Butler Memorial Hospital

Butler, PA

BHS Butler Health System

Building for the Future: A New Era Begins

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Technical Report #3

Lateral System Analysis & Design Confirmation

Structural Option

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Due December 01, 2009

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Executive Summary:

This third of three technical reports for The Butler Health System – New Inpatient Tower Addition and Remodel focuses on the lateral support system of the structure. The lateral system for the two below grade levels is comprised of two reinforced concrete shear walls which extend up two levels and 3 of the 6 braced frames that extend down to the lowest level. Two of the three frames that extend down to the ground level are found in the elevator core area and are orthogonal to each other sharing a common lateral column.

The braced frame along gridline 2 and between gridlines D & E is the focus of this report. This frame extends down to level 1 and is braced with backfilled earth at that level; therefore no deflection was assumed at level one. It was determined that wind is the controlling lateral force.

Relative stiffness of individual frames was used to determine both strength and serviceability factors and to determine which one would control the design of the members. Both hand and SAP 2000 (2D) evaluation methods were used to evaluate the frames.

Several assumptions were made in order to simplify the analysis. First the floor systems were considered to be infinitely rigid which enabled the assumption that the frames would take load according to their relative stiffness to each other on a particular level and in the direction of loading. Another assumption made was that load at or below level 2 would not affect building drift since at this level or below there is minimal exposed exterior facade surfaces and all other perimeter walls are backfilled braced against lateral forces.

From the hand and SAP 2000 analyses drift does not appear to be the controlling design factor of the bracing members. The IBC 2006 wind drift limits were approximately 3.5 times greater than the SAP values and 7.5 times more than the hand calculated values. The construction document values, hand calculations and SAP figures for strength of a bracing member vary by a factor of 2 from highest to lowest; indicating that strength not drift was the controlling design factor.

Introduction:

Butler Health System's new addition located in Butler, PA consists of two sub grade levels which have limited facade and entrances at ground level on the plan west end of the structure. There are five other at or above grade levels that comprise the bulk of the hospitals general facilities. One more final level, the penthouse level, encompasses the mechanical equipment on the roof top.

The structure is approximately 206,000 square feet with floor to floor heights of 14'-8" each. It stands at just a little over 100' tall above the highest grade level and is situated on the middle of a hillside. With the exception of the slightly arcing plan north facade the floor plan is quite regular with typical bay sizes being 28' x30'.

Drilled caissons were used for the foundation system which range from 30" – 78" in diameter and reach depths of up to 79'. Grade beams between the caissons on the below grade level areas transfer wall loads to the foundation system and provide interior perimeter walls for the lower levels as well as provide support for the slab on grade at the second level. The superstructure is composed of steel W-shape members for the gravity load transfer components and steel HSS members in primarily an inverted chevron bracing pattern which provides the lateral force resisting system for the structure. Almost all member connections are shear connections with the exception of a few moment connections at cantilevering beams. These moment connections however do not contribute to the lateral force resisting system.

This report focuses on the existing lateral force resisting system, how loads are applied to the system, the load combinations used to determine the system, and how the system reacts to and distributes these lateral forces. A 2D frame computer analysis is performed as well as hand calculations to compare to the computer output results and to verify minimal spot checks.

For the scope and purpose of this report the braced frames at or above level two; the first level that is completely exposed above grade, will primarily be the focus for both the computer and hand calculation analysis and spot checks. The penthouse levels above the roof level 8 will also be omitted from the analysis.

Structural System:

Existing System: Rigid Diaphragm

Existing conditions for the originally designed floor system consists of composite steel decking with lightweight concrete (f'c = 3500psi @28 days). It has 20 gauge steel decking with 3" deep flutes, ¾" diameter 5" long shear studs and an additional 3.5" of concrete. The girders supporting the beams and floor system are typically W21x50, 28' long with 38 shear studs. There are typically four beams per bay including the ones at each column line. The beams are W18x40 evenly spaced at ten foot intervals and are 30 feet long with 28 shear studs each.

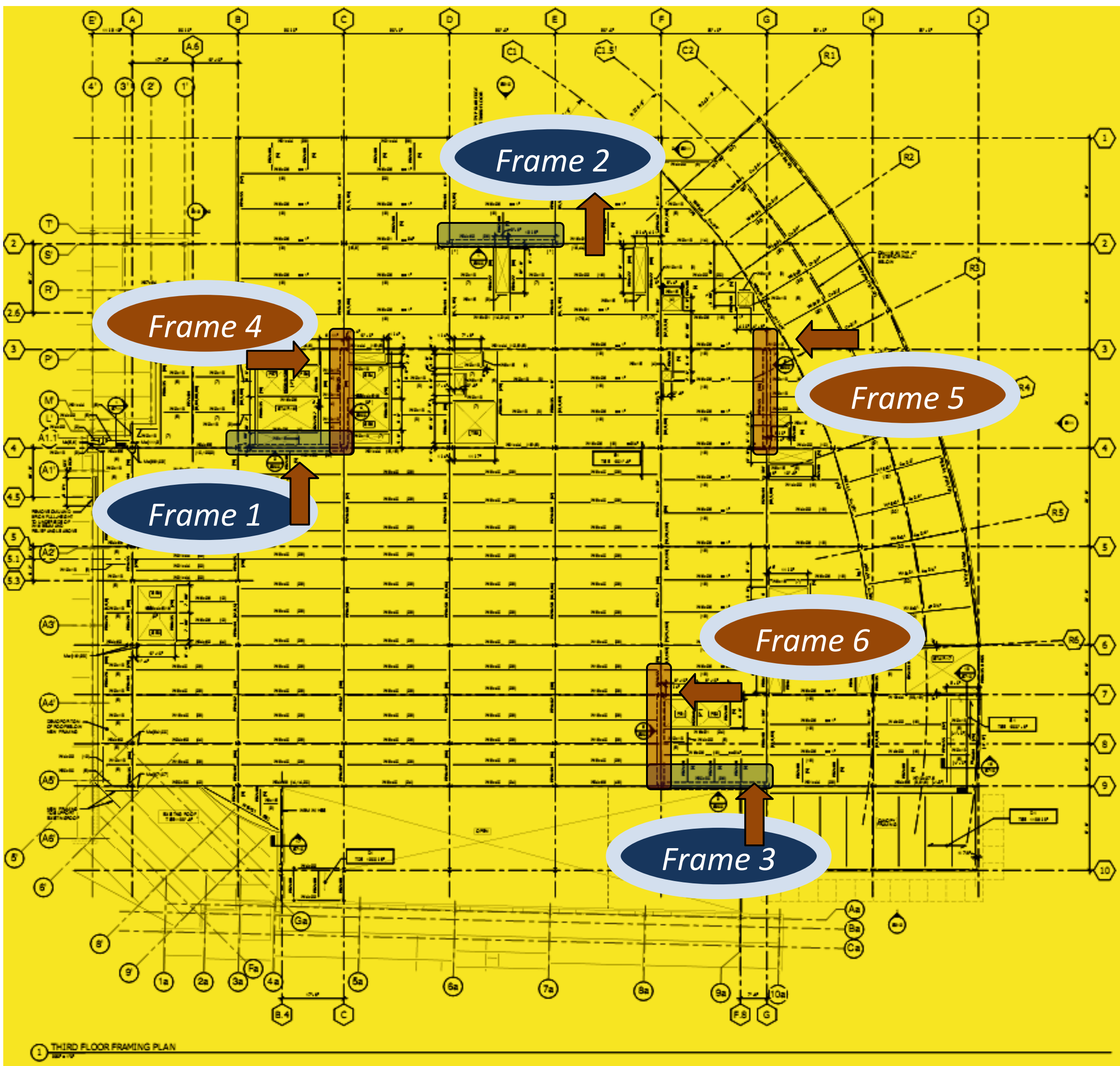


Figure 3.1: Third floor framing plan with braced frame locations shown

This composite deck and composite beam floor system is what comprises the rigid diaphragm to transfer the lateral loads into the lateral load resisting system as shown in the partial system of level 3 in figure 3.2 below. The highlighted red areas indicate the braced frame locations.

