Dulles Town Center Building One

Dulles, Virginia

Technical Report III

Prepared for: Dr. Linda Hanagan

Prepared by: David Geiger - Structural Option

November 21, 2008

Table of Contents

Executive Summary

The Dulles Town Center Building One project is located in Dulles, Virginia; five minutes north of Dulles International Airport and 25 miles outside of Washington, D.C. It consists of seven stories of office space above grade and one story below grade that includes rentable space, storage, mechanical rooms, a loading area, a trash room, building service offices, and a workout space. The building is approximately 202,000 square feet and reaches a total height of 118 feet above grade. The building has an open floor plan and an average floor-to-floor height of 12'-6" making it ideal for office space. A typical bay is 20 feet by 40 feet, and the gravity system consists of a post-tension concrete beam and non-post-tension one-way slab system with typical 24"x24" columns.

In this third technical report a detailed investigation of the existing lateral system of Dulles Town Center Building One was conducted using computer analysis and hand calculations. Direct and torsional shears caused by wind and seismic loads were computed and used to find the total lateral force at each story. From this, frame analyses were performed by both computer software and hand calculations to access the strength of certain critical members within the lateral system.

The results from the computer analysis and hand calculations verify that wind loads control lateral design in the east-west direction while seismic forces control in the north-south direction. Using the calculated story and base shears, member checks were performed to confirm member sizes while drift analysis was used to validate that they were acceptable when compared to code limits. The member checks showed that the large beams could take the moments from the lateral and gravity loads, but the drop panel along the exterior could not as designed. Uplift and overturning were not considered in this report. The gravity loads from of the building along with the soil friction on the caissons is more than enough to assume overturning could be neglegted.

Introduction

The Dulles Town Center Building One project consists of seven stories of office space above grade and one story below grade that includes rentable space, storage, mechanical rooms, a loading area, a trash room, building service offices, and a workout space. It is located in Dulles, Virginia; five minutes north of Dulles International Airport and 25 miles outside of Washington, D.C. The building's architectural use of precast concrete and glass curtain-wall have helped set the tone for the modernist themes conveyed along the Route 28 corridor. At night, this building is one of the most recognizable buildings along Route 28 with its linear neon focal points.

The building is approximately 202,000 square feet and reaches a total height of 118 feet above grade. The building has an open floor plan and an average floor-to-floor height of 12'-6" making it ideal for office space. A typical bay is 20 feet by 40 feet, and consists of a post-tension concrete beam and non-post-tension one-way slab system.

The following report will investigate the current lateral resistance system of Dulles Town Center Building One with a brief overview of the structural system along with a more in-depth look at the lateral load distribution using computer analysis and hand calculations. The following topics will be determined and discussed to help explain the methods of analysis and results:

- Direct and Torsional Shear Forces
- Controlling Load Combinations
- Load Distribution
- Building Drift
- Strengths of Lateral Members

Existing Structural System Overview

Floor Systems

The typical floor is a post-tensioned beam and non-post-tensioned one-way slab system. The 7″ thick slab is of normal weight with continuous edge drops that are 3′ wide and 5 ½″ deep along the east face to help support the precast concrete and ribbon window façade. A typical bay is 20′x 40′ with a typical beam length of 40′. Slab reinforcement consists of #4 top bars spaced at 6″ on center and #4 bottom bars at 12″ on center. Reinforced concrete beams are located at stairwells and elevator shafts.

Lateral System

The lateral resistance system in the east-west direction, as seen in Figure 1, is comprised predominantly of concrete moment frames with typical beams being 17″x 48″ and typical columns being 24″x 24″.

Typical Floor - Concrete Moment Frame in East – West Direction

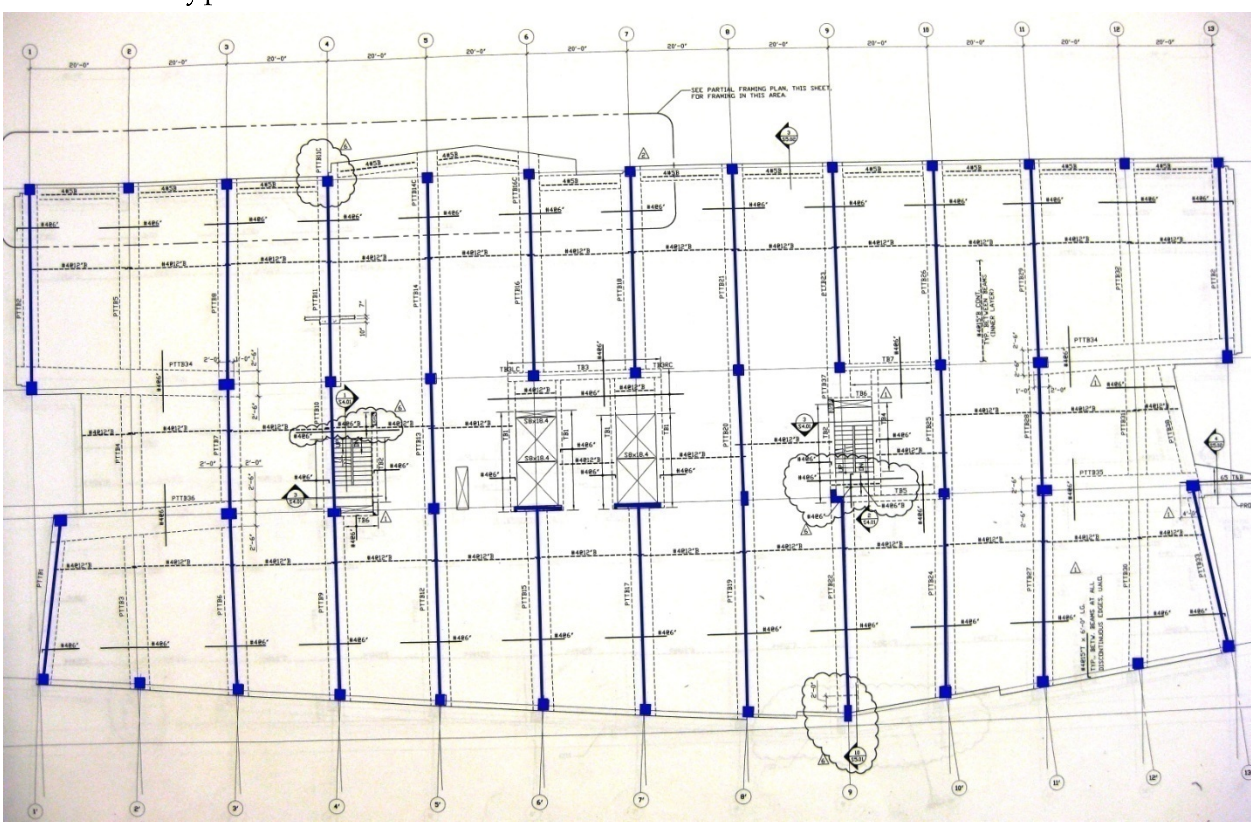


Figure 1

The north-south lateral system, seen in Figure 2, is also made up of concrete moment frames. The middle frames have large 24″x 60″ beams, shown as solid lines, at the frame-ends with the floor slab working laterally throughout the rest of the frame, shown with dashed lines, on typical 24″x 24″ columns. The exterior frames use the 7″ slab, along with a 36″x 5 ½″ drop panel along the frame at plan north, with typical 24″x 24″ columns for lateral resistance.

