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Hyatt Regency – Hotel and Conference Center
Pittsburgh International Airport, PA
November 21, 2005
Technical Assignment 3

EXECUTIVE SUMMARY

The Hyatt Regency – Hotel and Conference Center at the Pittsburgh International Airport, PA, is a 275,000 square foot multi-use building located directly adjacent to the airport's landside terminal. The building consists of an 11-story tower and 1-story conference center with an additional partial level below grade.

The tower is a concrete structure with typical 22"x28" or 22x32" columns and an 8" filigree floor and roof system. The lateral resisting system in the tower consists of concrete moment frames. The conference center is a steel framed structure, with typical W10x33 columns, and different beam sizes, ranging from W12x19 to W21x44 beams. The conference center has composite steel decking with a concrete slab and steel roof decking. The lateral resisting system in the conference center are four braced frames (2 in each direction), each consisting of two K-braces.

The tower and the conference center are independent structures, and are quite different structurally. Lateral loads have been calculated based on the ASCE 7-02. The original design for the buildings did not incorporate seismic loadings, so part of this analysis is to determine the impact of the new loading case.

Through computer modeling and hand calculations, the structures have been analyzed for the design loading combinations. The controlling load combination was determined to be: 1.2 D + 0.2 S + E + L. Under this loading, the deflection at the top of the tower was determined to be 20.7 inches, which is within this case's seismic drift limit of 26.4 inches. The seismic loading does not meet the l/400 standard for drift; however, under the controlling wind loading of: 1.2 D + 0.5 S + 1.6 W + L, the drift at the roof is limited to 3.2 inches, which is less than the l/400 value of 3.67 inches. The drift in the conference center is only 0.07 inches, so it is negligible.

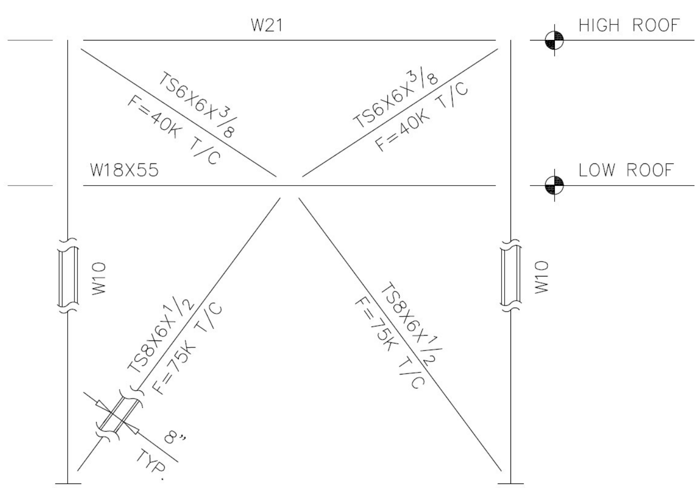
The analysis has shown that although almost all of the members are the adequate size for the loads they are required to take, many of the concrete columns in the tower require additional reinforcing steel to carry the critical load case. The only columns that were not satisfactory under the critical loading are the slender columns around the stair towers, although they are satisfactory under the critical wind loading. Overall, with a new controlling seismic load case that is much larger than the controlling wind case, the building still performs as it was designed. Most of the members simply need additional reinforcement, and do not even require re-sizing.

DESCRIPTION OF LATERAL SYSTEMS

The Hyatt combines two separate building systems: a concrete tower and a steel conference center. The two systems are structurally independent and have different systems to resist lateral forces: concrete moment frames for the tower and steel braced frames for the conference center.

Conference Center

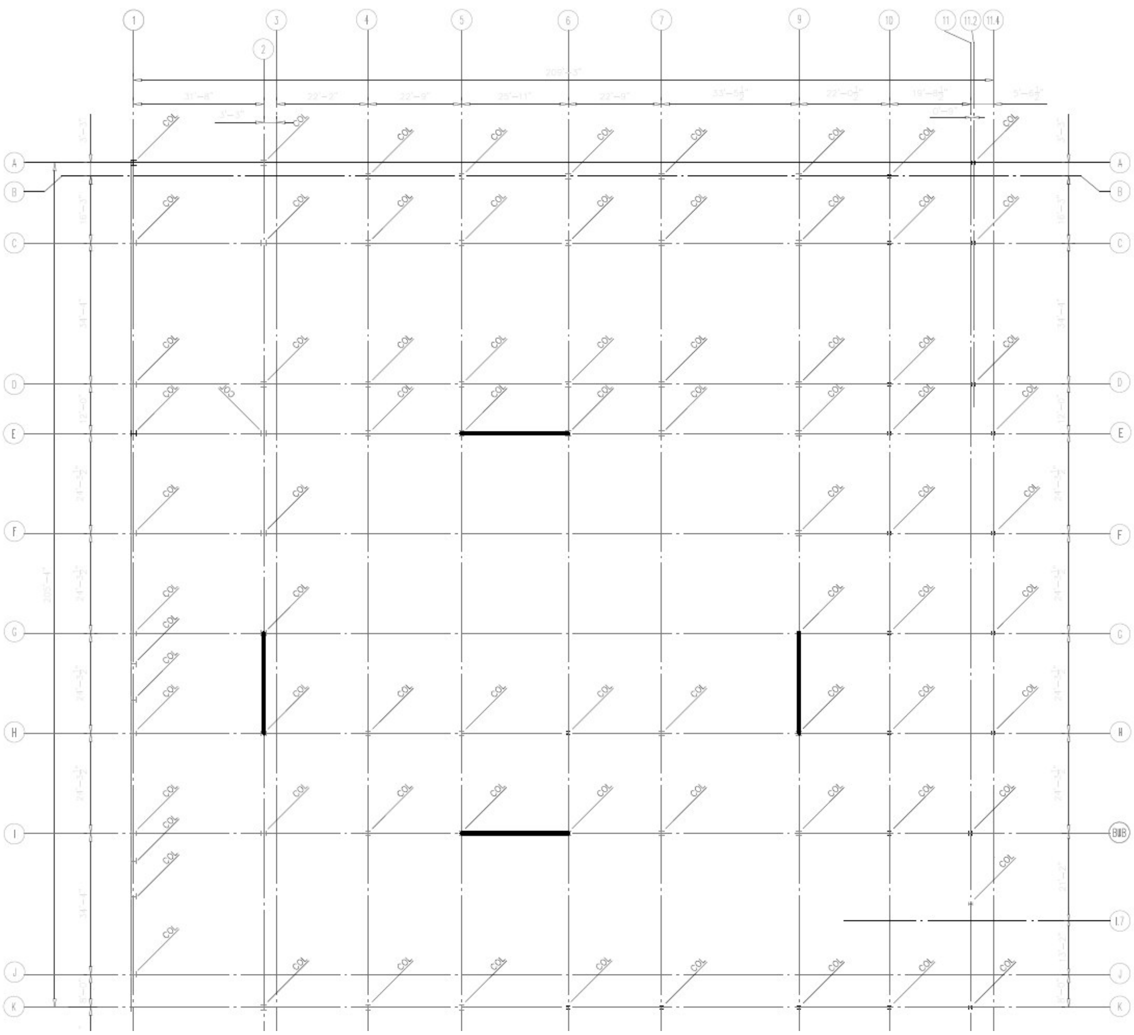
The conference center is framed with steel and has steel braced frames from the ground level to the high-roof. HSS 8X6X½ members brace from the ground level to the low-roof level and HSS 6X6X$^3/_8$ members brace from the low-roof level to the high-roof. The system has 2 braced frames in each direction to resist lateral loads. The braces used are K-braces, which requires less material than X-braces on each floor. The members are designed to take both compressive and tensile loads, so all bracing members participate in resisting lateral forces, regardless of the in-plane direction of the loading.