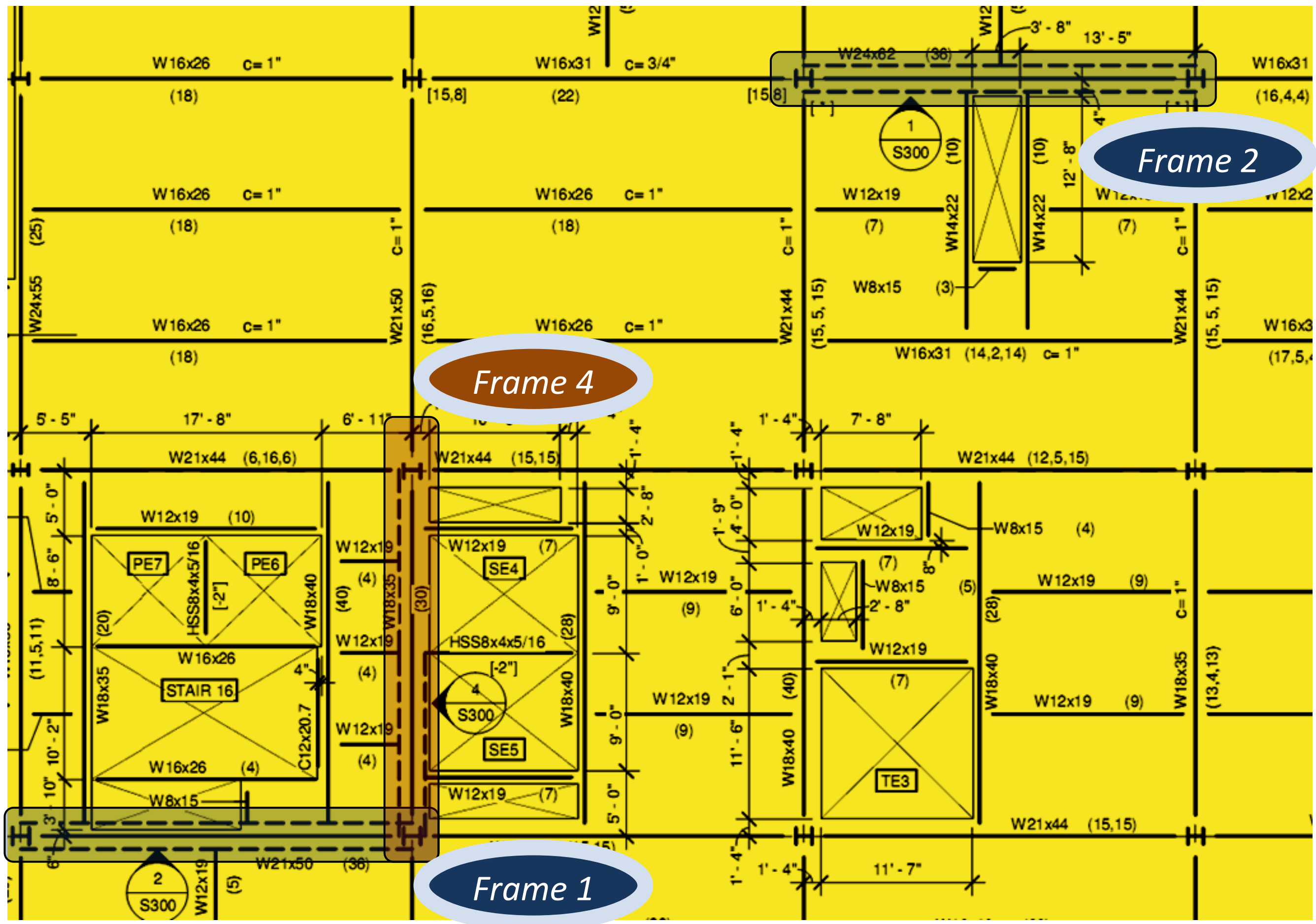


Figure 3.2: Enlarged view from Figure 3.1

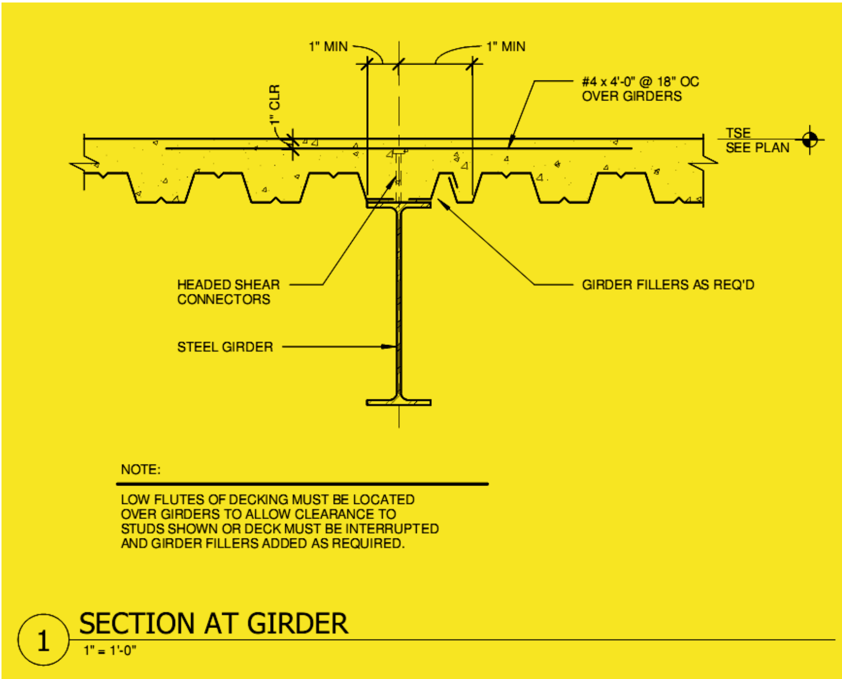


Figure 3.3: Existing slab & beam/girder conditions

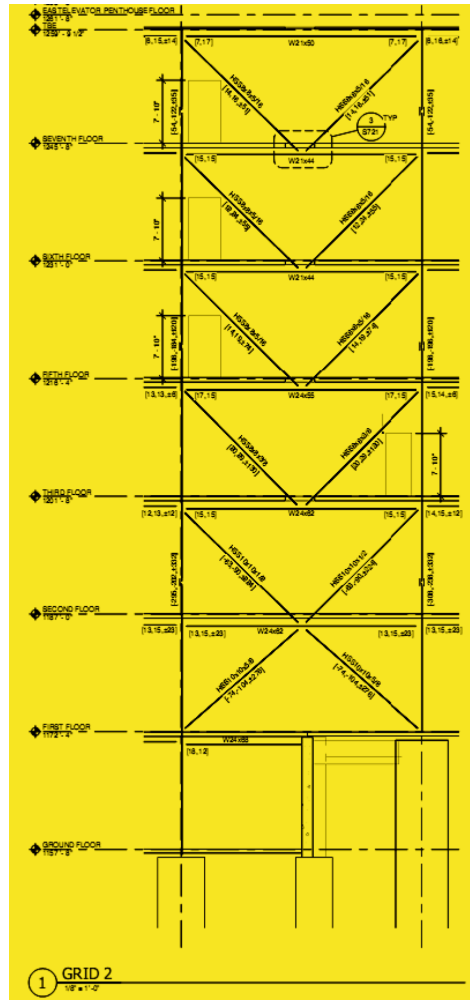


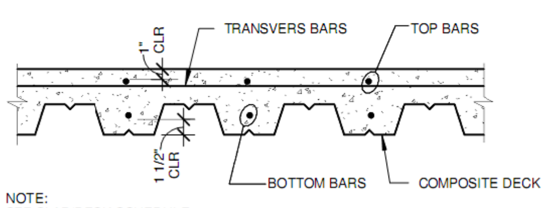
Figure 3.4: Typical lateral bracing elevation

SLAB/DECK SCHEDULE

MARK	TOTAL THICKNESS	TYPE	DECK			CONCRETE		STUD LENGTH	REINFORCING	
			DEPTH	GAGE	FINISH	THICKN	TYPE		REINF	DETAIL
S1	6 1/2"	COMP DECK	3"	20	GALV	3 1/2"	LW	5"	WWF 6x6 W2.1xW2.1	
S2	6 1/2"	COMP DECK	3"	18	GALV	3 1/2"	LW	5"	#5 @ 12"OC T & B #4 @ 12"OC TRANSVERSE	
D1	3"	ROOF DECK	3"	20	GALV	-	-	-	-	

NOTES:

1. ALL COMPOSITE SHEAR CONNECTORS (STUDS) ARE 3/4"Ø UNO.
2. NW=NORMAL WEIGHT CONCRETE; LW=LIGHTWEIGHT CONCRETE.
3. STUD LENGTHS ARE LENGTHS AFTER WELDING.
4. SEE DETAILS 1,2,3/S701 FOR SLAB REINFORCING.
5. SEE 14-16/S700 FOR DECK WELDING.
6. SEE 17/S700 FOR COMPOSITE DECK STUD PLACEMENT.