Typical Floor - Concrete Moment Frame in East – West Direction

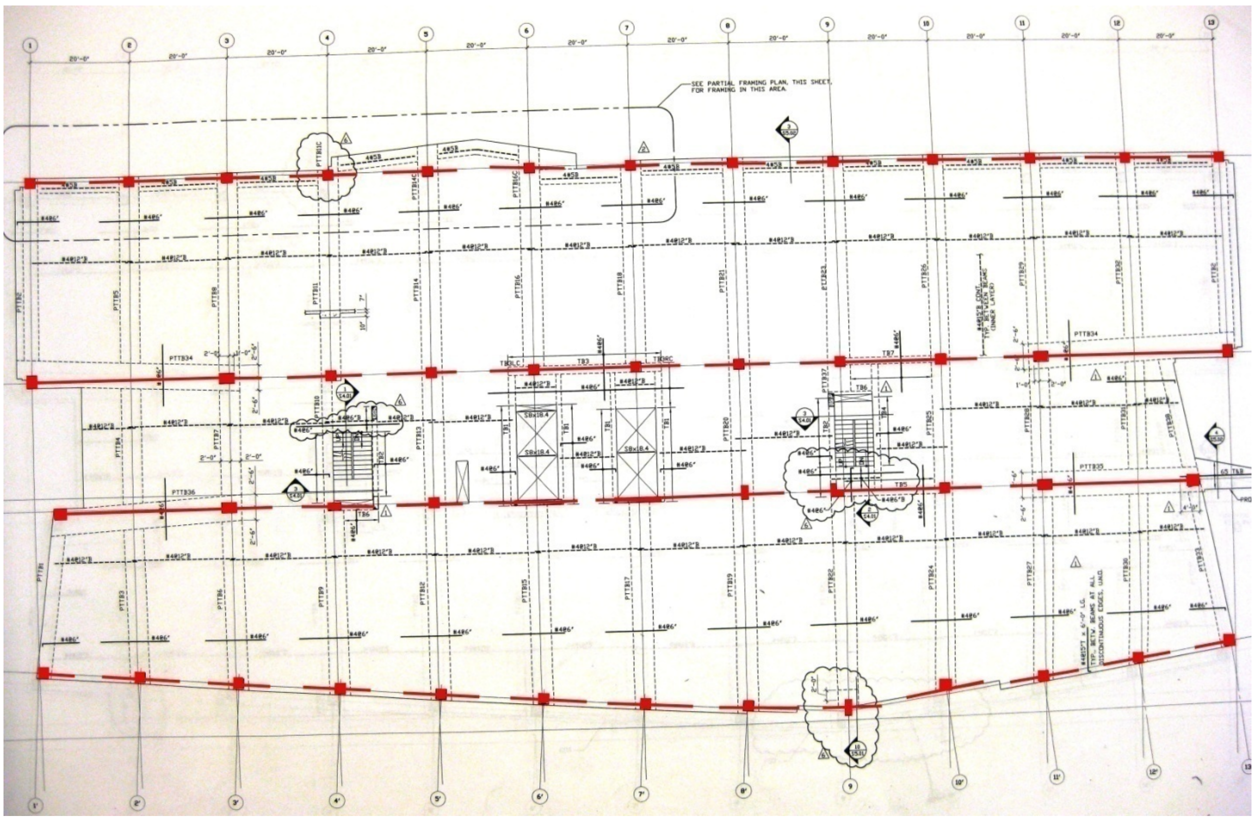


Figure 2

Foundation

The foundation system consists of a slab on grade with strap beams and caissons. The slab is 5″ thick and reinforced with 6x6 – W2.0xW2.0 welded wire fabric. It sits on a 6 mil. polyethylene vapor barrier over 6″ of washed, crushed stone. Strap beams ranging from 24″x 36″ to 48″x 48″ rest on a 2′-0″ thick foundation wall to help support the slab at grade changes. The cast-in-place caissons are capped with reinforced concrete and have shaft diameters that range from 30″ to 75″.

Codes and Design Criteria

These are the codes, load cases, and design criterion used to assist in the analysis of Dulles Town Center Building One's existing lateral system.

Codes and References

- Building Code Requirements for Structural Concrete (ACI 318-08), American Concrete Institute (ACI)
- International Building Code 2006
- Minimum Design Loads for Buildings and Other Structures (ASCE 7-05), American Society of Civil Engineers (ASCE)
- *STAAD.Pro 2006*
- *PCA Column*

Load Cases and Combinations

The following are the load cases considered for analysis per IBC 2006, Section 1605.2:

$1.4D$
$1.2D + 1.6L + .5L_r$
$1.2D + 1.6L_r + (1.0L \text{ or } .8W)$
$1.2D + 1.6W + 1.0L + .5L_r$
$1.2D + 1.0E + 1.0L$
$.9D + 1.6W$
$.9D + 1.0E$

Deflection Criteria

The following are design criteria considered for analysis per ASCE 7-05:

Allowable Story and Building Drift due to Wind Forces:

$\Delta_{WIND} = h/400$

Allowable Story and Building Drift due to Seismic Forces:

$\Delta_{SEISMIC} = .025h_{sx}$

Load Distribution and Analysis

The distribution of lateral loads to Dulles Town Center Building One is based on the relative stiffness of each frame within the lateral system. The 7″ concrete floor slab is considered a rigid diaphragm and thus distributes load to the reinforced concrete moment frames according to their stiffness. When conducting the frame analysis an approved simplified typical floor layout, which is shown below, was used. In the north-south direction columns are typically spaced at 20′ on center and in the east-west direction the columns of the outer bays and inner bay are spaced at 40′ and 25′-6″ on center, respectively.

Approved Simplified Floor Plan - Typical Frame Layout

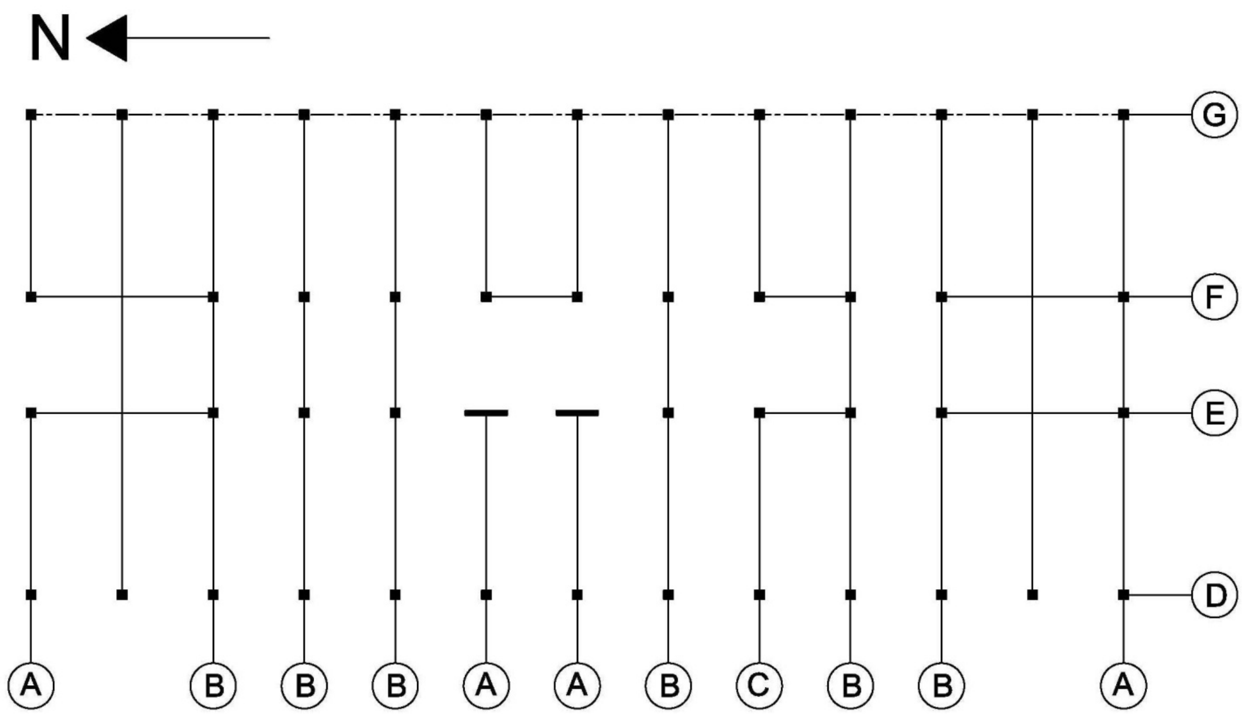


Figure 3

To begin the lateral analysis, the relative stiffness of each frame had to be determined. First, gross moments of inertia, I_g, were taken from the existing beams and slab areas along each frame and adjusted per ACI 10.10.4.1. A factor of .7 was applied to beams and a factor of .25 to slab areas. Members with equivalent moments of inertia were then placed in *STAAD.Pro 2006* to get a frame that could be used for analysis. A one kilo-pound load was then applied to each type of frame and the deflection obtained was placed in the following equation to get the relative stiffness. A stiffness layout can be found in Appendix A.

$$k = \frac{P}{\Delta}$$

Direct shears from wind and seismic loads were then determined using the relative stiffness and the initial loads in the following equation:

$$F_{DIRECT} = \frac{k_i}{\sum k} F_{INITIAL}$$

The center of rigidity and center of mass were then calculated. Calculations for both can be found in Appendix A. These two centers are used to find shear forces caused by torsion when wind and seismic loads are applied to the building. The following equation is how those forces were derived:

$$F_{TORSION} = \frac{M\ k_i x_i}{I_p}$$

The Calculation for $\boldsymbol{I_p}$ is located in Appendix A. Table 1 shows the torsion constants used to find total story shears.