Elevation view of K-braces

The braces are all the same, so therefore they have the same stiffness each. The center of rigidity is located at the point between each of the braced frames, since they all have equal stiffness values. Their orientation places the center of rigidity close to the center of

the building. (See figure below) This minimizes torsion effects on the building from eccentricities in the loading. The lateral wind loads are transferred through the exterior cladding to the rigid floor and roof diaphragms which distribute the loads between the braced frames.



Plan of conference center with lateral brace locations shown (darkened lines)

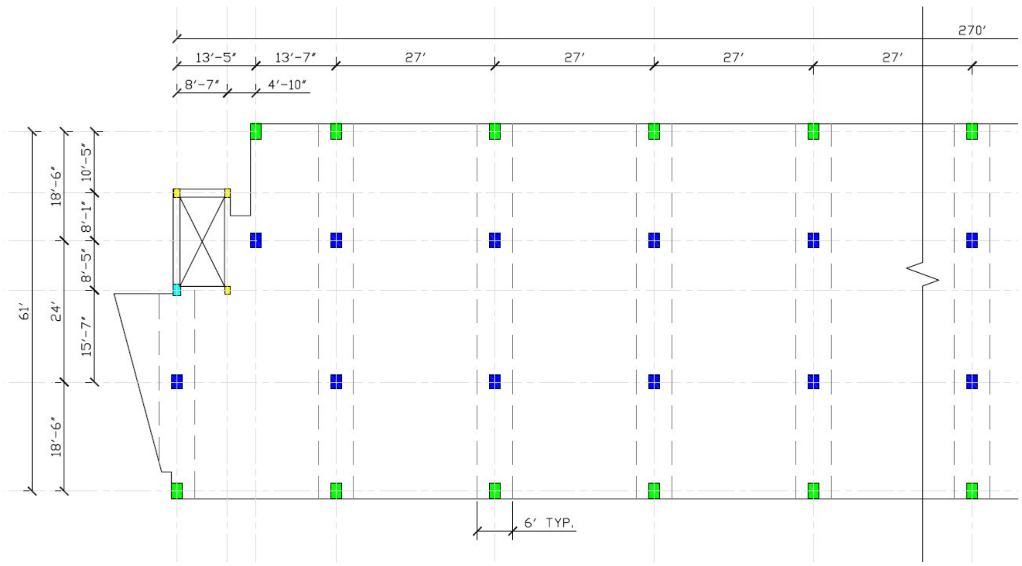
Tower (Guest Rooms)

The tower is framed with concrete columns and a filigree slab system. The primary structural columns are 22”x32” for exterior columns and 22”x28” for interior columns. The slab acts as a rigid diaphragm to distribute the lateral loads to the columns. 6’ column strips in the slab do not have voids and act as shallow beams spanning between the columns. See figures below of tower framing plans for column spacing and 6’ column strip orientation. All tower columns use 5000 psi concrete and the floor system is 4500 psi concrete. The columns are oriented with their strong axis in the North-South direction, which will experience the greatest wind forces.

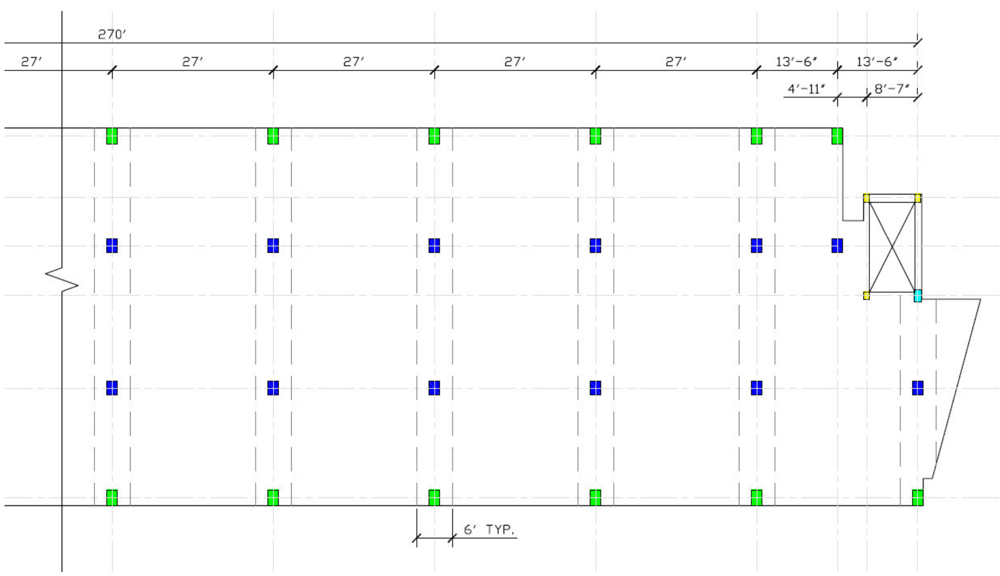
The shape of the tower is very close to rectangular. The only deviations from a perfect rectangular shape are triangular sections in the suites at the East and West end of each floor and the location of the emergency stair towers at each end. This results in the center of mass and rigidity being very close to the exact center of the tower (due to tower symmetry). The only area of the tower that slightly alters the centers of mass and rigidity are the elevator shafts; the openings in the slab change the center of mass and the diaphragm center of rigidity. Although this does change the results slightly, it does not greatly impact the lateral systems of the building.