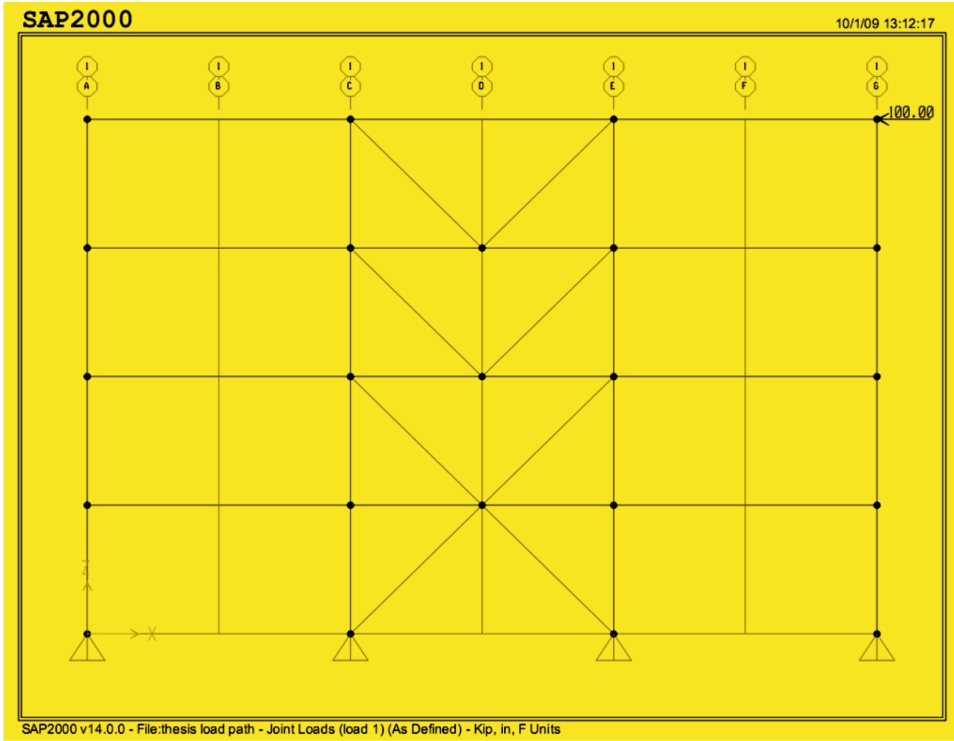
NOTE:
SEE SLAB/DECK SCHEDULE
FOR BAR SIZES & SPACING

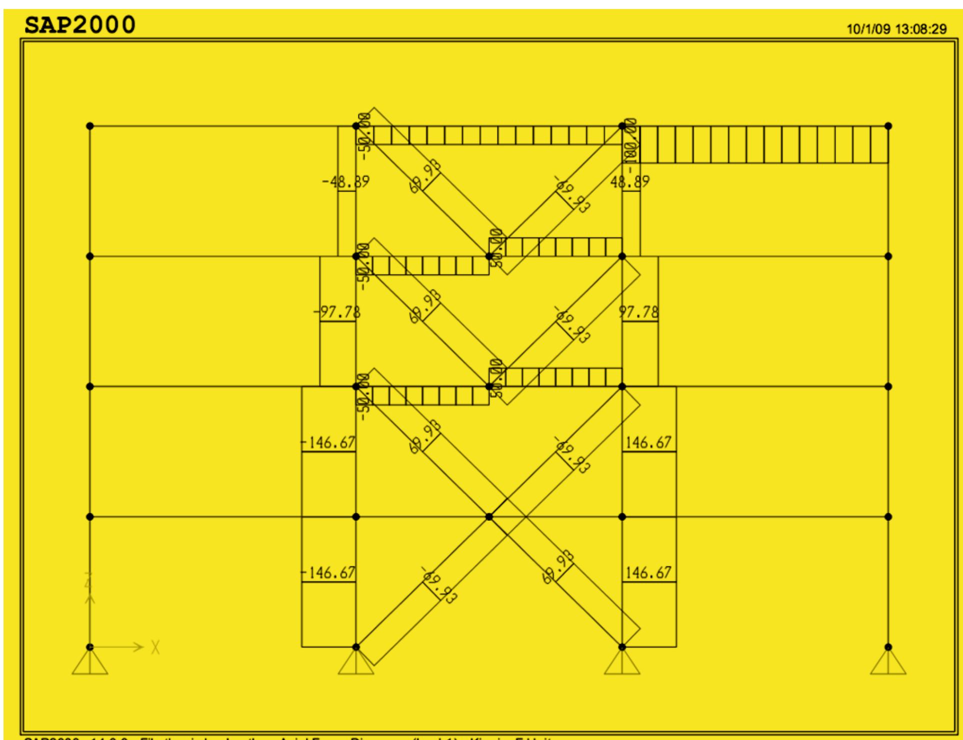
1 SLAB/DECK SCHEDULE
1" = 1'-0"

Figure 3.5: Existing slab/deck schedule

Existing System: Lateral Resistance

Lateral loads caused by wind pressures / earthquake loading are calculated using ASCE 7-05 and are resisted by the structure through the use of diagonal inverted Chevron bracing (see Figure 3.4) located at every floor level in both directions. The differing wind pressures on the exterior facade are converted to forces per square foot of wall area and are distributed to each floor level by tributary areas through the glazing and brick facade system. From there the floors are assumed to act as rigid diaphragms and distribute each floor load to the braced frames at each level according to their relative stiffnesses. This assumption can be made by viewing the composite floor system as being approximately 22-30” thick including the reinforced composite slab and composite beam/girder construction. Where there are openings in the floor extra beams are located along side them to help keep rigidity around them. Braced frames #1 & 4 are located in the elevator and stairwell core area to collect and maintain rigidity in that area where there are larger openings.
These loads are then transferred axially through the HSS members and into their corresponding beams. At the beam/girder to HSS connection there is a concentric compressive force from one brace and a concentric tension force from the other brace which cancel each other’s vertical components being transferred into the beam/girder; therefore, the force transferred into the member is axial. See figure 3.8 on page 10 for a typical brace to beam connection and figures 3.6 & 3.7 on the following page for how the load is distributed from the initial lateral force to the individual bracing and framing elements. Note how the singe lateral force at the top of the structure creates the same compressive/tensile force from top to bottom in all bracing members, but the load being transferred axially into the columns increases linearly by the force in the top column until the frame reaches its foundation support. From there the load is transferred to the ground.





Figures 3.6 & 3.7: Simplified example of lateral force distribution to braced frame and lateral load columns

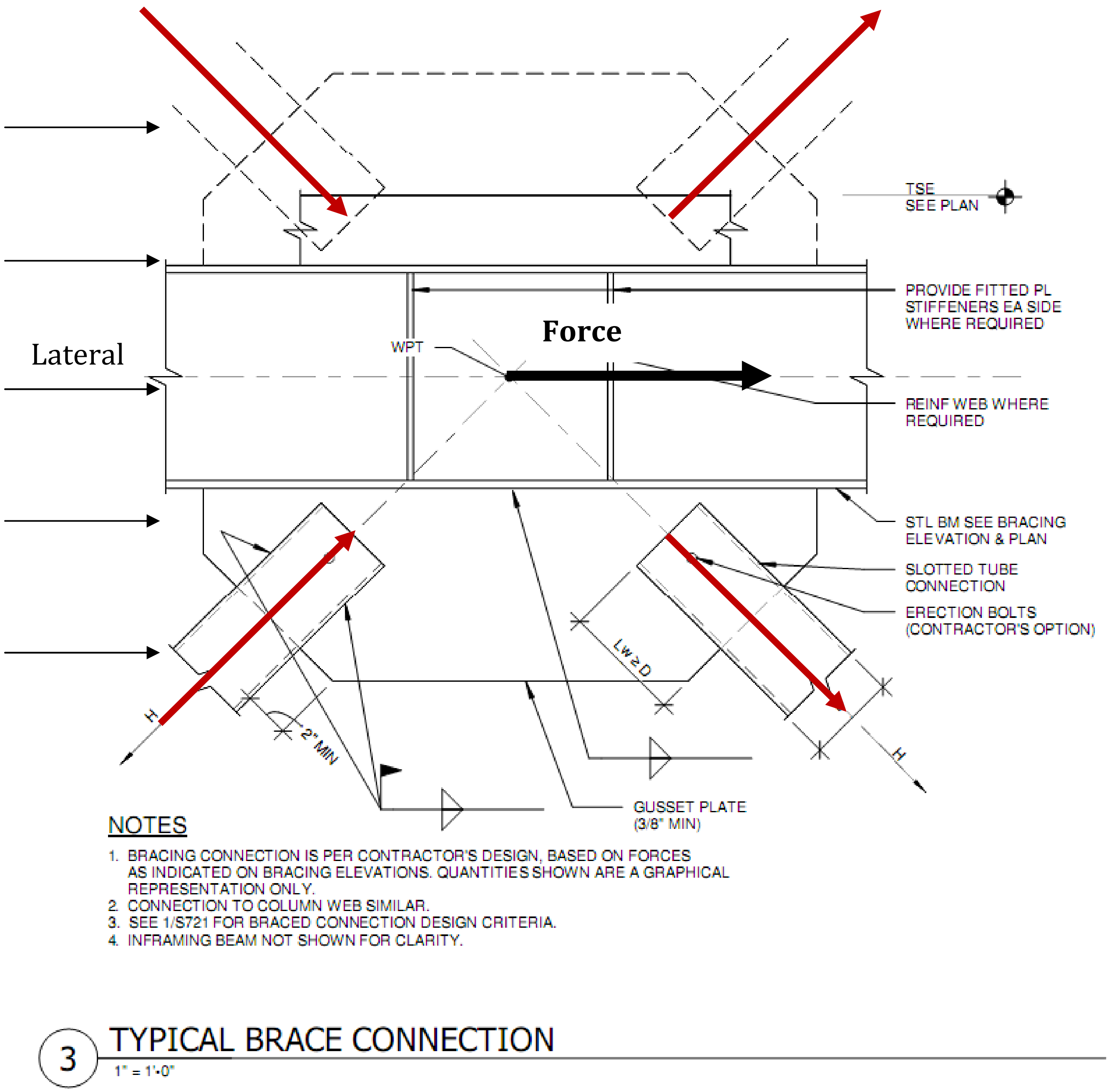


Figure 3.8: Resultant force from wind into beam/girder

Design Standards & Codes:

