = \frac{\sum A y}{\sum A} \qquad \bar{x} = \frac{\sum A x}{\sum A}$$

1: 98' × 84 — $A = 8232$, $\hat{y} = 42$

2: 98' × 28' — $A = 2744$, $\hat{y} = 84 + \frac{28}{2} = 98$

3: 74 × 12 — $A = 888$, $\hat{y} = 118$

4: 28' × 84 — $A = 2352$, $\hat{y} = 42'$

5: 12' × 60 — $A = 720$, $\hat{y} = 42'$

1: $\hat{x} = 49$
2: $\hat{x} = 49$
3: $\hat{x} = 49$
4: $\hat{x} = 112$
5: $\hat{x} = 132$

$$\bar{y} = \frac{(8232 \times 42) + (2744 \times 98) + (888 \times 118) + (2352 \times 42) + (720 \times 42)}{(8232 + 2744 + 888 + 2352 + 720)}$$

$\bar{y} = 56.81'$

$$\bar{x} = \frac{(8232 \times 49) + (2744 \times 49) + (888 \times 49) + (2352 \times 112) + (720 \times 132)}{(8232 + 2744 + 888 + 2352 + 720)}$$

$\bar{x} = 62.92'$

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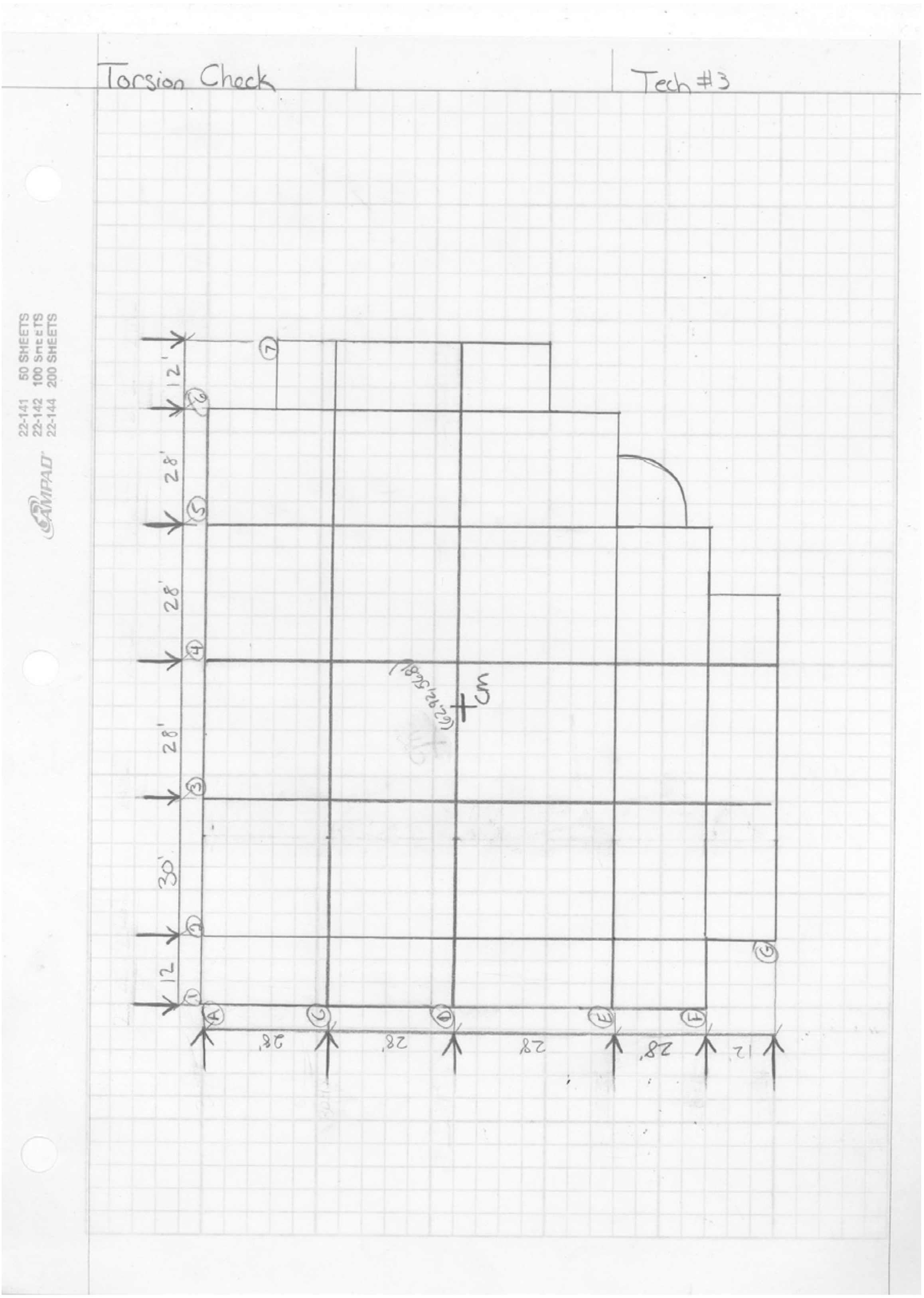