The wind loads on the tower are distributed to the slab through the exterior pre-cast wall panels. The slab then acts as a rigid diaphragm, transferring the load to the frames based on their relative stiffness. In N-S direction of the tower, almost all the frames are identical (with the exception of the East and West end frames, based on configuration of the stair towers) which means that all the frames’ stiffness will be equal and the distribution of load will also be approximately equal. In the E-W direction, the exterior frames (on the North and South faces of the building) are composed of larger columns than the interior frames. This leads to the exterior frames having a larger stiffness and therefore more of the lateral load transferred to those frames. Once the lateral loads are in the columns, the loads are transferred down through the column to the foundation of steel piles which are driven approximately 60’ below finished grade.

The figures below show the column layout on the typical tower floor plan and the moment frames with column size and orientation detailed.

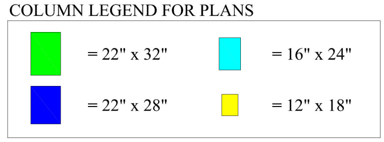


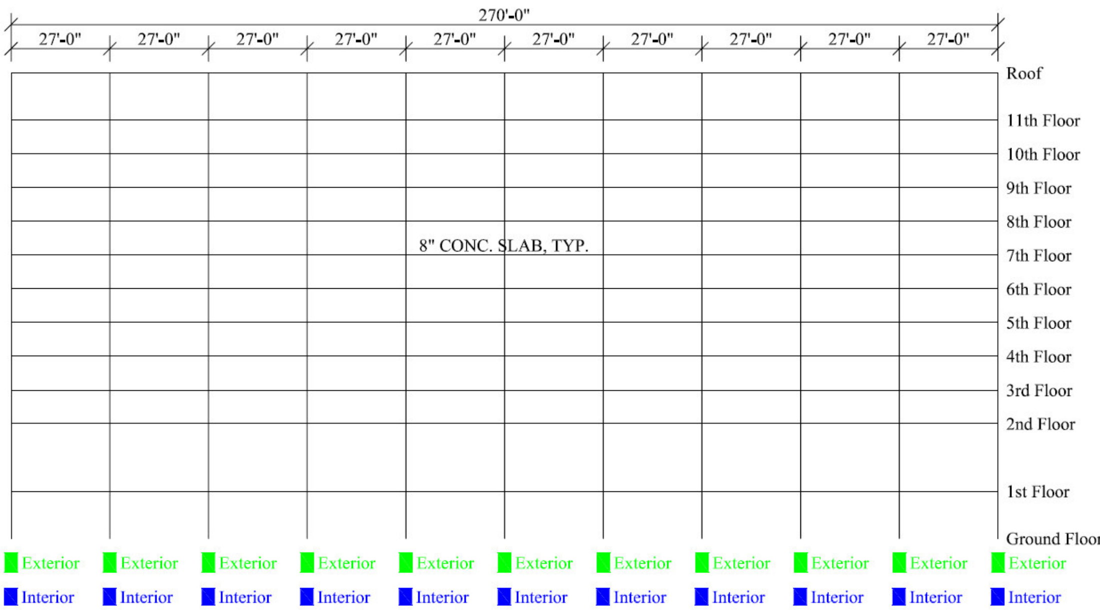
TOWER FRAMING PLAN – WEST END



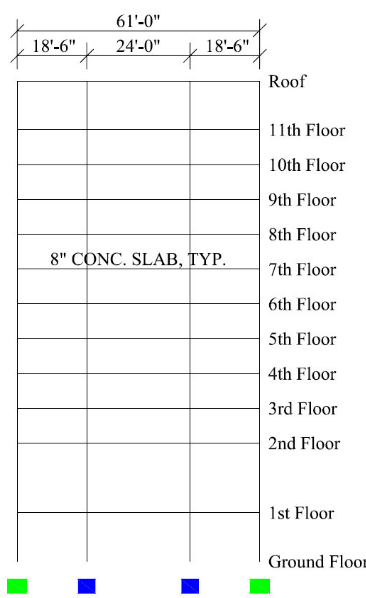
TOWER FRAMING PLAN – EAST END

COLUMN LEGEND FOR PLANS





Typical frame resisting lateral loads in E/W direction. (See column legend for sizes)



Typical frame resisting lateral loads in N/S direction. (See column legend for sizes)

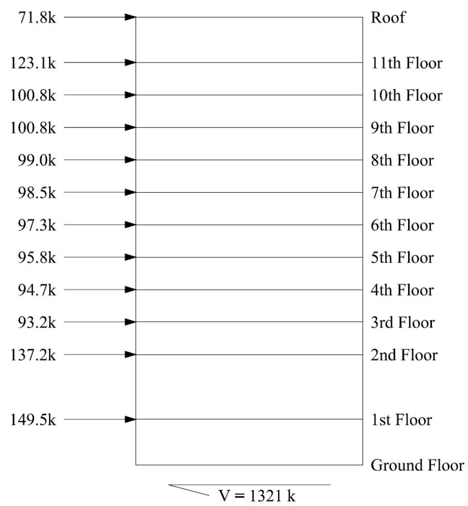
WIND LOADS

The design wind loads have been determined in accordance to IBC 2003 and ASCE 7-02. Wind loads have been calculated based on the 11-story, 140-foot tower of the building. The main building factors for determining the wind loads are the basic wind speed of 90mph, exposure C, importance category II. The calculations assume that the building behaves as a rigid, rectangular structure. There is some variation between the calculated loads and those in the design documents; however, this is most likely due to code changes, and the values are not significantly different. Since the wind loading on the tower is more critical than that of the low-rise conference center, the conference center loads will assume the same loading for the equivalent height of the tower calculations, using the loading for the N/S direction, as the conference center is a roughly square building of approximately the same length. Wind loading calculations are detailed in Appendix A.

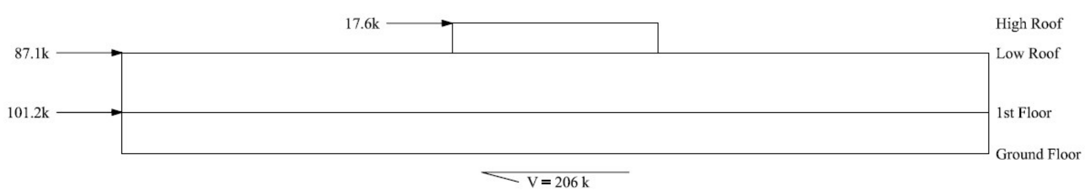
STORY SHEARS

Story shears have been determined from the tributary area to each story. Refer to Appendix A for area and shear calculations.