2006 IBC
2000 NFPA 101
2006 Guidelines for Design & Construction of Health Care Facilities
1998 Pennsylvania Department of Health Rules and Regulations for Hospitals
ASCE 7-05: for wind, seismic, snow and gravity loads
ACI 318-08: for concrete construction
AISC Thirteenth Edition: for steel members

Possible load case combinations: From ASCE 7-05 § 1605.2.1

(Only combinations that include Wind, Earthquake and/or Snow)

*Note: The snow load would be added to the total weight of the building for the earthquake loading calculations; therefore, snow by itself would not be considered.

D=Dead, L=Live, W=Wind, E=Earthquake, S=Snow, F=Fluid, T=Temperature, H=Lateral Earth Pressure, L_r=Live roof, R=Rain

1) 1.2D + 1.6(L_r or S or R)+ (L or 0.8W)
   1.2D + 1.6L_r + 0.8W for gravity and lateral
   0.8W for just lateral
2) 1.2D + 1.6W + L + 0.5(L_r or S or R)
   1.2D + 1.6W + L + 0.5L_r) for gravity and lateral
   1.6W for just lateral
3) 1.2D + 1.0E + L + S
   1. 2D + 1.0E + L + S for gravity and lateral
   1.0E for just lateral
4) 0.9D + 1.6W + 1.6H
   0.9D + 1.6W for gravity and lateral
   1.6W for just lateral
5) 0.9D + 1.0E + 1.6H
   0.9D + 1.0E for gravity and lateral
   1.0E for just lateral

1.6W or 1.0E will control for just lateral loading on the structure, whichever proves to be higher.

Design Load Summary:

Gravity Loads					
Description/location	DL/ LL	ASCE 7-05/ IBC 1607.9 values	HGA's values	Reduction available/used	Design value
Concrete floors	DL	90-115pcf	115pcf	NO/NO	115pcf
MEP/partitions/finishes	SDL	20-25psf		NO/NO	25psf
1st floor mechanical	LL		125psf	YES/NO	125psf
2nd floor/ lobby	LL	100psf	100psf	YES/NO	100psf
Hospital floors	LL	40-80psf	80psf	YES/YES	80psf
Stairs & exits	LL	100psf	100psf	NO/NO	100psf
5th floor roof	LL		115psf	NO/NO	115psf
Mech. Penthouse floor	LL		125psf	NO/NO	125psf
Elevator Machine room floor	LL		125psf	YES/NO	
Roof top equipment areas	LL		125psf (or actual equipment wt.)	NO/NO	125psf
Balconies	LL	100psf	100psf	YES/YES	psf
*Snow	LL	24-30psf	24-30psf	NO/NO	24-30psf

* See Appendix C for calculations

Table 3.1: For total dead weight of building for seismic loading

Wind Loads are determined using ASCE 7-05 Section 6.5, which is Main Wind Force Resisting System (MWFRS) method 2- analytical procedure. See Table 1B for design factor values needed in calculations. All values, factors and equations are derived from section 6. To Determine the Gust Effect Factor (G) the structure had to be determined as a rigid structure. To make this assumption 100/h has to be ≤ 1. Making the assumption that h was just under 100 feet based on the fact that the ground and second floors are minimal compared to the rest of the structure and there is only one wall face exposed; therefore the bulk of the structure completely exposed above ground would meet the requirement.
See Appendix A of structure under construction for clarity, lowest level faces west.
The wind and seismic calculations from the previous technical reports were revisited and final values were adjusted based on more accurate factor values.
See Appendix B for wind calculations.
See Appendix D for seismic calculations.

WIND LOAD
BASIC WIND SPEED (3 SECOND GUST) .... 90 MPH
WIND IMPORTANCE FACTOR 1.15
WIND EXPOSURE CATEGORY C
MEAN ROOF HEIGHT .... 122 FT
INTERNAL PRESSURE COEFFICIENT .... ±0.18
TOPOGRAPHIC FACTOR, Kzt .... 1.62 MAX AT BASE
1.09 MIN AT MEAN ROOF HEIGHT

Figure 3.9: Wind load data from construction documents

Wind Load Data for Calculations

East-West direction			ASCE section
Basic wind speed	V	90mph	6.5.4 (Figure 6-1)
Mean roof height	h	122ft	
Wind directionality factor	K_d	0.85	6.5.4 (Table 6-4)
Importance Factor (Occupancy category IV)	I	1.15	6.5.5 (Table 6-1)
Exposure category		C	6.5.6.3
Velocity pressure coefficient	K_z	varies	6.5.6 (Table 6-3)
Topographic factor	K_{zt}	varies	6.5.7.1 (Figure 6-4)
Gust effect factor	G	0.856	6.5.8
Enclosure Classification		Enclosed	6.5.9
Internal pressure coefficient	GC_{pi}	±0.18	6.5.11.1 (Table 6-3)
External pressure coefficients windward side	C_p	0.8	6.5.11.2 (Figure 6-6)
External pressure coefficients leeward side	C_p	-0.5	(Figure 6-6)
Velocity pressure @ height Z	q_z	varies	6.5.10
Velocity pressure @ mean roof height	q_h	30.41lb/ft^2	6.5.10
Design wind load	F	determined	

Table 3.2: Wind load data table for East - West loading

East – West Base & Story Shears with Overturning Moment

Level	Height (ft)	Pressure (lbs/ft ^{2})	Force (F)kips	Shear (V) kips	Moment (M) Kips*ft
		Windward + leeward			
0- Ground	0	24.59	21.64	545.75	4000.3
1	14'-8"	24.59	52.10	524.11	3841.7
2	29'-4"	26.48	69.29	472.01	3459.8
3	44'-0"	27.30	81.26	402.72	2951.9
5	58'-8"	27.57	84.91	321.46	2356.3
6	73'-4"	27.59	84.64	236.55	1733.9
7	88'-0"	27.38	83.51	151.91	1113.5
8-Roof	102'-8"	26.87	49.5	68.4	501.4
9- P.H. 1	122'-0"	26.30	13.52	18.9	182.7
10- P.H. 2	135'- 0"	25.87	5.38	5.38	34.97
			Base Shear =	**545.75**	
			Overturning Moment =		**20176.52**

Table 3.3: See Appendix B for calculations and drawings

Wind Load Data for Calculations

North-South direction			ASCE section
Basic wind speed	V	90mph	6.5.4 (Figure 6-1)
Mean roof height	h	122ft	
Wind directionality factor	K_d	0.85	6.5.4 (Table 6-4)
Importance Factor	I	1.15	6.5.5 (Table 6-1)
Exposure category		C	6.5.6.3
Velocity pressure coefficient	K_z	varies	6.5.6 (Table 6-3)
Topographic factor	K_{zt}	varies	6.5.7 (Figure 6-4)
Gust effect factor	G	0.857	6.5.8
Enclosure Classification		Enclosed	6.5.9
Internal pressure coefficient	GC_{pi}	±0.18	6.5.11.1 (Table 6-3)
External pressure coefficients windward side	C_p	0.8	6.5.11.2 (Figure 6-6)
External pressure coefficients leeward side	C_p	-0.5	(Figure 6-6)
Velocity pressure @ height Z	q_z	varies	6.5.10
Velocity pressure @ mean roof height	q_h	30.41/ft^2	6.5.10
Design wind load	F	determined	

Table 3.4: Wind load data table for North – South loading

North - South Base & Story Shears with Overturning Moment

Level	Height (ft)	Pressure (lbs/ft²)	Force (F)kips	Shear (V) kips	Moment (M) Kips*ft
		Windward + leeward			
0- Ground	0	0	0	557.55	4086.84
1	14’-8”	24.60	15.69	557.55	4086.84
2	29’-4”	26.61	72.10	541.86	3971.83
3	44’-0”	27.33	98.45	469.76	3443.34
5	58’-8”	27.61	100.27	371.31	2721.70
6	73’-4”	27.63	93.73	271.04	1986.72
7	88’-0”	27.43	86.37	177.31	1299.68
8-Roof	102’-8”	26.91	62.53	90.94	666.59
9- P.H. 1	122’-0”	26.34	23.96	28.41	274.58
10- P.H. 2	135’- 0”	25.90	4.45	4.45	28.93
			Base Shear =	**557.55**	
			Overturning Moment =		**22567.05**

Table 3.5: See Appendix B for calculations and drawings

Snow loads are determined using ASCE 7-05 Chapter 7. The design values in sections 7.1-7.3 all agree with HGA’s values (see Appendix C notes on snow loads.) A minimum roof design load of 30psf will be used for calculations.