Torsion Constants							
Floor	Center of Mass		Center of Rigity		I_x (in^4)	I_y (in^4)	I_p (in^4)
	x_r (ft)	y_r (ft)	x_r (ft)	y_r (ft)			
Roof	122.26	52.75	119.30	52.34	1057807	1052000	2109807
Seventh	122.26	52.75	119.30	52.34	1057807	1052000	2109807
Sixth	122.26	52.75	119.30	52.34	1057807	1052000	2109807
Fifth	122.26	52.75	119.30	52.34	1057807	1052000	2109807
Fourth	122.26	52.75	119.30	52.34	1057807	1052000	2109807
Third	122.26	52.75	119.30	52.34	1057807	1052000	2109807
Second	122.26	52.75	119.30	52.34	1057807	1052000	2109807

Table 1

Wind Analysis: ASCE 7-05, Chapter 6

Initial wind loads for each level were calculated using the Analytical Procedure found in Section 6.5. For this analysis the lateral loads on the architectural fin were added to the roof. Wind design factors can be found in Appendix B. Initial wind forces, direct shear tables and torsional shear tables can be found in Appendix C. Tables 2 and 3 show the resulting shears at each floor caused by wind forces.

Total Shear Due to Wind						
Floor	Frame					
	A			B		
	Direct Shear (kips)	Torsional Shear (kips)	Total Shear (kips)	Direct Shear (kips)	Torsional Shear (kips)	Total Shear (kips)
Roof	10.78	0.0039	10.78	16.16	0.0062	16.17
Seventh	4.04	0.0015	4.05	6.07	0.0023	6.07
Sixth	3.86	0.0014	3.86	5.79	0.0022	5.79
Fifth	3.73	0.0013	3.73	5.59	0.0022	5.60
Fourth	3.59	0.0013	3.59	5.38	0.0021	5.39
Third	3.42	0.0012	3.42	5.13	0.0020	5.13
Second	3.49	0.0013	3.49	5.23	0.0020	5.23

Table 2

Total Shear Due to Wind						
Floor	Frame					
	F			G		
	Direct Shear (kips)	Torsional Shear (kips)	Total Shear (kips)	Direct Shear (kips)	Torsional Shear (kips)	Total Shear (kips)
Roof	9.28	0.0119	9.29	1.95	0.0073	1.96
Seventh	8.63	0.0087	8.64	1.82	0.0063	1.82
Sixth	8.17	0.0082	8.18	1.72	0.0060	1.73
Fifth	7.86	0.0079	7.87	1.65	0.0058	1.66
Fourth	7.50	0.0076	7.51	1.58	0.0055	1.58
Third	7.06	0.0071	7.07	1.49	0.0052	1.49
Second	7.08	0.0072	7.08	1.49	0.0052	1.49

Table 3

As a result, wind was found to control lateral design in the east-west direction.

Seismic Analysis ASCE 7-05, Chapter 12

Initial seismic forces were determined using the Equivalent Lateral Force Procedure found in Section 12.8. For this analysis, the gravity loads from the architectural fin and penthouse were included in the roof. For general seismic design information refer to Appendix B. For initial seismic shears, total gravity loads, and direct and torsional shear tables refer to Appendix D. Tables 4 and 5 show the resulting shears at each floor caused by seismic forces.

Total Shear Due to Seismic						
Floor	Frame					
	A			B		
	Direct Shear (kips)	Torsional Shear (kips)	Total Shear (kips)	Direct Shear (kips)	Torsional Shear (kips)	Total Shear (kips)
Roof	8.23	0.0110	8.24	12.35	0.0131	12.36
Seventh	5.89	0.0079	5.90	8.84	0.0094	8.85
Sixth	4.82	0.0064	4.83	7.23	0.0077	7.24
Fifth	3.73	0.0050	3.73	5.59	0.0059	5.59
Fourth	2.68	0.0036	2.69	4.03	0.0043	4.03
Third	1.70	0.0023	1.71	2.56	0.0027	2.56
Second	0.82	0.0011	0.82	1.23	0.0013	1.23

Table 4

Total Shear Due to Seismic						
Floor	Frame					
	F			G		
	Direct Shear (kips)	Torsional Shear (kips)	Total Shear (kips)	Direct Shear (kips)	Torsional Shear (kips)	Total Shear (kips)
Roof	45.20	0.0349	45.23	9.51	0.0073	9.52
Seventh	32.35	0.0250	32.38	6.81	0.0053	6.81
Sixth	26.48	0.0204	26.50	5.57	0.0043	5.58
Fifth	20.45	0.0158	20.47	4.30	0.0033	4.31
Fourth	14.74	0.0114	14.75	3.10	0.0024	3.10
Third	9.35	0.0072	9.36	1.97	0.0015	1.97
Second	4.51	0.0035	4.51	0.95	0.0007	0.95

Table 5

As a result, seismic forces control lateral loading in the north-south direction. The total shears for both wind and seismic loads prove to be smaller than those derived from tributary area. This is because the loads are distributed more efficiently to each frame using a ratio of frame stiffness over total stiffness. This allows for computed forces applied to each frame to be more accurate. The higher accuracy results in more reliable portal frame, strength and drift analyses.

Frame Analysis

Portal Frame Analysis

After direct and torsional shears were calculated, story forces were applied to the structure. Using the portal frame method, frames A, B, F and G were analyzed. Frames A and B run in the east-west direction and were analyzed using the total shears caused by wind. Frames F and G run in the north-south direction and were analyzed using total shears caused by seismic forces. The resulting moments in the beams and columns found through this analysis were then used in the strength checks. Results from the portal frame analyses can be found in Appendix C.

STAAD Analysis

A frame analysis using *STAAD.Pro 2006* was also performed. This analysis was conducted not only to find story and overall building drifts, but to also find moments in the beams and columns. The building drifts were then compared to the allowable drift values seen in Tables 6 and 7, while the moments were compared to those found via the portal frame analysis. Generally, STAAD was used to quickly verify hand calculations and drift analysis. The following figures show wind loading diagrams for frames A and B.

Frame A

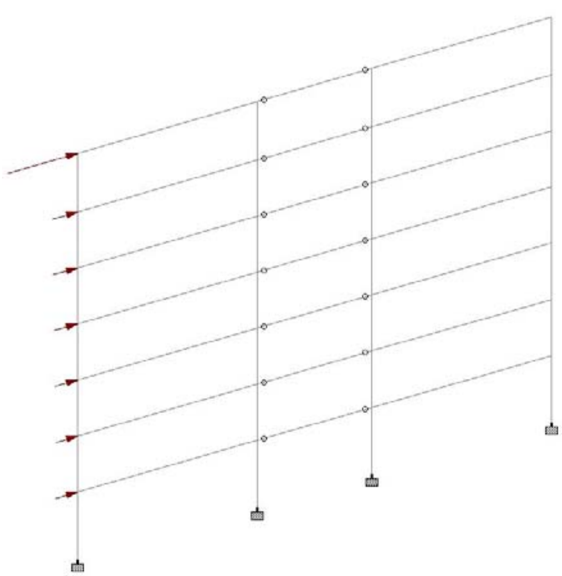


Figure 4

Frame B

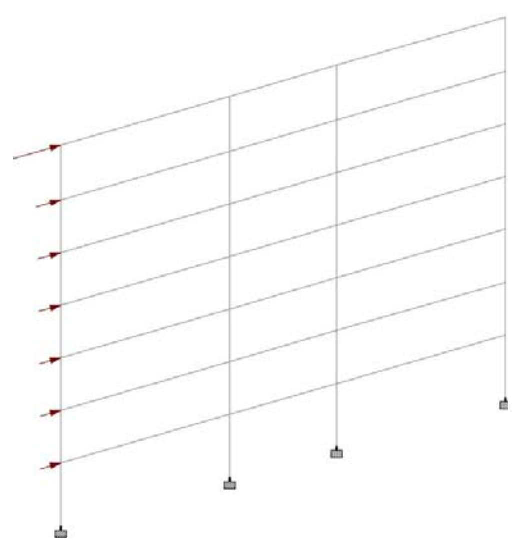


Figure 5

Member Checks

In order to finish the analysis of Dulles Town Center Building One, select members were checked for strength. Dead and live loads used for this part of the analysis can be found in Appendix B. The following lateral system components, seen in Figure 6, were the members checked in this section. They are taken from the third floor and analyzed as such.