E/W Tower Story Shears



N/S Tower Story Shears



N/S & E/W Conference Center Story Shears

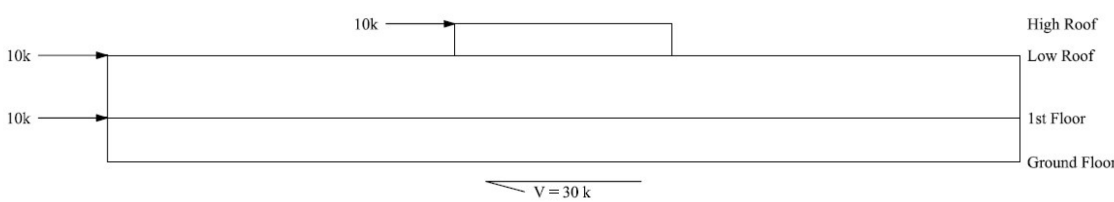
SEISMIC LOADS

Seismic calculations have been calculated using IBC 2003 and ASCE 7-02. The loads for the tower and conference center have been calculated separately, as the weight of the structure is a factor in the forces. Based on this, the tower's weight will greatly increase the lateral loads on the building as compared to the weight of the conference center.

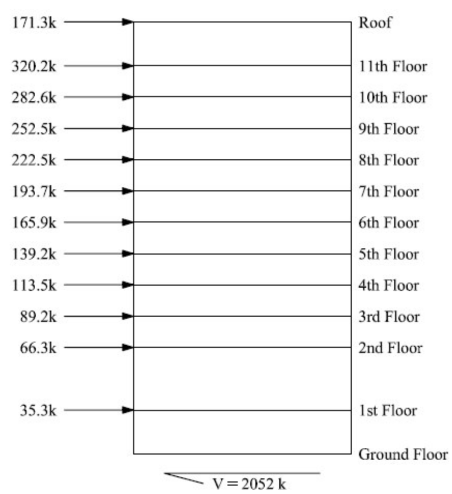
The original design of the building did not include seismic requirements, so these loadings were most likely not considered during the design of the concrete moment framing or steel braced frames that serve as the lateral resisting system for the buildings. The calculations were made using the Equivalent Lateral Force Procedure. The building weights were approximated for the calculations based on a typical tower floor plan for the tower and a RAM model summary for the conference center; though value may vary slightly from the actual weight, it should not change the loading significantly.

Based on the original geotechnical report, no shear wave velocities for the soil was determined. Therefore, based on the Plasticity Index, site class D is used. It can be noted that with further site investigation, it is likely that an accurate site class designation can be determined, which will greatly decrease the seismic effects on the building.

Seismic loads are the same from each direction. Detailed calculations can be found in Appendix B.



E/W and N/S Story Shears from Seismic Loading on Conference Center



E/W (Above) and N/S (Below) Story Shears from Seismic Loading on Tower

DESIGN CHECKS

Tower Design Checks

The lateral loads on the tower are much greater than those of the conference center due to its height for wind loads as well as its much greater weight for seismic loads. For the analysis of the tower, ETABS was used to create a 3-D model and analyze the load combinations from ASCE 7-02. ETABS has provisions in the program for ACI 318-99 and ASCE 7-99, so the analysis and design parameters were updated to match the current ACI 318-02 and ASCE 7-02 design codes. The model created used the design loads determined previously in Technical Assignment 1 with user-defined lateral load cases instead of loadings generated by the program.

Based on the analysis of all of the ASCE 7-02 load combinations, the critical design loading case is: 1.2 D + 0.2 S + E + L. This load case was then checked for maximum drift. A point in the North-West end of the tower was selected to analyze story drift criteria. The drifts under the loading are tabulated below. The seismic drift limit at each story is defined to be 0.02*(height of the story below the level being analyzed) from ASCE 7-02, for buildings larger than 4 stories that aren't masonry, and with Seismic Use Group I. When the actual drifts are compared to the drift limit, the drift under the critical loading is allowed. NOTE: It can also be noted that the drift under the seismic controlled loading does not meet the l/400 design criteria; however, under the controlling wind loading, 1.2 D + 0.5 S + 1.6 W + L, the drift at the roof is limited to 3.2 inches, which is less than the l/400 value of 3.67 inches.

Story Number	Story Height (in.)	Story Drift in E/W Direction (in.)	Story Drift in N/S Direction (in.)	Drift Limit (in.)
Roof	1468	20.7	5.0	26.4
11	1320	19.4	4.7	24.0
10	1200	18.2	4.4	21.6
9	1080	16.7	4.0	19.2
8	960	15.1	3.6	16.8
7	840	13.2	3.1	14.4
6	720	11.2	2.5	12.0
5	600	9.0	1.9	9.6
4	480	6.8	1.3	7.2
3	360	4.6	0.7	4.8
2	240	2.5	0.4	4.8
Ground Level	0	0.0	0.0	n/a

An axial/bending interaction strength check in ETABS shows that most all of the columns are within the interaction envelope. The columns around the stair towers at each end of the building are shown to be outside of the acceptable envelope. If the columns are analyzed without taking seismic loading into effect, they are within the acceptable interaction envelope. The columns were not designed for seismic loading, so it is not surprising that they are under the required strength for the critical loading that includes seismic forces. A typical exterior column was also checked for strength, based on the critical loading case (see Appendix C). For that loading, the area of steel required is 40.4 in^2, which is much greater than the 25 in^2 provided by the (12)-#9 and (4)-#14 bars in the original design. With the increased area of steel, the column meets strength requirements; this is another case where the addition of seismic loads to the design requires changes to the structural systems.

The overturning moment was checked in the short direction of the tower, see Appendix C. An overturning moment was determined to be 163,894 ft-k which is restrained by the resisting moment of 1,390,800 ft-k, generated by the tower weight. The weight of the building can easily resist any overturning issues, so overturning will not be a concern.

Each column transfers 250-300 kips axial load and 1000-1150 ft-kips bending moment to the piles that form the foundation system for the tower. As the foundation consists of piles driven approximately 60' deep, the axial loads and moments will not have much impact on the system, since it is very rigid and had virtually no chance of rotating or settling under those conditions.

Torsion is negligible in the tower of the building. The shape of the building is very nearly rectangular, with typical framing both directions. Since there are very few irregularities in the diaphragm and framing, the torsion forces are very small.