SNOW LOAD
GROUND SNOW LOAD, Pg .... 25 PSF
FLAT ROOF SNOW LOAD, Pf .... 24 PSF
MINIMUM ROOF DESIGN LOAD .... 30 PSF
SNOW IMPORTANCE FACTOR 1.2
SNOW EXPOSURE FACTOR, Ce 1.0
THERMAL FACTOR, Ct (BUILDING) 1.0
THERMAL FACTOR, Ct (CANOPIES) 1.2

Figure 3.10: Construction document values

As per ASCE 7-05 § 12.7.2; effective seismic weight:
4) where the flat roof snow load exceeds 30psf use 20%; otherwise it is not required. (P_f designed and calculated = 24psf Therefore not applicable)

Seismic design criteria are based off of ASCE 7-05 Chapters 11, 12, 14 & 22 for seismic design. Using Table 12.2-1Design Coefficients and Factors for Seismic Force-Resisting Systems, category B: Building Frame Systems #4 (ordinary steel concentrically braced frames), several factors can be obtained. Response modification coefficient (R^a) =3.25, system overstrength factor (Ω_o^g)=2, deflection amplification factor (C_d^b)=3.25, seismic design category C, no limitation to building height.

ASCE 7-05 Section 14.1is where the detailing requirements are specified. The designer's data is listed below in Figure 3.11. All of the design criteria compare to my calculations except for the discrepancy in the spectral response acceleration (S_1) value. Designed is 0.0055, determined is 0.046 from ASCE 7-05 Figure 22-2.

SEISMIC DESIGN DATA
SPECTRAL RESPONSE ACCELERATION, Ss 0.0127
SPECTRAL RESPONSE ACCELERATION, S1 .... 0.0055
SITE CLASS... C
SEISMIC IMPORTANCE FACTOR..................... 1.5
SEISMIC DESIGN CATEGORY (SDC) .... A

Figure 3.11: Construction document data for seismic

The effective seismic weight (W_T) is determined using information from section 12.7.2., and totaled using an excel spreadsheet found in Appendix D.

V= base shear = Cs*W_T
Cs= 0.0456
W_T= 18675.1 kips
V= 851.58 kips
See Appendix D for data and calculations.

Once the base shear has been calculated then this load is distributed to each individual floor level using relative floor stiffnesses which are based off of the proportions of floor areas and ½ column weights of the floor above and below. The square footage of the roof area is broken up and divided out over levels 8, 7, 5 and 3, which are the levels that contain roof areas.

Total Dead Load for Seismic Calculation

W_T

Load type

Floor Level	square footage	wall square footage	Concrete deck psf 51.0	Superimposed MEP/Partitions 25.0	Columns kips	Beams lb/ft^2 7.0	equipment psf 1.0	roof psf 42.0	exterior walls psf/wall 28.6	Floor weight Totals for relative stiffness
Ground	8240									
Level 1	20405	170	1040.66	510.13	58.39	142.84	20.41	0	4.86	1772.4
Level 2	45545	458	2322.80	1138.63	50.59	318.82	45.55	0	13.10	3876.4
Level 3	42165	458	2150.42	1054.13	68.99	295.16	42.17	0	13.10	3610.9
Level 5	31525	458	1607.78	788.13	41.83	220.68	31.53	0	13.10	2689.9
Level 6	27720	678	1413.72	693.00	39.50	194.04	27.72	0	19.39	2368.0
Level 7	27760	678	1415.76	694.00	29.86	194.32	27.76	0	19.39	2361.7
Level 8 (roof)	45545							1912.89		1912.9
TOTALS	**248905**	**2900**	**9951.12**	**4878**	**289.159**	**1365.84**	**195.12**	**1912.89**	**82.94**	

W_T = 18675.1 kips

C_S = 0.0456

$V = C_S * W_T$ = 851.58

Figure 3.12: EXCEL spreadsheet calculating seismic base shear
See Appendix C for additional calculations

<u>Controlling lateral load combination:</u> 1.6W or 1.0E for just lateral loading

<u>1.6W = 1.6(557.550) =892kips, from wind N-S; CONTROLS</u>

1.6W = 1.6(545.75) =873kips, from wind E-W

1.0E = 1.0(851.58) =852kips, from Seismic

A factored load of 1.6 times the wind force at each level will be used in calculations to determine, relative stiffness of braces on each level, distribution of load to braces, and eventually the force in the members.

Force Distribution:

For the scope and purpose of this report the braced frame section from level 3 to level 5 along grid line 2 between grid lines D-E will be analyzed; which is what I am calling frame #2 and will be assumed to be resisting N-S applied wind forces. See Figure 3.13 below for frame detail.