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Torsion — Tech #3

$M = Pe$ $\quad F_{i\,Torsion} = M(k_i d_i)/(\Sigma k_i d_i^2)$

Loads

roof : 14.66ᵏ (wind)
4th : 43.98ᵏ (seismic)
3rd : 49.42ᵏ (seismic)
2nd : 53.1ᵏ (seismic)
1st : 16.9ᵏ (wind)

Roof:

Frame: (E-W direction)

A = 0.1746(14.66) = 2.56ᵏ
C = 0.1710(14.66) = 2.51ᵏ
D = 0.1680(14.66) = 2.46ᵏ
E = 0.1650(14.66) = 2.42ᵏ
F = 0.1616(14.66) = 2.37ᵏ
G = 0.1603(14.66) = 2.35ᵏ

↻ $M_y = -(1.91 \times 62.92') - (1.94 \times 50.92) - (2.01 \times 20.92) + (2.08 \times 7.08) + (2.17 \times 35.08) + (2.25 \times 63.08) + (2.29 \times 75.08)$
$= 143.702^{'k}$

Frame: (N-S direction)

1 = 0.1304(14.66) = 1.91ᵏ
2 = 0.1323(14.66) = 1.94ᵏ
3 = 0.1373(14.66) = 2.01ᵏ
4 = 0.1424(14.66) = 2.08ᵏ
5 = 0.1477(14.66) = 2.17ᵏ
6 = 0.1536(14.66) = 2.25ᵏ
7 = 0.1563(14.66) = 2.29ᵏ

$M_x = (2.56 \times 56.81) + (2.51 \times 28.81) + (2.46 \times .81) - (2.42 \times 27.19) - (2.37 \times 55.19) - (2.35 \times 67.19)$
$= -134.75$

$M = +8.95^{'k}$

4th Floor:

A = 7.68ᵏ
C = 7.52ᵏ
D = 7.39ᵏ
E = 7.26ᵏ
F = 7.11ᵏ
G = 7.05ᵏ

$M_y = +431.32^{'k}$

$M_x = -404.55^{'k}$

$M = 26.77^{'k}$

1 = 5.73ᵏ
2 = 5.82ᵏ
3 = 6.04ᵏ
4 = 6.26ᵏ
5 = 6.50ᵏ
6 = 6.76ᵏ
7 = 6.87ᵏ

$\dfrac{k_i d_i}{\Sigma k_i d_i^2} = .00435 \rightarrow$ Frame A

$F_{i\,torsional} = 26.77^{'k}(.00435) = .1164^{k}$

* When compared to the story shear, the torsional force is less than .5% and will not have an impact on structural members. Due to this Torsional effects may be neglected.

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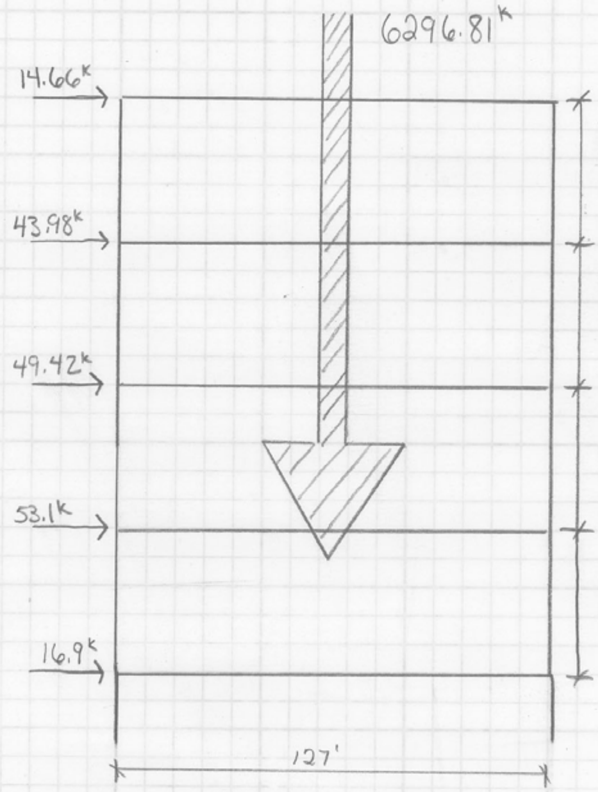
Appendix C-2: Over-turning Moment

Over-Turning — Tech # 3

Building Weight = 6296.81 K



$$\Sigma M = (14.66 \times 55.32') + (43.98 \times 42) + (49.42 \times 28.66) + (53.1 \times 15.33) - (6296.81 \times 63.5)$$

$$= -394{,}958.8^{K}$$

* Building weight is more than enough to resist over turning moment.