Conference Center Design Checks

More simplified design checks have been performed on the conference center than the tower, since the lateral loads have much less impact on the low-rise section of the Hyatt. The conference center is nearly square and also has its center of rigidity very near the center of the building. The 4 identical braced frames that make up the lateral system are located around the high-roof portion of the building and extend down through the low-roof to the ground level. Since the center of rigidity in the high-roof section is at the center of mass and the center of rigidity of the low-roof section is close to centered in the

building, the loading has been simplified to assume symmetry in the loads on the braced frames.

The virtual work calculations have distributed the story shears from the wind loading (which controls the conference center due to its reduced weight) equally between the two braced frames in each direction (see Appendix C). From the analysis, the story drift at the top of the high-roof section is only 0.07 inches, which is basically negligible. The l/400 drift criterion is: 0.855 inches, so the criteria are met by the 0.07 inch drift. It is obviously within the drift limits that were determined. In the calculations, the diagonal bracing members were allowed to take both tensile and compressive forces, as in the original design.

The strength of the bracing members has been checked to ensure that they are not taking too much tensile or compressive force. From the analysis, under critical loadings, the maximum tensile/compressive forces are approximately 50 kips for the lower level braces and 6 kips for the upper level braces, which is much lower than the design allowable forces of 75 kips and 40 kips, respectively.

In the conference center, torsion is a little more of a concern than in the tower, still it is very minimal, as the braced frames are typical in both directions and there are very few irregularities in the building shape. The fact that the center of rigidity is slightly away from the center of mass will add some minimal torsion forces; however, since the lateral forces are very small and the eccentricities are very small, it is not as much of a concern as it would be if the same eccentricities were in the tower.

SUMMARY AND CONCLUSIONS

From performing thorough analyses on the structure, it has been determined that the lateral force resisting systems in the building are adequate for the loadings that have been calculated. Since the original design did not include seismic forces, which ended up controlling the structure, some of the members do require more reinforcement steel than the original design, but they will still perform as intended even with the increased loading from seismic forces. Under standard wind loading, the drift criteria of l/400 is met, however, when the controlling seismic loading is considered, the l/400 criteria is not met, but the seismic drift criteria is. The lateral forces on the conference center are very minimal and the 4 braced frames that are used to resist those forces are adequate under the loadings and are sufficient to limit drift to l/400.

APPENDIX

APPENDIX A

<u>Wind Loading Calculations:</u>
(Using ASCE 7-02 Method 2 – Analytical Procedure)

$V = 90$ mph
Exposure C
$I = 1.0$
$K_{zt} = 1.0$ (no topographic features)
$K_d = 0.85$ (main lateral system)
$G = 0.85$ (for rigid structures - assumed)
$GC_{pi} = \pm 0.18$ (for enclosed buildings)

Velocity Pressure, q_z

z (ft)	K_z	$q_z = 0.00256\ K_z\ K_{zt}\ K_d\ V^2\ I$ *(lb/ft²)*
15	0.85	15.0
20	0.90	15.9
25	0.94	16.6
30	0.98	17.3
40	1.04	18.3
50	1.09	19.2
60	1.13	19.9
70	1.17	20.6
80	1.21	21.3
90	1.24	21.9
100	1.26	22.2
120	1.31	23.1
140	1.36	24.0

$q_h = 24.0$ lb/ft^2

Wall Pressure Coefficients, C_p

Surface	*Direction*	*L (ft)*	*B (ft)*	*L/B*	***C_p***
LEEWARD	N/S	65	273	0.2	-0.5
	E/W	273	65	4.2	-0.2
WINDWARD	N/S, E/W	All Values			0.8

WIND PRESSURE CALCULATIONS:

$p = qGC_p - q_i(GC_{pi})$ *(lb/ft²)*

WINDWARD WIND PRESSURES:

$p_{0\text{-}15}$ = 14.5 psf
p_{20} = 15.1 psf
p_{25} = 15.6 psf
p_{30} = 16.1 psf
p_{40} = 16.8 psf
p_{50} = 17.4 psf
p_{60} = 17.9 psf
p_{70} = 18.3 psf
p_{80} = 18.8 psf
p_{90} = 19.2 psf
p_{100} = 19.4 psf
p_{120} = 20.0 psf
p_{140} = 20.6 psf

LEEWARD WIND PRESSURES:

$p_{N/S}$ = -14.5 psf
$p_{E/W}$ = -8.4 psf

STORY SHEARS AND BASE SHEAR CALCULATIONS:

East-West Tower Story Shears

Story	*Actual Elevation (ft)*	*Adjusted Elevation (ft)*	*Lower 'h' (ft)*	*Upper 'h' (ft)*	*Tributary Height (ft)*	*Tributary Width (ft)*	*p_W (psf)*	*p_L (psf)*	*Story Shear (k)*
Ground	1117	0	0	7	7	73	14.5	8.4	11.7
1	1131	14	7	24	17	73	15.6	8.4	29.8
2	1151	34	24	39	15	73	16.8	8.4	27.6
3	1161	44	39	49	10	73	17.4	8.4	18.8
4	1171	54	49	59	10	73	17.9	8.4	19.2
5	1181	64	59	69	10	73	18.3	8.4	19.5
6	1191	74	69	79	10	73	18.8	8.4	19.9
7	1201	84	79	89	10	73	19.2	8.4	20.1
8	1211	94	89	99	10	73	19.4	8.4	20.3
9	1221	104	99	109	10	73	20.0	8.4	20.7
10	1231	114	109	119	10	73	20.0	8.4	20.7
11	1241	124	119	131	12	73	20.6	8.4	25.4
Roof	1255	138	131	138	7	73	20.6	8.4	14.8

E-W Base Shear = 287 kips

North-South Tower Story Shears

Story	*Actual Elevation (ft)*	*Adjusted Elevation (ft)*	*Lower 'h' (ft)*	*Upper 'h' (ft)*	*Tributary Height (ft)*	*Tributary Width (ft)*	*p_W (psf)*	*p_L (psf)*	*Story Shear (k)*
Ground	1117	0	0	7	7	292	14.5	14.52	59.3
1	1131	14	7	24	17	292	15.6	14.52	149.5
2	1151	34	24	39	15	292	16.8	14.52	137.2
3	1161	44	39	49	10	292	17.4	14.52	93.2
4	1171	54	49	59	10	292	17.9	14.52	94.7
5	1181	64	59	69	10	292	18.3	14.52	95.8
6	1191	74	69	79	10	292	18.8	14.52	97.3
7	1201	84	79	89	10	292	19.2	14.52	98.5
8	1211	94	89	99	10	292	19.4	14.52	99.0
9	1221	104	99	109	10	292	20.0	14.52	100.8
10	1231	114	109	119	10	292	20.0	14.52	100.8
11	1241	124	119	131	12	292	20.6	14.52	123.1
Roof	1255	138	131	138	7	292	20.6	14.52	71.8