See enlarged view on following page Figure 3.14 for more as designed details

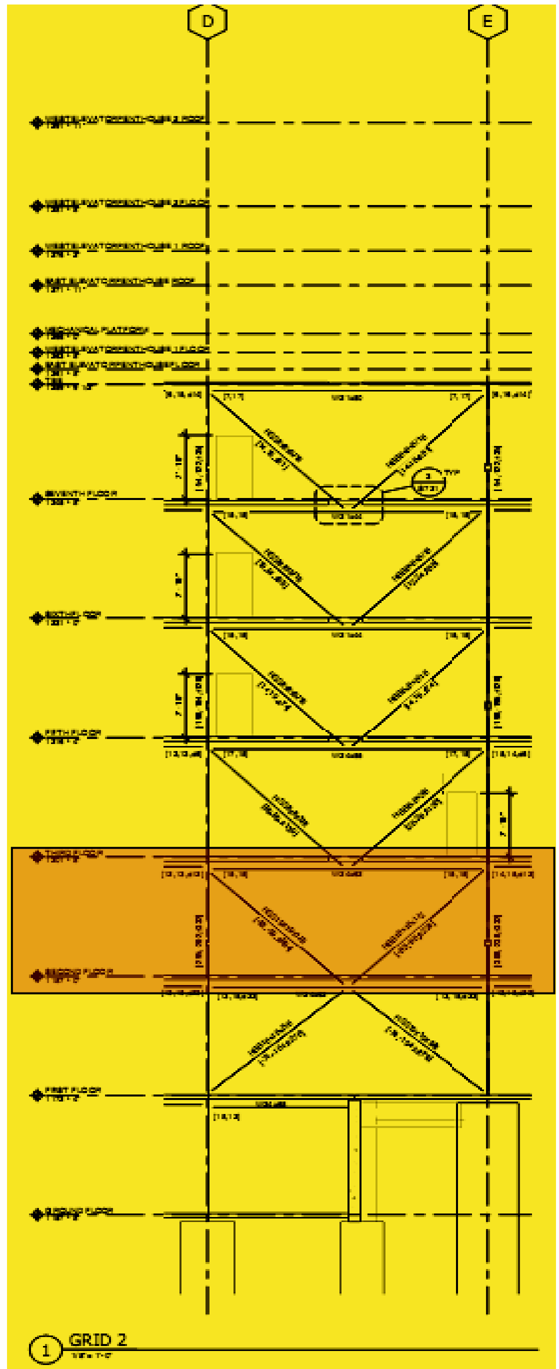


Figure 3.13: Braced frame at grid 2 between D & E

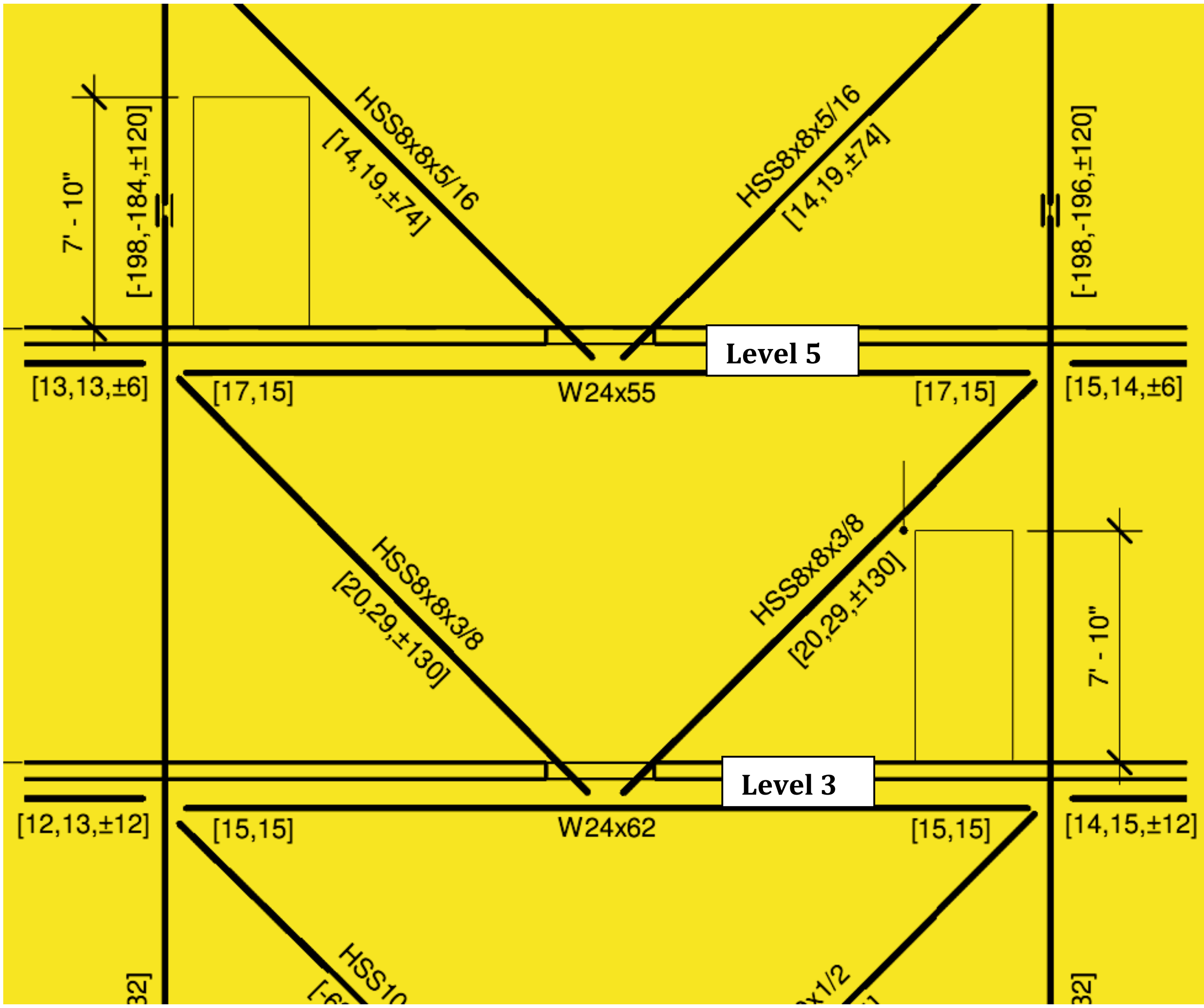


Figure 3.14: Enlarged view of braced frame at grid 2 between D & E

Analysis Method:

As shown earlier in Figures 3.6 & 3.7 a force on an upper level of a Chevron type braced frame will induce a compressive force in one brace and a tension force in the other that will carry itself down through all bracing members below that level. It will also introduce a compressive force in one column and a tensile force in the other that will compound itself linearly in each respective vertical element. Therefore the forces at each level cannot be analyzed individually; they will have to be combined with the forces acting on the levels above to get a more accurate result. This is also part of the reason why the HSS member sizes increase in section and wall thickness as more floors are added above even as the forces at each level are relatively the same.

The first step in the analysis process is to assume the floor levels are acting as rigid diaphragms and to determine the center of mass for the rigid diaphragm above the level being analyzed, which is the area/mass that is applying the load to the braces. See Appendix E for these calculations.

Next would be to calculate the center of rigidity for each of these levels to determine how much of the force at the respective level will go into each brace at that level based on their relative stiffnesses to each other and torsional effects due to eccentric differences in center of mass versus center of rigidity. This is the axial force being introduced into the bracing elements below the level. Note: Only the diagonal braces in the same direction of the loading will be considered to be resisting the lateral load in that direction; and the columns and beams that make up part of the braced frame are not considered for stiffness criteria.

Once the level forces, center of mass, center of rigidity and relative stiffnesses have been determined then the direct force and eccentric force at that level can be calculated. These two forces can then be added together to determine the force being applied at that level to each individual frame. The value for the eccentric force being added to or subtracted from the direct force will always be considered positive since load reversal can be applied and the eccentric forces would switch signs but he direct forces would remain the same.
These forces can then be applied to the Free Body Diagram for the frame and the element member forces can be determined and checked against the computed design values and subsequent sizes.
See Appendix E for drawings and calculations including FBD of braced frame #2 and SAP verification of FBD and member forces.

Tabulated values of hand calculations

Level	Frame Stiffness (kip/in)						Center of Rigidity		Story Shear (kips)		Eccentricity	
	1	2	3	4	5	6	X (ft)	Y (ft)	N-S	E-W	e_x (ft)	e_y (ft)
3	1198.01	1198.01	2419.07	1573.30	2050.06	2120.89	106.06	121.8	98.45	81.26	19.77	12.3
5	1198.01	1198.01	1520.56	1573.30	2050.06	2120.89	106.06	107.53	100.27	84.91	19.77	1.97
6	1009.10	1198.01	1198.01	1239.57	1573.30	1627.66	105.51	101.37	93.73	84.64	25.89	3.67
7	1009.10	1009.10	1009.10	1044.10	1239.57	1080.18	103.12	100.67	86.37	83.51	7.20	32.13
8	1009.10	1009.10	1009.10	1044.10	1239.57	1080.18	103.12	100.67	62.53	49.50	7.20	32.13
Average	1355.83	1403.06	1788.96	1618.59	2038.14	2007.45	130.9675	133.01			19.9575	20.55
								Total=	441.35	383.82		

Table 3.6: Tabulated values to evaluate member forces

Level	Direct Shear (kips)						Torsional Shear (kips) *5% minimum of Direct					
	1	2	3	4	5	6	1	2	3	4	5	6
3	24.50	24.50	49.46	22.26	29.00	30.00	1.91	8.05	16.668	9.604	3.844	*1.50
5	30.67	30.67	38.93	23.26	30.30	31.35	2.80	11.13	13.93	1.49	*1.52	*1.57
6	32.98	32.98	27.78	23.63	29.99	31.02	2.26	14.35	16.61	2.76	2.07	*1.55
7	28.79	28.79	28.79	25.92	30.77	26.82	*1.44	3.63	4.28	23.78	18.10	5.68
8	20.84	20.84	20.84	15.36	18.24	15.90	*1.04	2.63	3.10	14.10	10.73	3.37
Total	137.78	137.78	165.80	110.43	138.30	135.09	9.45	39.79	37.92	42.13	36.26	13.67

Total Shear (kips)						
Frame	**1**	**2**	**3**	**4**	**5**	**6**
Level 3	26.41	57.51	66.13	31.86	32.84	31.50
Level 5	33.47	41.80	52.86	24.75	31.82	32.92
Level 6	35.24	47.33	44.39	26.39	32.06	32.57
Level 7	30.23	32.42	33.07	49.70	48.87	32.50
Level 8	21.88	23.47	23.94	29.46	28.97	19.27
Total	147.23	202.53	220.39	162.16	174.56	148.76

Table 3.7: Resulting shears due to wind loads

Deflection criteria as per 2006 International Building Code:

Allowable building drift: $\Delta_{wind} = H/400$

Allowable story drift: $\Delta_{seismic} = 0.10h_{sx}$ (Table 12.12-1 ASCE 7-05)

Equation used to calculate story drift Δ_s: $K=P/\Delta_p$ $\Delta_p=P/K$

Wind Drift Comparison of Frame #2										
Level	Story Height (ft)	Story Drift (in)	Allowable Story Drift $\Delta_{wind} = H/400$ (in)			Total Drift (in)	Allowable Total Drift $\Delta_{wind} = H/400$ (in)			
3	14.67	0.0782	<	0.44	Acceptable	0.0782	<	1.32	Acceptable
5	14.67	0.0837	<	0.44	Acceptable	0.162	<	1.76	Acceptable
6	14.67	0.0782	<	0.44	Acceptable	0.240	<	2.2	Acceptable
7	14.67	0.0856	<	0.44	Acceptable	0.326	<	2.64	Acceptable
8/roof	14.67	0.0620	<	0.44	Acceptable	0.388	<	3.08	Acceptable

Table 3.8: Drift Values from hand calculations

SAP 2000 2d Frame Analysis to compare with hand calculations:

The relative stiffness of each frame can be approximated by taking the inverse of the deflection of each frame and relating them to each other by taking its value an dividing by the sum of a the other frames in the same participating direction. This could also be done on a level by level basis to get a more accurate assumption. Since the second approach was used for the hand calculations the computer analysis will be done the same way for more consistency.