Lateral Design Components Checked

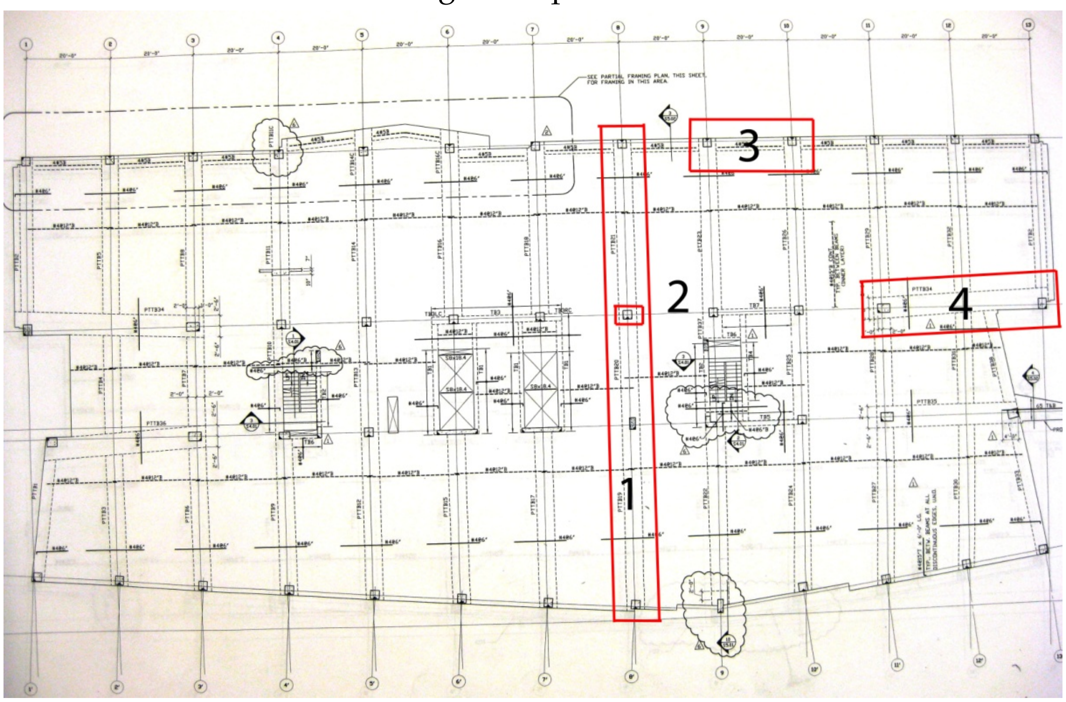


Figure 6

With axial forces and moments calculated from the portal frame analysis along with moments derived from gravity loads, *PCA Column* was used to check a continuous beam (1), an interior column (2), an edge drop panel (3), and a beam running in the north south direction (4). These were chosen due to their location within the lateral resisting system. Refer to Appendix E for partial calculations.

Drift Analysis

Drift is one of the most important factors when dealing with building design. Building facades and other systems, as well as building occupants, can be affected by too much drift. The result could be damage to expensive architectural facades and building systems. Design parameters set forth in the IBC and ASCE 7-05 for both wind and seismic loads are placed to help with this problem. The drifts obtained from STAAD for both wind and seismic forces were compared to these parameters to perform a serviceability check. Drift caused by wind was compared to Δ_W=H/400, which is found in Table 1604.3 of the IBC, while drift caused by seismic forces was compared to Δ_S=.025h_{sx}, which is located in Table 12.12-1 of the ASCE 7-05. Tables 6 and 7 show these comparisons.

Controlling Wind Drift										
Floor	Story Height (ft)	Total Height (ft)	Story Drift (in)	Allowable Story Drift (in) Δ_{WIND} = h/400			Total Drift (in)	Allowable Story Drift (in) Δ_{WIND} = h/400		
Roof	13.0	90.5	0.138	<	0.390	Acceptable	1.492	<	2.715	Acceptable
Seventh	12.5	77.5	0.172	<	0.375	Acceptable	1.354	<	2.325	Acceptable
Sixth	12.5	65.0	0.213	<	0.375	Acceptable	1.182	<	1.950	Acceptable
Fifth	12.5	52.5	0.249	<	0.375	Acceptable	0.969	<	1.575	Acceptable
Fourth	12.5	40.0	0.271	<	0.375	Acceptable	0.720	<	1.200	Acceptable
Third	12.5	27.5	0.263	<	0.375	Acceptable	0.449	<	0.825	Acceptable
Second	15.0	15.0	0.186	<	0.450	Acceptable	0.186	<	0.450	Acceptable

Table 6

Controlling Seismic Drift										
Floor	Story Height (ft)	Total Height (ft)	Story Drift (in)	Allowable Story Drift (in) $\Delta_{SEISMIC}$ = .025h_{sx}			Total Drift (in)	Allowable Story Drift (in) $\Delta_{SEISMIC}$ = .025h_{sx}		
Roof	13.0	90.5	0.125	<	0.325	Acceptable	1.410	<	2.263	Acceptable
Seventh	12.5	77.5	0.163	<	0.313	Acceptable	1.285	<	1.938	Acceptable
Sixth	12.5	65.0	0.209	<	0.313	Acceptable	1.122	<	1.625	Acceptable
Fifth	12.5	52.5	0.244	<	0.313	Acceptable	0.913	<	1.313	Acceptable
Fourth	12.5	40.0	0.260	<	0.313	Acceptable	0.669	<	1.000	Acceptable
Third	12.5	27.5	0.244	<	0.313	Acceptable	0.409	<	0.688	Acceptable
Second	15.0	15.0	0.165	<	0.375	Acceptable	0.165	<	0.375	Acceptable

Table 7

The above tables show that the story and total drifts of Dulles Town Center Building One comply with both wind and seismic parameters. Story drift and total drift for both wind and seismic are acceptable and do not go over 1.5 inches.

Conclusions

Dulles Town Center Building One utilizes ordinary reinforced concrete moment frames as its lateral resistance system and that system was analyzed and discussed in detail within this technical report. Both hand methods and computer software were used throughout this project to find the necessary loads and constants needed for an accurate lateral system analysis. Using *STAAD.Pro 2006*, the relative stiffness of each frame was determined as well as the direct and torsional shears being applied along each frame. Computer analysis and portal frame analysis were then used to find moments and shears at each member within the frame. The moments found through computer modeling were lower than that of the hand method and the reason is due to member being taken into account in STAAD.

The computer output was also used to determine which load case governed lateral design for each direction. Wind forces ended up controlling design in the east-west direction and seismic forces controlled north-south design. STAAD also gave the deflection of each story and of the overall frame, which was then compared to the parameters set forth in IBC 2006 and in ASCE 7-05. Both wind drift and seismic drift were acceptable according to the code.

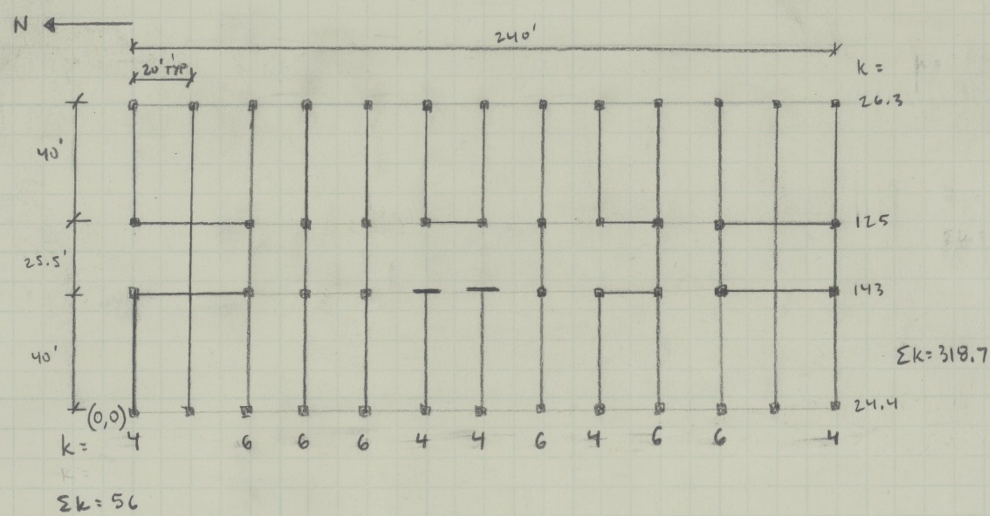
Design strength of critical members within the lateral system was also analyzed using forces derived from the portal frame analysis and gravity loading. These moments and axial forces were put into *PCA Column* and were analyzed and compared to standards set forth ACI 318-08. The continuous beam, interior column and 60"x 24" beam in Frame F were all able to carry the applied loads, but the drop panel located along Frame G failed to work. This could be due to the initial assumptions when putting members with equivalent gross moments of inertia into STAAD to be analysis. Uplift and overturning were neglected in this report because by inspection, the gravity loads of the building along with the friction of soil against the caisson foundation system are large enough to prevent those happening.

Ultimately, the lateral system of Dulles Town Center Building One is adequate designed to carry the lateral loads applied to it. The member strengths will be discussed and investigated further through the proposal and research to follow.