N-S Base Shear = 1321 kips

East-West Conference Center Story Shears

Story	*Actual Elevation (ft)*	*Adjusted Elevation (ft)*	*Lower 'h' (ft)*	*Upper 'h' (ft)*	*Tributary Height (ft)*	*Tributary Width (ft)*	*p_W (psf)*	*p_L (psf)*	*Story Shear (k)*
Ground	1117	0	0	7	n/a	n/a	n/a	n/a	n/a
1	1131	14	7	23	16	210	15.6	14.52	101.2
Low Roof	1149	32	23	36.25	13.25	210	16.8	14.52	87.1
High Roof	1157.5	40.5	36.25	40.5	4.25	130	17.4	14.52	17.6

E-W Base Shear = 206 kips

North-South Conference Center Story Shears

Story	*Actual Elevation (ft)*	*Adjusted Elevation (ft)*	*Lower 'h' (ft)*	*Upper 'h' (ft)*	*Tributary Height (ft)*	*Tributary Width (ft)*	*p_W (psf)*	*p_L (psf)*	*Story Shear (k)*
Ground	1117	0	0	7	n/a	n/a	n/a	n/a	n/a
1	1131	14	7	23	16	210	15.6	14.52	101.2
Low Roof	1149	32	23	36.25	13.25	210	16.8	14.52	87.1
High Roof	1157.5	40.5	36.25	40.5	4.25	130	17.4	14.52	17.6

N-S Base Shear = 206 kips

APPENDIX B

<u>Seismic Loading Calculations – Tower:</u>
(using ASCE 7-02 Equivalent Lateral Force System)

For Pittsburgh, PA

$S_s = 0.127g$
$S_1 = 0.054g$

Occupancy II
Seismic Use Group I
$I_E = 1.0$

Site Class: D *(without sufficient detail to determine a Site Class, class D shall be used. As found in the geotechnical report prepared by L. Robert Kimball & Associates, the samples have a plasticity index (PI) ranging from 8-20. Site Class E is not used since the PI indicates that it is not a soft clay (PI>20)*

Based on site class, S_s, and S_1,

$F_a = 1.6$ $F_v = 2.4$

$S_{DS} = {}^2/_3 S_{MS} = {}^2/_3 F_a Ss = {}^2/_3(1.6)(0.127) = 0.135$
$S_{D1} = {}^2/_3 S_{M1} = {}^2/_3 F_v S_1 = {}^2/_3(2.4)(0.054) = 0.086$

S_{DS}, S_{D1}, and Seismic Use Group I, yields:

Seismic Design Category B

Based on this Seismic Design Category, the Equivalent Lateral Force System is permissible.

Base Shear Calculation:

$V_{BASE} = C_S W$

$C_S = S_{DS}/(R/I_E) \geq \mathit{0.044S_{DS}I_E}$

R = 3.0 (for ordinary reinforced concrete moment frames)

$C_S = 0.135/(3.0/1.0) \geq \mathit{0.044(0.135)(1.0)}$
$= 0.045 \geq \mathit{0.006}$ *(OK)*

W (Total weight is calculated from the typical floor plan for the tower. It is assumed for simplification that each floor has the same total weight, although some minor differences will occur.)

Weight of Each Floor

Columns							
	L	*W*	*H*	*lb/ft3*	*#/floor*		*Wt.*
	22"	32"	10'	150 pcf	44/floor	=	**322.7k**
	12"	18"	10'	150 pcf	8/floor	=	**13.5k**
Col. Strip							
	61'	72"	8"	150 pcf	11/floor	=	**402.6k**
Slab							
	t	*SF*		*lb/ft^3*			
	8"	17000 sf		150 pcf		=	**1700k**
Dead Load							
		SF		*lb/ft^2*			
		17000 sf		80		=	**1360k**

$W_{TOT} = 12 * \Sigma W_{FLOOR} = 12 * (3800k) = 45600$ k

$V_{BASE} = 0.045(45600k) = 2052$ k

Distribution to Floors:

The building does not exceed 12 stories, the lateral resisting system is entirely concrete, and the story height is at least 10ft, therefore the following assumption is valid:

$T_a = 0.1N = 0.1(11) = 1.1$ sec

$k = 1.3$ *(linear interpolation between 1 and 2 for a value of $T_a = 1.1$ sec)*

$F_x = C_{vx}V$ *(force at story x)*

$C_{vx} = w_x h_x^k / (\Sigma w_i h_i^k)$

Story Forces

Story	w_x	h_x	$w_x h_x^k$	C_{vx}	Story Force
1	3800	24	236624	0.017	35.3
2	3800	39	444803	0.032	66.3
3	3800	49	598465	0.043	89.2
4	3800	59	761888	0.055	113.5
5	3800	69	933872	0.068	139.2
6	3800	79	1113522	0.081	165.9
7	3800	89	1300141	0.094	193.7
8	3800	99	1493170	0.108	222.5
9	3800	109	1692147	0.123	252.2
10	3800	119	1896683	0.138	282.6
11	3800	131	2148999	0.156	320.2
Roof	1900	138	1149731	0.083	171.3
		Σ	13770045	1	2052

Seismic Loading Calculations – Conference Center:
(using ASCE 7-02 Equivalent Lateral Force System)

For Pittsburgh, PA

$S_s = 0.127g$
$S_1 = 0.054g$

Occupancy III
Seismic Use Group II
$I_E = 1.25$

Site Class: D *(without sufficient detail to determine a Site Class, class D shall be used. As found in the geotechnical report prepared by L. Robert Kimball & Associates, the samples have a plasticity index (PI) ranging from 8-20. Site Class E is not used since the PI indicates that it is not a soft clay (PI>20)*

Based on site class, S_s, and S_1,

$F_a = 1.6$ $F_v = 2.4$

$S_{DS} = {}^2/_3 S_{MS} = {}^2/_3 F_a Ss = {}^2/_3(1.6)(0.127) = 0.135$
$S_{D1} = {}^2/_3 S_{M1} = {}^2/_3 F_v S_1 = {}^2/_3(2.4)(0.054) = 0.086$

S_{DS}, S_{D1}, and Seismic Use Group II, yields:

Seismic Design Category B

Based on this Seismic Design Category, the Equivalent Lateral Force System is permissible.