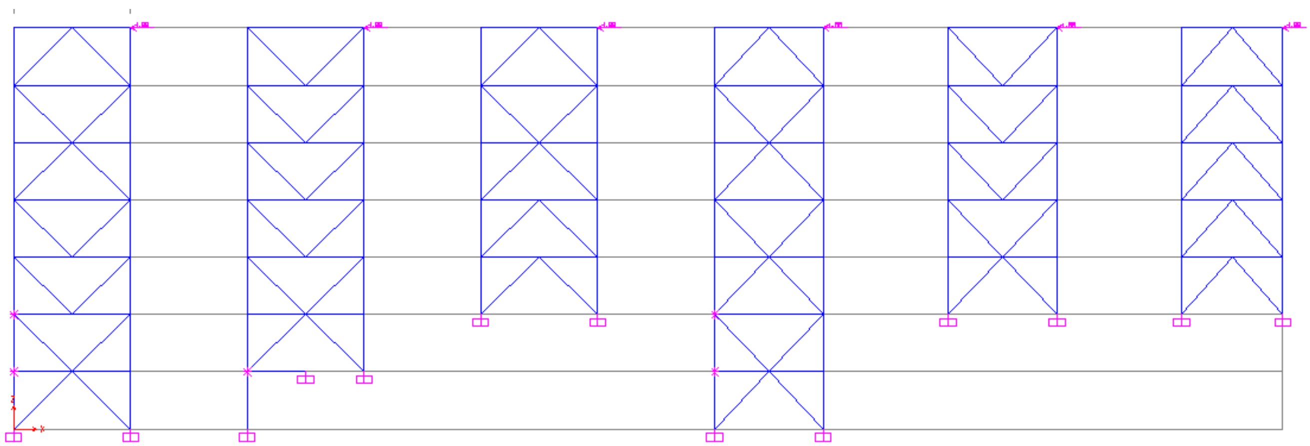
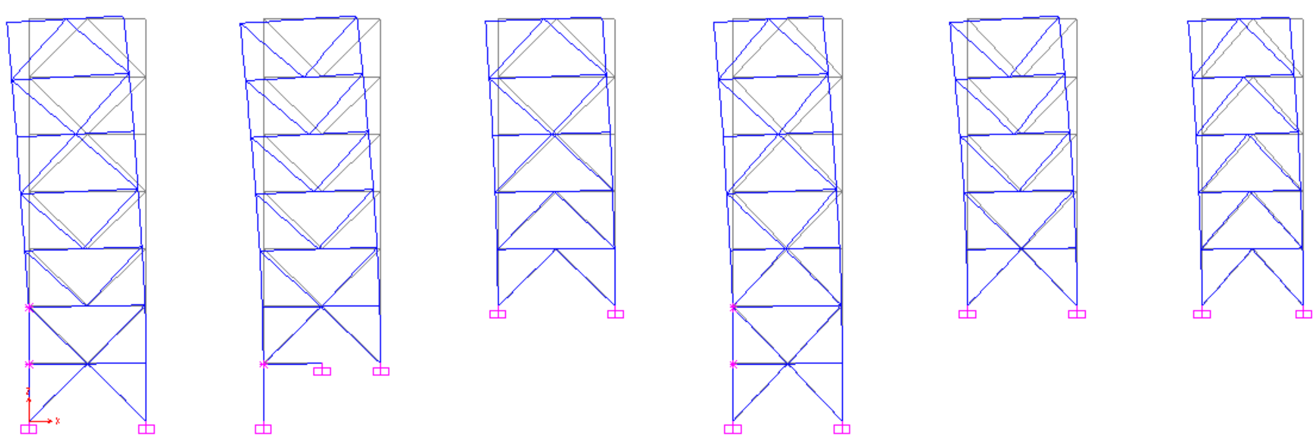


Figure 3.15: Frames 1-6 with 1 kip load applied to determine relative stiffnesses of frames.

Δ_T=-0.01342 Δ_T=-0.01423 Δ_T=-0.00773 Δ_T=-0.01156 Δ_T=-0.01078 Δ_T=-0.00840



#1 #2 #3 #4 #5 #6

Figure 3.16: Deflected shapes with total displacements caused by 1 kip at top

Level	Displacement for Stiffness Calculations (Δ_s)					
	Frames 1-6					
3	0.00251	0.00220	0.00005	0.00157	0.00005	0.00007
5	0.00507	0.00461	0.00168	0.00371	0.00203	0.00182
6	0.00741	0.00761	0.00317	0.00581	0.00445	0.00353
7	0.01042	0.01093	0.00545	0.00860	0.00756	0.00585
8/roof	0.01342	0.01423	0.00773	0.01156	0.01078	0.00840

Table 3.9: Frames 1-6 showing displaced shape due to 1 kip load @ top of frame and relative displacements at each level.

Level	K for each brace ($1/\Delta_s$)					
	Frame 1	Frame 2	Frame 3	Frame 4	Frame 5	Frame6
3	398.406	454.545	20000	636.943	20000	14285.71
5	197.239	216.920	595.238	269.542	492.611	549.451
6	134.953	131.406	315.457	172.117	224.719	283.286
7	95.9691	91.4913	183.486	116.279	132.275	170.940
8/roof	74.5156	70.2741	129.366	86.5052	92.7644	119.048

Table 3.10: Stiffness of each brace at each level (k/in) based off of 1k load @ top of frame

Level	% Stiffness per Brace ($K/\Sigma K$)					
	Frame 1	Frame 2	Frame 3	Frame 4	Frame 5	Frame6
3	1.91	2.18	95.91	1.83	57.31	40.86
5	19.54	21.49	58.97	20.55	37.56	41.89
6	23.20	22.59	54.22	25.31	33.04	41.65
7	25.87	24.66	49.46	27.72	31.53	40.75
8/roof	27.18	25.54	47.19	29.00	31.10	39.91

Table 3.11: Percentage of load to each frame at each level based off of 1k load @ top of frame

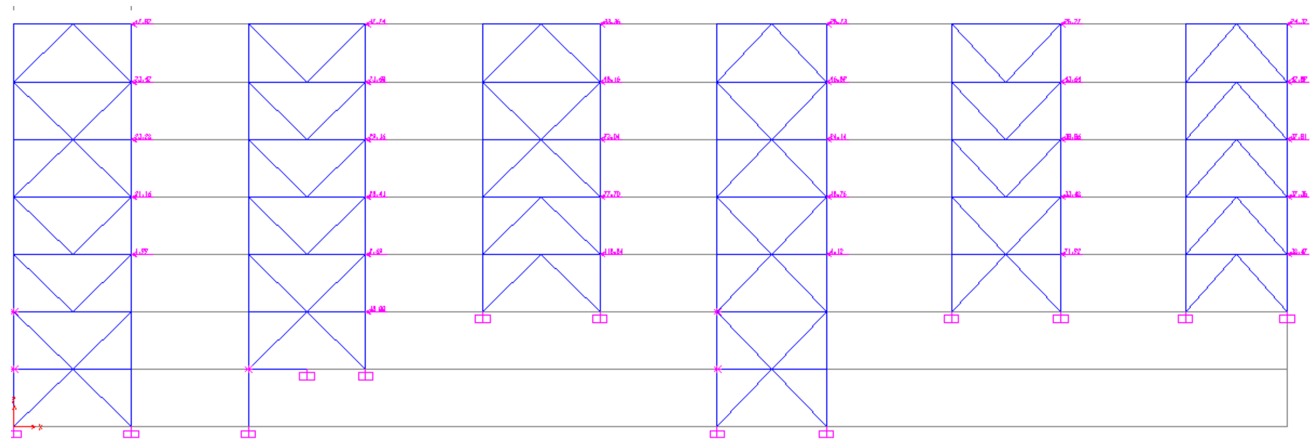
To determine the total force that is transmitted into each brace on each level the values from Table 3.11; as a fraction, are multiplied by the story shear at the corresponding level, which can be found in Tables 3.3 & 3.5. This however does not account for the torsional shear; which can be seen from Table 3.7 in the hand calculations could be close to 30% of the direct shear. To try and reasonably account for these torsional shears the eccentricities calculated by hand are assumed to be accurate here.