Appendix A

General Lateral Calculations

CENTER OF RIGIDITY



FINDING CENTER OF RIGIDITY

$$X_r = \frac{\Sigma k_{y_i} x_i}{\Sigma k_{y_i}}$$

$$x_r = \frac{\left[(4)(0)+(6)(40')+(6)(60')+(6)(80')+(4)(100')+4(120')+6(140')+4(160')+6(180')+6(200')+4(240')\right]}{4+6+6+6+4+4+6+4+6+6+4}$$

$$x_r = \frac{6680'}{56} = 119.3'$$

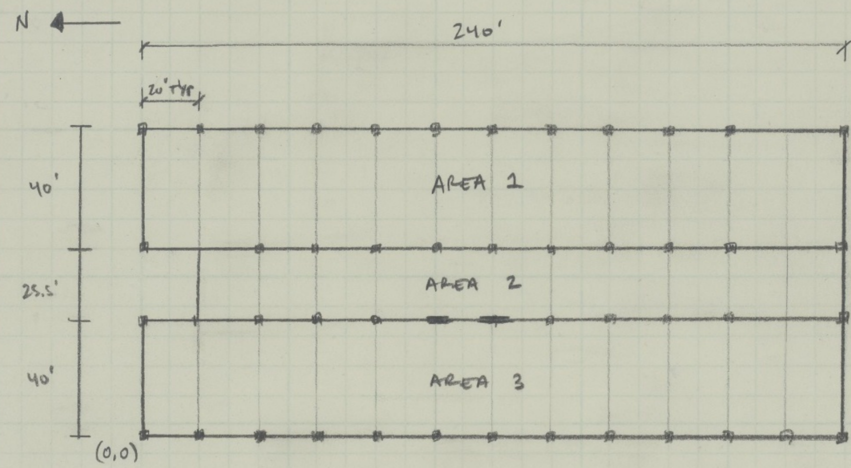
$$Y_r = \frac{\Sigma k_{x_i} y_i}{\Sigma k_{x_i}}$$

$$Y_r = \frac{\left[(24.4)(0)+(143)(40')+(125)(65.5')+(26.3)(105.5')\right]}{26.3+125+143+24.4}$$

$$Y_r = \frac{16682.15}{318.7} = 52.34'$$

CENTER OF RIGIDITY : $(119.3', 52.34')$

CENTER OF MASS



$A_1 = 9600\ ft^2$ CENTROID$_1$ = (120, 85.5)

$A_2 = 5610\ ft^2$ CENTROID$_2$ = (130, 52.75)

$A_3 = 9600\ ft^2$ CENTROID$_3$ = (120, 20)

$$\bar{x} = \frac{(9600)(120) + (5610)(130) + (9600)(120)}{9600 + 5610 + 9600}$$

$$\bar{x} = \frac{1152000 + 729300 + 1152000}{24810} = 122.26'$$

$$\bar{y} = \frac{(9600)(85.5) + (5610)(52.75) + (9600)(20)}{9600 + 5610 + 9600}$$

$$\bar{y} = \frac{820800 + 295927.5 + 192000}{24810} = 52.75'$$

CENTER OF MASS: (122.26', 52.75')

I values used for Torsion

I VALUES FOR
TORSIONAL SHEAR

$I_x = \Sigma k_{i_x} y_i^2$

$I_x = [(24.4)(0)^2 + (143)(40')^3 + (125)(65.5)^2 + (26.3)(105.5)^2]$

$= 1057807$

$I_y = \Sigma k_{i_y} x_i^2$

$I_y = [4(0)^2 + 6(40)^2 + 6(60)^2 + 6(80)^2 + 4(100)^2 + 4(120)^2 + 6(140)^2 + 4(160)^2 + 6(180)$

$+ 6(200)^2 + 4(240)^2]$

$= 1052000$

$I_p = I_x + I_y$

$= 1057807 + 1052000 = 2109807$

Appendix B

Building Design Loads and Criteria

Gravity Loads

Live Loads	
Description	Load (lb/ft^2)
Floor	100
Mechanical	150
Elevator Machine Room	125
Stairs and Corridors	100
Slab on Grade	100

Roof Live Loads	
Description	Load (lb/ft^2)
Rain	35
Snow	21

Dead Loads	
Material	Load
Reinforced Concrete	150 pcf
Steel	per member
Curtain Wall	15 psf
Glass Windows	8 psf
Metal Panels	3 psf
Ceiling	15 psf

Wind Design Criteria

Gust Factor Variables				
H (ft)	n_1	g_q	g_v	g_R
118	0.549	3.4	3.4	4.057
V (mph)	b	c	β	α
90	0.45	0.3	2	7

Wind Variables

North - South Wind Direction								
ż (ft)	I_z	L_z	B	L	Q	V_z (ft/s)	N_1	h
70.8	0.264	406.21	105.5	240	0.797	71.89	3.41	118
Rn	η_h	R_h	η_B	R_B	η_L	R_L	R	G_f
0.0645	4.485	0.1981	4.01	0.218	28.23	0.0344	0.276	0.87

East - West Wind Direction								
ż (ft)	I_z	L_z	B	L	Q	V_z (ft/s)	N_1	h
67.5	0.266	412.72	240	105.5	0.837	71.04	3.4	112.5
Rn	η_h	R_h	η_B	R_B	η_L	R_L	R	G_f
0.065	4.3	0.206	9.23	0.102	13.58	0.071	0.196	0.83

General Seismic Design Information

General Seismic Information		
Occupancy Category		II
Site Class		B
Seismic Design Category		A
Short Period Spectral Response	S_s	0.16
Spectral Response (1 sec)	S_1	0.051
Maximum Short Period Spectral Response	S_{MS}	0.16
Maximum Spectral Response (1 sec)	S_{M1}	0.051
Design Short Period Spectral Response	S_{DS}	0.107
Design Spectral Response (1 sec)	S_{D1}	0.034
Response Modification Coefficient	R	3
Seismic Response Coefficient	C_s	0.012
Approx. Fundamental Period	T_a	0.923 s
Height Above Grade	h_n	90.5 ft
Base Shear	V	390.25k

Appendix C

Wind Calculations and Tables

Initial Wind Tables

Wind (North - South Direction)									
Floor	Height (ft)	Tributary Height (ft)	K_z	q_z	Windward (psf)	Leeward (psf)	Total (psf)	Story Force (kips)	Story Shear (kips)
Ground	0.00	0.00	0.575	0.000	0.00	0.00	0.00	0.00	155.95
Second	15.00	13.75	0.575	10.130	7.04	-4.76	11.80	17.12	155.95
Third	27.50	12.50	0.683	12.045	8.37	-4.76	13.13	17.32	138.83
Fourth	40.00	12.50	0.761	13.406	9.32	-4.76	14.08	18.57	121.51
Fifth	52.50	12.50	0.822	14.489	10.07	-4.76	14.83	19.56	102.94
Sixth	65.00	12.50	0.874	15.401	10.71	-4.76	15.47	20.40	83.38
Seventh	77.50	12.75	0.919	16.195	11.26	-4.76	16.02	21.55	62.98
Roof	90.50	20.25	0.960	16.928	11.77	-4.76	16.53	35.31	41.43
Top of Fin	118.00	13.75	1.036	18.262	12.70	-4.76	17.46	6.12	6.12

Wind (East - West Direction)									
Floor	Height (ft)	Tributary Height (ft)	K_z	q_z	Windward (psf)	Leeward (psf)	Total (psf)	Story Force (kips)	Story Shear (kips)
Ground	0.00	0.00	0.575	0.000	0.00	0.00	0.00	0.00	411.92
Second	15.00	13.75	0.575	10.130	6.73	-7.48	14.20	46.87	411.92
Third	27.50	12.50	0.683	12.045	8.00	-7.48	15.47	46.42	365.05
Fourth	40.00	12.50	0.761	13.406	8.90	-7.48	16.38	49.13	318.63
Fifth	52.50	12.50	0.822	14.489	9.62	-7.48	17.10	51.29	269.50
Sixth	65.00	12.50	0.874	15.401	10.23	-7.48	17.70	53.11	218.21
Seventh	77.50	12.75	0.919	16.195	10.75	-7.48	18.23	55.78	165.10
Roof	90.50	15.25	0.960	16.928	11.24	-7.48	18.72	68.50	109.32
Mean Fin Ht.	112.50	8.75	1.022	18.014	11.96	-7.48	19.44	40.82	40.82

Direct Shear Tables

Direct Shear Due to Wind (North-South Direction)					
Floor	Frame Stiffness (k/in)		Story Force (kips)	Direct Shear (kips)	
	F	G		F	G
Roof	125	26.3	23.66	9.28	1.95
Seventh	125	26.3	22.00	8.63	1.82
Sixth	125	26.3	20.84	8.17	1.72
Fifth	125	26.3	20.05	7.86	1.65
Fourth	125	26.3	19.12	7.50	1.58
Third	125	26.3	18.00	7.06	1.49
Second	125	26.3	18.04	7.08	1.49

Direct Shear Due to Wind (East-West Direction)					
Floor	Frame Stiffness (k/in)		Story Force (kips)	Direct Shear (kips)	
	A	B		A	B
Roof	4	6	150.85	10.78	16.16
Seventh	4	6	56.62	4.04	6.07
Sixth	4	6	54.00	3.86	5.79
Fifth	4	6	52.20	3.73	5.59
Fourth	4	6	50.25	3.59	5.38
Third	4	6	47.85	3.42	5.13
Second	4	6	48.81	3.49	5.23