Base Shear Calculation:

$V_{BASE} = C_S W$

$C_S = S_{DS}/(R/I_E) \geq \mathit{0.044S_{DS}I_E}$

R = 5.0 (for ordinary steel concentrically braced frames)

$C_S = 0.135/(5.0/1.25) \geq \mathit{0.044(0.135)(1.25)}$
$= 0.034 \geq \mathit{0.007\ \ (OK)}$

W = 160,000 (Beams) + 31,000 (Joists) + 71,000 (Columns) + 115,000 (Floors) + 40,000 (Walls and glass) Weights taken from RAM model and approximated.

W = 417 kips

$V_{BASE} = 0.024(417k) = 10$ k

Frame Forces:

The lateral resisting frame will have 10k applied to it at each level. This is very conservative, as the total base shear is only 10k; however, since the wind loading is much greater than 10k at each floor, the simplification allows a simplified approach that does not require the calculation of the period of the conference center, and the distribution of the load between only 3 floor levels.

APPENDIX C

Tower Checks

A typical exterior column was checked under the critical loading case in ETABS. The design parameters have been updated for ACI 318-02.

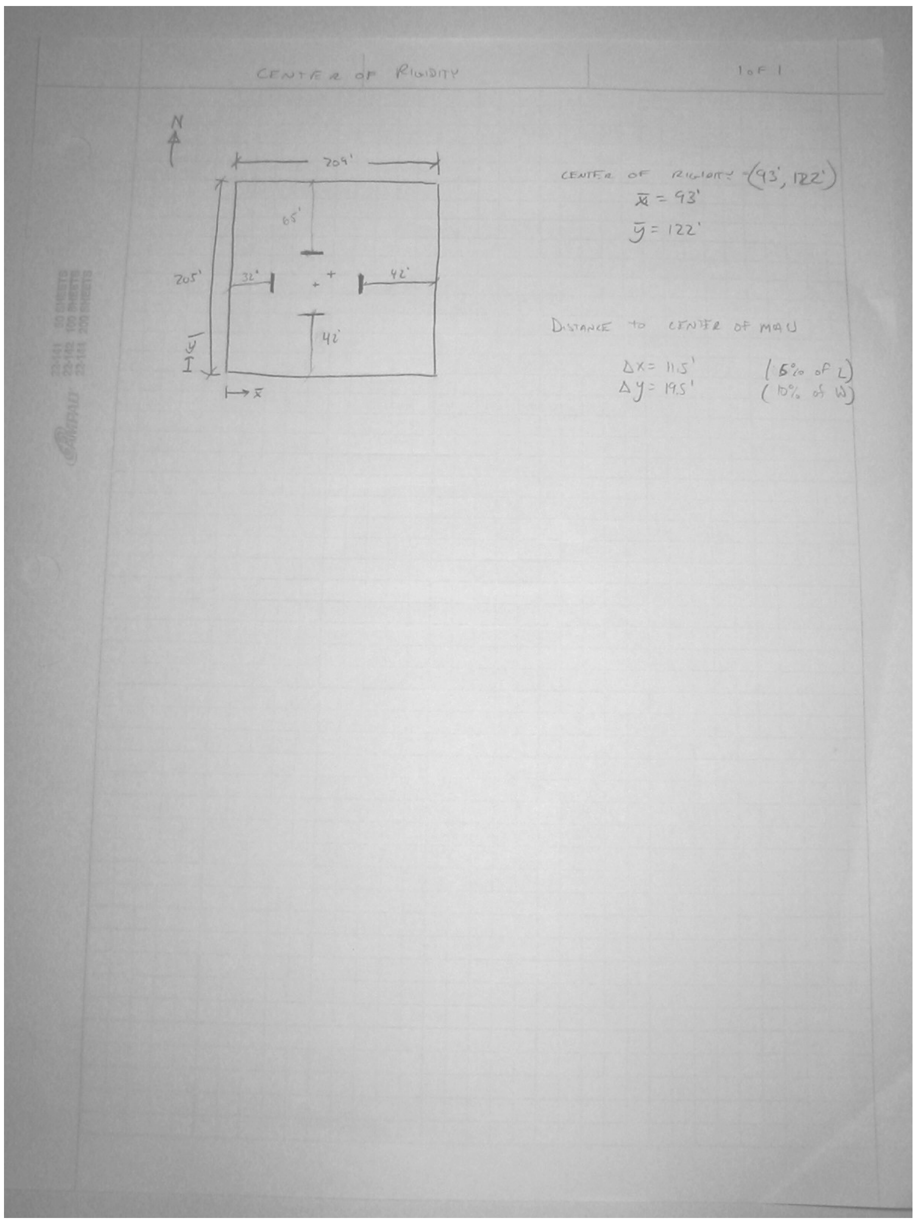
```
ACI 318-99 COLUMN SECTION DESIGN  Type: Sway Special  Units: Kip-in  (Summary)

Level       : 2                   L=240.000
Element     : C46                 B=32.000         D=22.000         dc=1.800
Station Loc : 0.000               E=3960.000       fc=5.000         Lt.Wt. Fac.=1.000
Section ID  : 22X32C5KSI          fy=60.000        fys=60.000
Combo ID    : ASCE5X              RLLF=1.000

Phi(Compression-Spiral): 0.700    Overstrength Factor: 1.25
Phi(Compression-Tied):   0.650
Phi(Tension):            0.900
Phi(Bending):            0.900
Phi(Shear/Torsion):      0.750


AXIAL FORCE & BIAXIAL MOMENT DESIGN FOR PU, M2, M3
               Rebar      Design      Design      Design     Minimum     Minimum
                Area          Pu          M2          M3          M2          M3
              40.365     286.843    -447.475   13783.909     447.475     361.422

AXIAL FORCE & BIAXIAL MOMENT FACTORS
                              Cm    Delta_ns     Delta_s           K           L
                          Factor      Factor      Factor      Factor      Length
   Major Bending(M3)       0.684       1.000       1.000       1.000     222.000
   Minor Bending(M2)       0.953       1.000       1.000       1.000     222.000

SHEAR DESIGN FOR V2,V3
                          Design       Shear       Shear       Shear       Shear
                           Rebar          Vu      phi*Vc      phi*Vs          Vp
   Major Shear(V2)         0.027      49.101      82.529      24.240       5.416
   Minor Shear(V3)         0.018      47.810      84.827      24.915      47.810

JOINT SHEAR DESIGN
                    Joint Shear        Shear       Shear       Shear       Joint
                           Ratio       VuTop       VuTot      phi*Vc        Area
   Major Shear(V2)           N/A         N/A         N/A         N/A         N/A
   Minor Shear(V3)           N/A         N/A         N/A         N/A         N/A

(6/5) BEAM/COLUMN CAPACITY RATIOS
                           Major       Minor
                           Ratio       Ratio
                             N/A         N/A


Notes:
   N/A: Not Applicable
   N/C: Not Calculated
   N/N: Not Needed
```

To check the overturning moment, the overturning moment M_O from the story shears was compared to the resisting moment M_R from the building weight.