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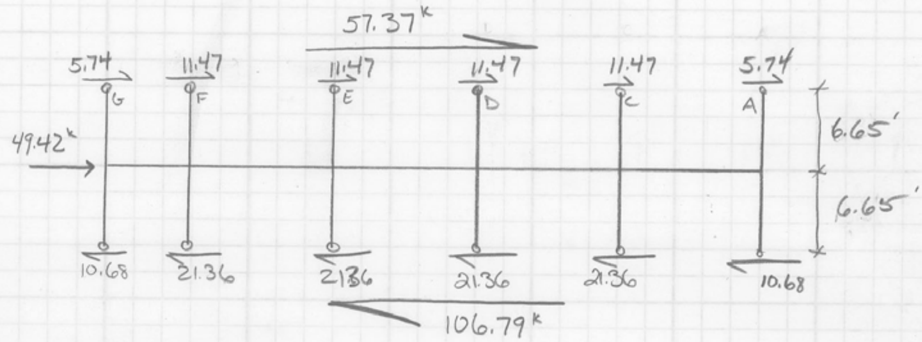
Appendix C-3: Story Drift

Moment Analysis | Tech #3

Portal Method : 3rd Story



Real Work Method

$W_e = \frac{P\Delta}{2}$ $W_i = \int \frac{m^2}{2EI}\,dx$

$W_e = \left[\frac{5.74\Delta}{2}\right]2 + \left[\frac{11.47\Delta}{2}\right]4 + \left[49.42 \times \frac{1}{6}\right]$

$W_e = 36.92\,\Delta$

I column

$I_{AC} = 209\,in^4$
$I_{CC} = 425\,in^4$
$I_{DC} = 425\,in^4$
$I_{EC} = 475\,in^4$
$I_{FC} = 307\,in^4$
$I_{GC} = 209\,in^4$

$E = 29000\,ksi$

$$W_i = 2\int_0^{6.5}\frac{(5.74x)^2}{2EI_A}dx + 2\int_0^{6.5}\frac{(11.47x)^2}{2EI_c} + \int_0^{6.5}\frac{(11.47x)^2}{2EI_E} + \int_0^{6.5}\frac{(11.47x)^2}{2EI_F}$$
$$+ 2\int_0^{6.5}\frac{(10.68x)^2}{2EI_A} + 2\int_0^{6.5}\frac{(21.36x)^2}{2EI_c} + \int_0^{6.5}\frac{(21.36x)^2}{2EI_E} + \int_0^{6.5}\frac{(21.36x)^2}{2EI_F}$$

$W_i = 4.976 \times 10^{-4} + 9.77 \times 10^{-4} + 4.37 \times 10^{-4} + 6.76 \times 10^{-4} + .0017 + .0034 + .0015 + .0023$

$W_i = .0115$

$W_e = W_i$

$36.92\,\Delta = .0115$

$\Delta = .000312$ inches

$h/240 = \frac{13.33(12")}{240} = .665"$

$.000312 << .665 \therefore$ ✓ ok

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Appendix C-4: Column Check

Spot Checking

Tech #3

Column check

$A_T = \frac{(28' \times 28')(28' \times 28')}{2}$ (1 story)

$= 1568\ ft^2$

$K_{LL} = 4$ (column)

$A_I = 1568(4) = 6272\ ft^2$

Live Load Reduction

$L = L_o\left(.25 + \frac{15}{\sqrt{A_I}}\right)$

$L = .474 L_o > .4L \rightarrow$ use $.474 L_o$

$L = 100(.474) = 47.4$

+

+

$\frac{wl^2}{11}$ ← interior column

$M = 61.4^k$

$P = 1.2D + 1.6W + .5L + .5S$

$= 1.2(136.1) + 1.6(15.6) + .5(47.4) + .5(20)$

↑ weight of 4th floor above

$P = 221.98$

$P_u = PA_T = 221.98(1568)$
$= 348.06$

$P_{eff} = P_u + \alpha M_u$

$= 348.06 + \left(\frac{24}{12}\right)(109.41^{k} + 61.4^{k})$

$P_{eff} = 689.68^k$

use W12x72 $\phi P_n = 717^k$

$717^k > 689.68^k$ ∴ OK ✓

The actual member used is

W12x79 $\phi P_n = 790^k$