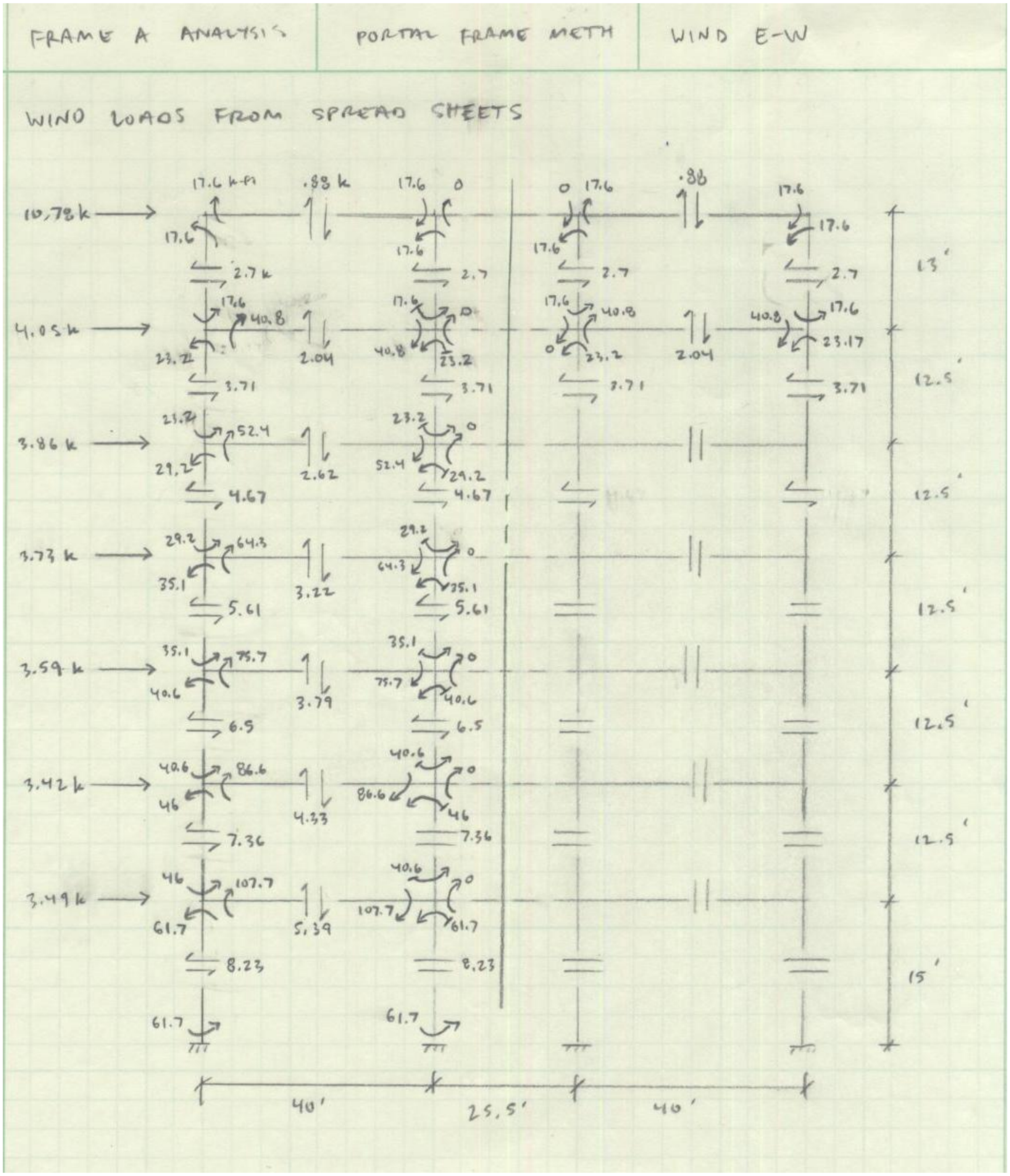
Torsional Shear Tables

Torsional Shear Due to Wind (North - South Direction)							
Floor	Moment (k-ft)	Frame					
		A			B		
		k (k/in)	x_i (ft)	Torsional Shear (kips)	k (k/in)	x_i (ft)	Torsional Shear (kips)
Roof	9.70	4	0.41	0.0000	6	0.41	0.0000
Seventh	9.02	4	0.41	0.0000	6	0.41	0.0000
Sixth	8.54	4	0.41	0.0000	6	0.41	0.0000
Fifth	8.22	4	0.41	0.0000	6	0.41	0.0000
Fourth	7.84	4	0.41	0.0000	6	0.41	0.0000
Third	7.38	4	0.41	0.0000	6	0.41	0.0000
Second	7.40	4	0.41	0.0000	6	0.41	0.0000

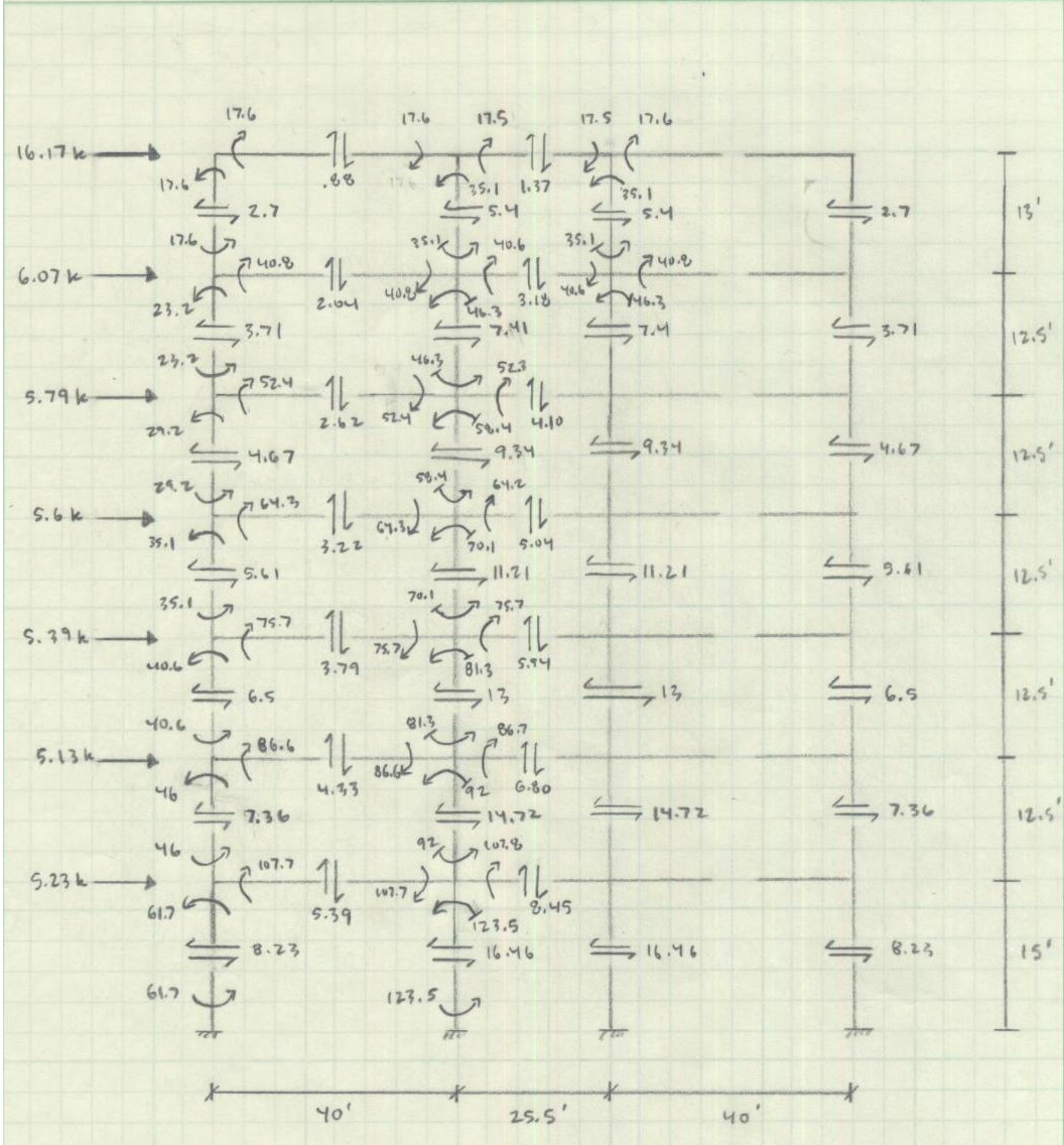
Torsional Shear Due to Wind (North - South Direction)							
Floor	Moment (k-ft)	Frame					
		F			G		
		k (k/in)	x_i (ft)	Torsional Shear (kips)	k (k/in)	x_i (ft)	Torsional Shear (kips)
Roof	9.70	125	13.16	0.0076	26.3	53.16	0.0064
Seventh	9.02	125	13.16	0.0070	26.3	53.16	0.0060
Sixth	8.54	125	13.16	0.0067	26.3	53.16	0.0057
Fifth	8.22	125	13.16	0.0064	26.3	53.16	0.0054
Fourth	7.84	125	13.16	0.0061	26.3	53.16	0.0052
Third	7.38	125	13.16	0.0058	26.3	53.16	0.0049
Second	7.40	125	13.16	0.0058	26.3	53.16	0.0049