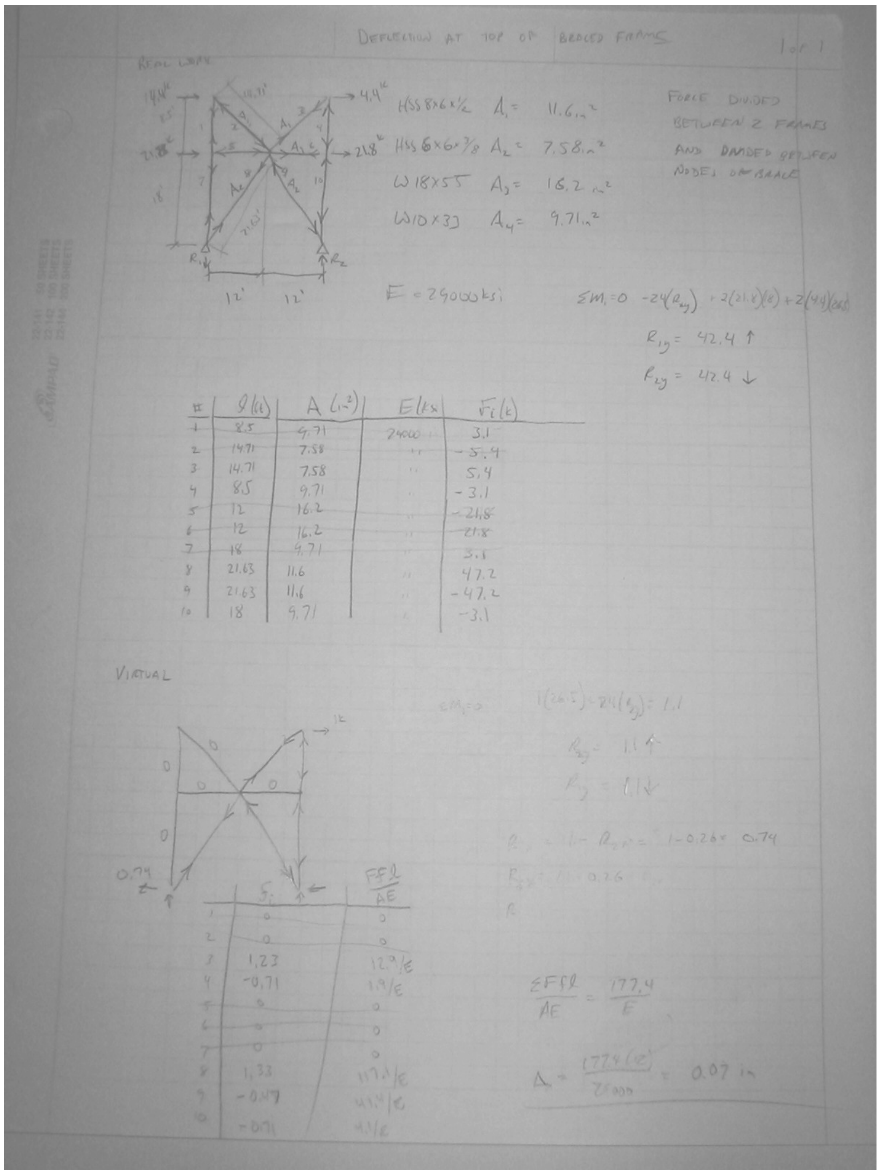
Story Height	Story Shear	Moment
122	171.3	20898.6
110	320.2	35222
100	282.6	28260
90	252.2	22698
80	222.5	17800
70	193.7	13559
60	165.9	9954
50	139.2	6960
40	113.5	4540
30	89.2	2676
20	66.3	1326
	Mo = ΣM =	**163894**

M_R = W*(h/2) = 45,600k*(61'/2) = 1,390,800 ft-k > M_O = 163,894 ft-k ---OK

Hand calculation of center of rigidity and eccentricity from center of mass of conference center.



Virtual work calculations for deflection at the top of one of the 4 braced frames.



DEFLECTION AT TOP OF BRACED FRAMES

1 of 1

REAL WORK

HSS 8x6x1/2 $A_1 = 11.6 in^2$

HSS 6x6x3/8 $A_2 = 7.58 in^2$

W18X55 $A_3 = 16.2 in^2$

W10X33 $A_4 = 9.71 in^2$

FORCE DIVIDED BETWEEN 2 FRAMES AND DIVIDED BETWEEN NODES OF BRACE

$E = 29000 ksi$

$\Sigma M_1 = 0 \quad -24(R_{2y}) + 2(21.8)(18) + 2(4.4)(26.5)$

$R_{1y} = 42.4 \uparrow$

$R_{2y} = 42.4 \downarrow$

#	l (ft)	A (in^2)	E (ksi)	F_i (k)
1	8.5	9.71	29000	3.1
2	14.71	7.58	"	-5.4
3	14.71	7.58	"	5.4
4	8.5	9.71	"	-3.1
5	12	16.2	"	-21.8
6	12	16.2	"	21.8
7	18	9.71	"	3.1
8	21.63	11.6	"	47.2
9	21.63	11.6	"	-47.2
10	18	9.71	"	-3.1

VIRTUAL

$\Sigma M_1 = 0 \quad 1(26.5) - 24(R_{2y}) = 1.1$

$R_{2y} = 1.1 \uparrow$

$R_{1y} = 1.1 \downarrow$

$R_{1x} = 1 - R_{2x} = 1 - 0.26 = 0.74$

$R_{2x} = 0.26$

#	f_i	$\frac{Ffl}{AE}$
1	0	0
2	0	0
3	1.23	12.9/E
4	-0.71	1.9/E
5	0	0
6	0	0
7	0	0
8	1.33	117.1/E
9	-0.47	41.4/E
10	-0.71	4.1/E

$\frac{\Sigma Ffl}{AE} = \frac{177.4}{E}$

$\Delta = \frac{177.4(12)}{29000} = 0.07 in$