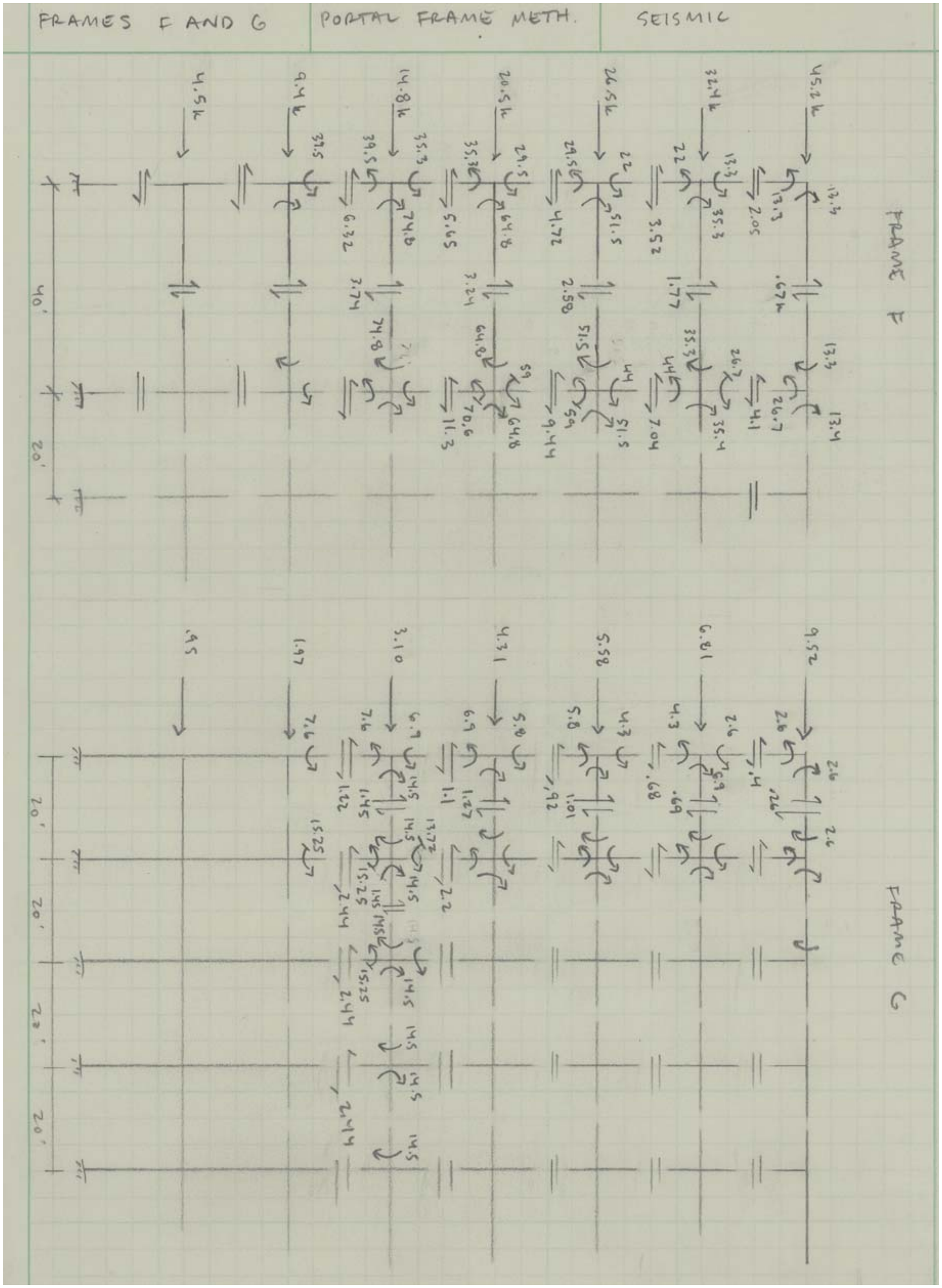
Torsional Shear Due to Wind (East - West Direction)							
Floor	Moment (k-ft)	Frame					
		A			B		
		k (k/in)	x_i (ft)	Torsional Shear (kips)	k (k/in)	x_i (ft)	Torsional Shear (kips)
Roof	105.60	4	19.3	0.0039	6	20.7	0.0062
Seventh	39.63	4	19.3	0.0015	6	20.7	0.0023
Sixth	37.80	4	19.3	0.0014	6	20.7	0.0022
Fifth	36.54	4	19.3	0.0013	6	20.7	0.0022
Fourth	35.18	4	19.3	0.0013	6	20.7	0.0021
Third	33.50	4	19.3	0.0012	6	20.7	0.0020
Second	34.17	4	19.3	0.0013	6	20.7	0.0020

Torsional Shear Due to Wind (East - West Direction)							
Floor	Moment (k-ft)	Frame					
		F			G		
		k (k/in)	x_i (ft)	Torsional Shear (kips)	k (k/in)	x_i (ft)	Torsional Shear (kips)
Roof	105.60	125	0.7	0.0044	26.3	0.7	0.0009
Seventh	39.63	125	0.7	0.0016	26.3	0.7	0.0003
Sixth	37.80	125	0.7	0.0016	26.3	0.7	0.0003
Fifth	36.54	125	0.7	0.0015	26.3	0.7	0.0003
Fourth	35.18	125	0.7	0.0015	26.3	0.7	0.0003
Third	33.50	125	0.7	0.0014	26.3	0.7	0.0003
Second	34.17	125	0.7	0.0014	26.3	0.7	0.0003


FRAME A ANALYSIS
PORTAL FRAME METH
WIND E-W
WIND LOADS FROM SPREAD SHEETS
10.78 k
4.05 k
3.86 k
3.73 k
3.59 k
3.42 k
3.49 k
13'
12.5'
12.5'
12.5'
12.5'
12.5'
15'
40'
25.5'
40'

FRAME B ANALYSIS | PORTAL FRAME METH. | WIND E-W




FRAMES F AND G
PORTAL FRAME METH.
SEISMIC
FRAME F
FRAME G

Appendix D
Seismic Calculations and Tables

Initial Seismic Table

Seismic Base Shear							
Floor	Height (ft)	Tributary Height (ft)	Dead Load (kips)	$w_x h_x^k$	C_{vx}	Lateral Force (kips)	Story Shear (kips)
Roof	90.5	6.5	5609.9	1307675	0.29531	115.24	115.24
Seventh	77.5	12.75	4843.4	935848.2	0.211341	82.48	197.72
Sixth	65	12.5	4905.7	766171.8	0.173023	67.52	265.24
Fifth	52.5	12.5	4905.7	591689.3	0.13362	52.15	317.39
Fourth	40	12.5	4905.7	425788.2	0.096155	37.52	354.91
Third	27.5	12.5	4905.7	270578.7	0.061104	23.85	378.76
Second	15	13.75	4922.7	130398.9	0.029448	11.49	390.25
Ground	0	7.5	4026	0	0	0.00	390.25
Total	90.5		39024.8	4428150	1.0000	390.25	390.25

Direct Shear Tables

Direct Shear Due to Seismic (North-South Direction)					
Floor	Frame Stiffness (k/in)		Story Force (kips)	Direct Shear (kips)	
	F	G		F	G
Roof	125	26.3	115.24	45.20	9.51
Seventh	125	26.3	82.48	32.35	6.81
Sixth	125	26.3	67.52	26.48	5.57
Fifth	125	26.3	52.15	20.45	4.30
Fourth	125	26.3	37.57	14.74	3.10
Third	125	26.3	23.85	9.35	1.97
Second	125	26.3	11.49	4.51	0.95

Direct Shear Due to Seismic (East-West Direction)					
Floor	Frame Stiffness (k/in)		Story Force (kips)	Direct Shear (kips)	
	A	B		A	B
Roof	4	6	115.24	8.23	12.35
Seventh	4	6	82.48	5.89	8.84
Sixth	4	6	67.52	4.82	7.23
Fifth	4	6	52.15	3.73	5.59
Fourth	4	6	37.57	2.68	4.03
Third	4	6	23.85	1.70	2.56
Second	4	6	11.49	0.82	1.23

Torsional Shear Tables

Torsional Shear Due to Seismic (East - West Direction)							
Floor	Moment (k-ft)	Frame					
		A			B		
		k (k/in)	x_i (ft)	Torsional Shear (kips)	k (k/in)	x_i (ft)	Torsional Shear
Roof	-260.45	4	-22.26	0.0110	6	17.74	-0.0131
Seventh	-186.40	4	-22.26	0.0079	6	17.74	-0.0094
Sixth	-152.60	4	-22.26	0.0064	6	17.74	-0.0077
Fifth	-117.85	4	-22.26	0.0050	6	17.74	-0.0059
Fourth	-84.81	4	-22.26	0.0036	6	17.74	-0.0043
Third	-53.89	4	-22.26	0.0023	6	17.74	-0.0027
Second	-25.97	4	-22.26	0.0011	6	17.74	-0.0013

Torsional Shear Due to Seismic (East - West Direction)							
Floor	Moment (k-ft)	Frame					
		F			G		
		k (k/in)	x_i (ft)	Torsional Shear (kips)	k (k/in)	x_i (ft)	Torsional Shear
Roof	-260.45	125	-2.26	0.0349	26.3	-2.26	0.0073
Seventh	-186.40	125	-2.26	0.0250	26.3	-2.26	0.0053
Sixth	-152.60	125	-2.26	0.0204	26.3	-2.26	0.0043
Fifth	-117.85	125	-2.26	0.0158	26.3	-2.26	0.0033
Fourth	-84.81	125	-2.26	0.0114	26.3	-2.26	0.0024
Third	-53.89	125	-2.26	0.0072	26.3	-2.26	0.0015
Second	-25.97	125	-2.26	0.0035	26.3	-2.26	0.0007

- Note: There is no torsional shear in the north-south direction due to the resultant force being located on the x-axis of the center of mass.

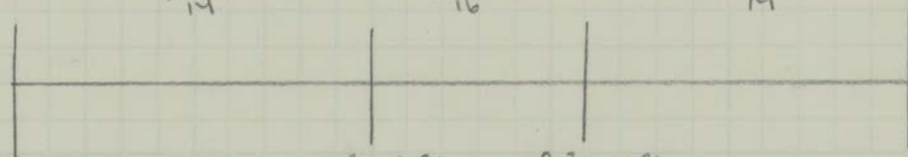
Appendix E

Member Calculations

Continuous Beam Check: Partial Calculation

CHECKING CONTINUOUS BM

$\frac{w l_n^2}{14}$ $\qquad$ $\frac{w l_n^2}{16}$ $\qquad$ $\frac{w l_n^2}{14}$



$\frac{-w l_n^2}{16}$ $\qquad$ $-\frac{w l_n^2}{10}$ $-\frac{w l_n^2}{11}$ $\qquad$ $-\frac{w l_n^2}{11}$ $-\frac{w l_n^2}{10}$ $\qquad$ $\frac{-w l_n^2}{16}$

$l_n = 40' - \frac{24''}{12} = 38'$ $\qquad$ $l_n = 25.5 - \frac{24''}{12} = 23.5'$ $\qquad$ $l_n = 38'$

DEAD

$-\frac{(1.75\ k/ft + .85\ k/ft)(38')^2}{16} = -134.2$ k-ft

$\frac{(2.6)(38)^2}{14} = 268.2$ k-ft

$-\frac{(2.6)\left(\frac{38' + 23.5}{2}\right)^2}{10} = -245.8$ k-ft

$-\frac{w l_n^2}{11} = -223.5$ k-ft

$\frac{(2.6)(23.5)^2}{16} = 89.7$ k-ft

LIVE

$-\frac{(1.26)(38')^2}{16} = 113.7$ k-ft

$\frac{(1.26)(38)^2}{14} = 130$ k-ft

$-\frac{(1.26)(30.75)^2}{10} = -119$ k-ft

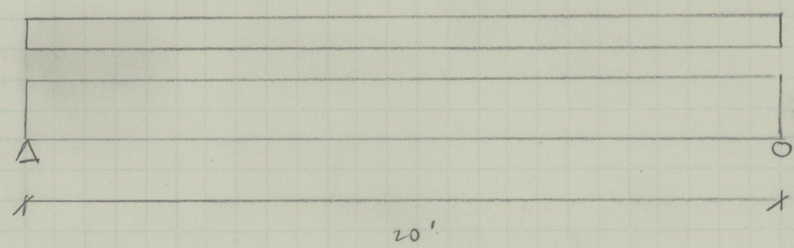
$-\frac{w l_n^2}{11} = 108.3$ k-ft

$\frac{(1.26)(23.5)^2}{16} = 43.5$ k-ft

Exterior Frame Drop Panel Check: Partial Calculation

CHECKING DROP PANEL PLUS SLAB : 36" x 12.5"

4 #8 ALONG BOTTOM



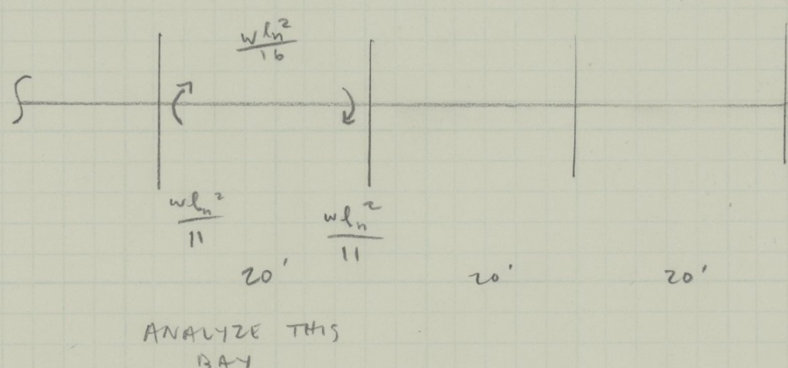
BM SW $= (150 \text{ pcf})(36''/12)(12.5'/12) = .467 \text{ k/ft}$

DL $= (93.75 \text{ lb/ft}^2)(6') + (8 \text{ lb/ft}^2)(6')$

$= .563 + .048 = .611 \text{ k/ft}$

TOTAL $D_L = 1.08 \text{ k/ft}$

MOMENT_{DL}



$l_n = 20 - \frac{24''}{12} = 18'$

NEGATIVE ENDS $-\frac{(1.08 \text{ k/ft})(18')^2}{11} = -31.8 \text{ k-ft}$

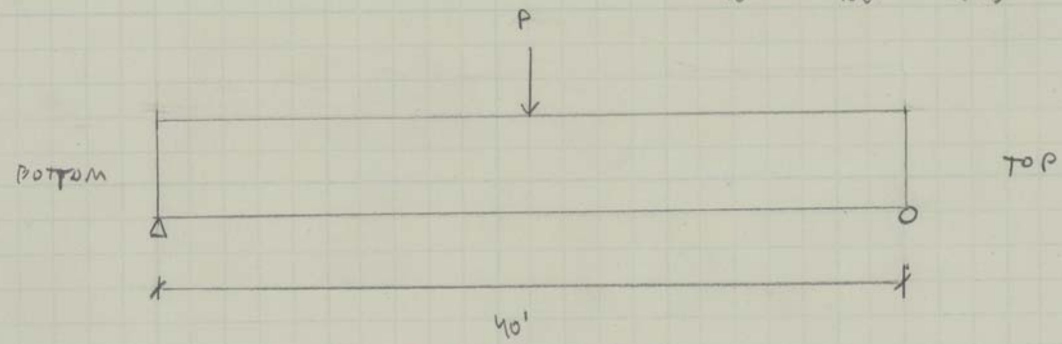
POSITIVE MOMENT $\frac{(1.08)(18')^2}{16} = 21.9 \text{ k-ft}$

60″ x 24″ Beam on Frame F: Partial Calculation

BEAM IN NORTH-SOUTH DIRECTION

60"x24" BM

TOP	TOP	BOT
14 #5	14 #5	14 #7
LE	RE	MID



P

$DL = (150)(7/12)(20)\left(\frac{40}{2} + \frac{25.5}{2}\right) + (15)(20)(32.75) = 57.3 + 9.8 =$

$SW = (150)(48''/12)(14''/12)(32.75') = .65$

$LL = (1.25)(32.75) = 40.9$

$P_D = 67.8\ k$ $\qquad P_L = 40.9\ k$

$w_D = 1.7\ k/ft$ $\qquad w_L = 1.02$

DEAD $\qquad\qquad$ LIVE

TOP $\quad -\frac{(1.7 + \cancel{.65})\left(40 - \frac{36}{12}\right)^2}{16} = -145.5\ k\text{-}ft$ $\qquad -\frac{(1.02)(37)^2}{16} = 87.3\ k\text{-}ft$

BOTTOM $\quad -\frac{(1.7 + \cancel{.65})\left(\frac{37+18}{2}\right)^2}{10} = -128.6\ k\text{-}ft$ $\qquad -\frac{(1.02)(27.5)^2}{10} = 77.1\ k\text{-}ft$