SAP Model Calculations												
Level	**Direct Shear** (kips)						**Torsional Shear** (kips) *5% minimum of Direct					
	1	2	3	4	5	6	1	2	3	4	5	6
3	1.88	2.15	94.42	1.49	46.57	33.20	0.11	0.54	24.42	2.63	25.35	5.27
5	19.59	21.55	59.13	17.45	31.89	35.57	1.57	6.86	18.57	1.31	1.59	1.78
6	21.75	21.17	50.82	21.42	27.97	35.25	1.53	7.99	22.22	2.72	2.09	1.76
7	22.34	21.30	42.72	23.15	26.33	34.03	1.08	2.30	5.44	23.74	17.31	8.06
8	17.00	15.97	29.51	14.36	15.39	19.76	0.82	1.77	3.85	14.37	9.88	4.56
Total	82.56	82.14	276.60	77.87	148.15	157.81	5.11	19.46	74.50	44.76	56.22	21.43

Total Shear (kips)						
Frame	**1**	**2**	**3**	**4**	**5**	**6**
Level 3	1.99	2.69	118.84	4.12	71.92	38.47
Level 5	21.16	28.41	77.70	18.76	33.48	37.35
Level 6	23.28	29.16	73.04	24.14	30.06	37.01
Level 7	23.42	23.60	48.16	46.89	43.64	42.09
Level 8	17.82	17.74	33.36	28.73	25.27	24.32
Total	87.67	101.60	351.10	122.63	204.37	179.24

Table 3.12: Resulting shears due to wind loads from SAP 2000

These computed total story shears from Table 3.12 are placed at the nodes of the frames on their corresponding levels in the 2D frame model to evaluate total drift and compare the values with the hand calculations and the 2006 IBC deflection criteria.



#1 #2 #3 #4 #5 #6

Figure 3.17: Frames 1-6 with loads applied to determine deflections of frames to compare with hand calculations.

Comparisons:

Wind Drift Comparison of Frame #2 using SAP 2000 2D									
Level	Story Height (ft)	Story Drift (in)	Allowable Story Drift Δ wind = H/400 (in)			Total Drift (in)	Allowable Total Drift Δ wind = H/400 (in)		
3	14.67	0.1214	<	0.44	Acceptable	0.18813	<	1.32	Acceptable
5	14.67	0.1858	<	0.44	Acceptable	0.37396	<	1.76	Acceptable
6	14.67	0.1912	<	0.44	Acceptable	0.56512	<	2.2	Acceptable
7	14.67	0.1703	<	0.44	Acceptable	0.73544	<	2.64	Acceptable
8/roof	14.67	0.1456	<	0.44	Acceptable	0.88103	<	3.08	Acceptable
Wind Drift Comparison of Frame #2 using hand calculations									
Level	Story Height (ft)	Story Drift (in)	Allowable Story Drift Δ wind = H/400 (in)			Total Drift (in)	Allowable Total Drift Δ wind = H/400 (in)		
3	14.67	0.0782	<	0.44	Acceptable	0.0782	<	1.32	Acceptable
5	14.67	0.0837	<	0.44	Acceptable	0.162	<	1.76	Acceptable
6	14.67	0.0782	<	0.44	Acceptable	0.240	<	2.2	Acceptable
7	14.67	0.0856	<	0.44	Acceptable	0.326	<	2.64	Acceptable
8/roof	14.67	0.0620	<	0.44	Acceptable	0.388	<	3.08	Acceptable

Table 3.13: Drift comparison table

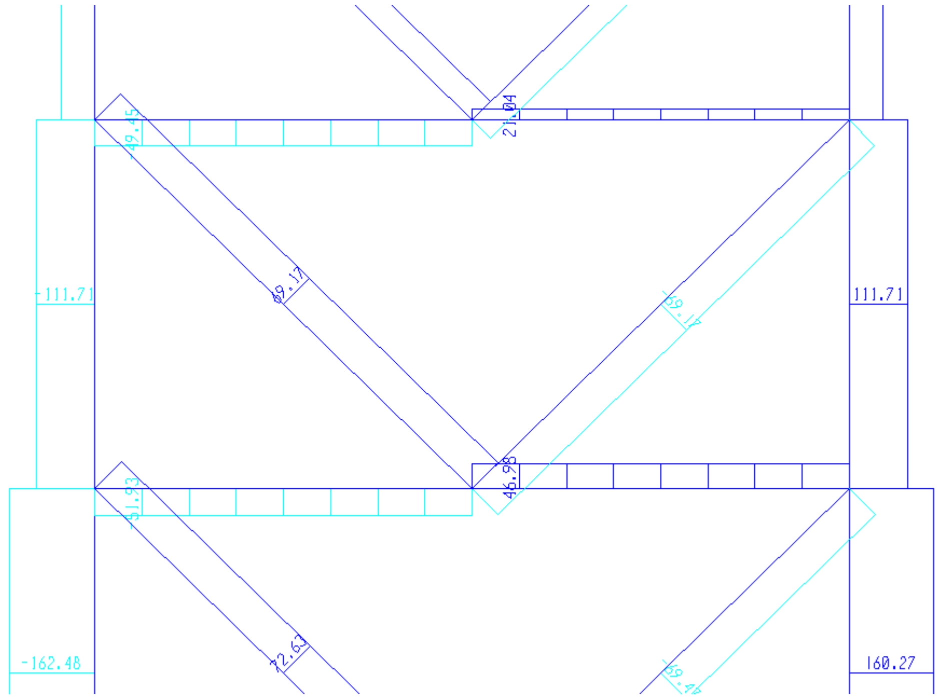


Figure 3.18: SAP 2000 frame #2 axial load output

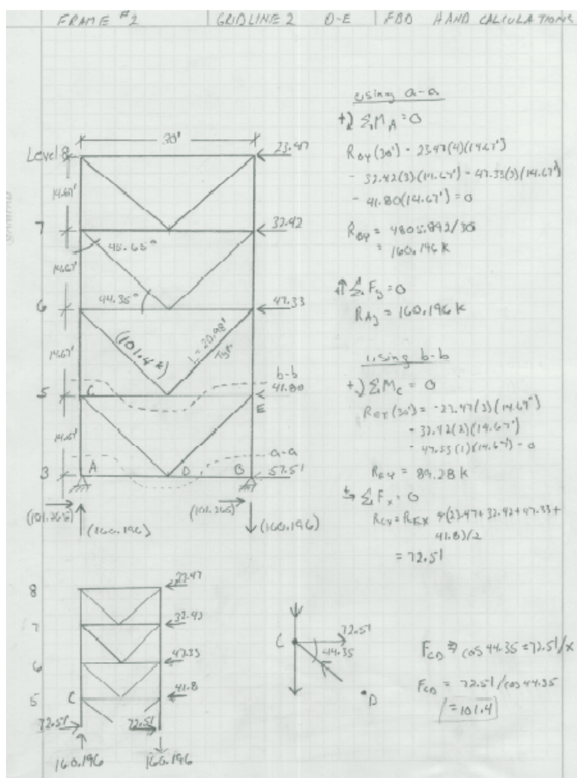
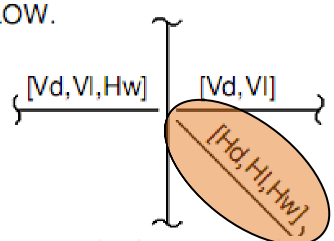


Figure 3.19: FBD of hand calculation for frame #2 to compare values with Figure 3.18. An enlarged view of this figure can be found in the beginning of Appendix F.

BRACED FRAME CONNECTION NOTES

1. SEE PLAN, BRACE ELEVATIONS, AND COLUMN SCHEDULE FOR MEMEBERS SIZES.
2. BRACING AND BEAM MEMBER SERVICE (UNFACTORED) FORCES ARE INDICATED ON ELEVATIONS, AS SHOWN IN FIGURE BELOW.
   Hd=AXIAL DEAD LOAD
   Hl=AXIAL LIVE LOAD
   Hw= AXIAL WIND LOAD
   Vd=SHEAR DEAD LOAD
   Vl=SHEAR LIVE LOAD



   TENSION AXIAL FORCE AND DOWNWARD SHEAR FORCE ARE POSITIVE.
   COMPRESSION AXIAL FORCE AND UPWARDS SHEAR FORCE ARE NEGATIVE.
3. NO REDUCTION IN SERVICE LEVEL FORCES OR INCREASES IN ALLOWABLE STRESSES SHALL BE ALLOWED IN DESIGN OF CONNECTIONS.
4. FABRICATOR TO ENSURE BRACE LENGTHS AND SLOT DIMENSIONS ALLOW PLACEMENT OF BRACE BETWEEN GUSSET [PLATES.
5. SEE GENERAL STRUCTURAL NOTES ON SHEET S001 FOR ADDITIONAL INFORMATION.
6. BEAMS AND COLUMNS ARE NOT DESIGNED FOR BENDING MOMENT DUE TO CONNECTION ECCENTRICITY. PROPORTIN CONNECTION TO TO ELIMINATE ADDITIONAL MOMENTS ON BEAMS AND COLUMNS
7. AXIAL FORCES IN BRACED FRAME BEAMS ARE NOT SHOWN. DETERMINE FORCE REQUIRED TO OBTAIN CONNECTION FORCE EQUILIBRIUM.

1 BRACED FRAME NOTES

1" = 1'-0"

Figure 3.20: Description from print to show value meanings and to compare with SAP and hand calculations.

Axial Force in Brace from Level 3 to Level 5 in Frame #2			
	Print	Hand Calculations	SAP 2000
H_w	130kips	101.4kips	69.17kips

Table 3.14: Brace comparison values

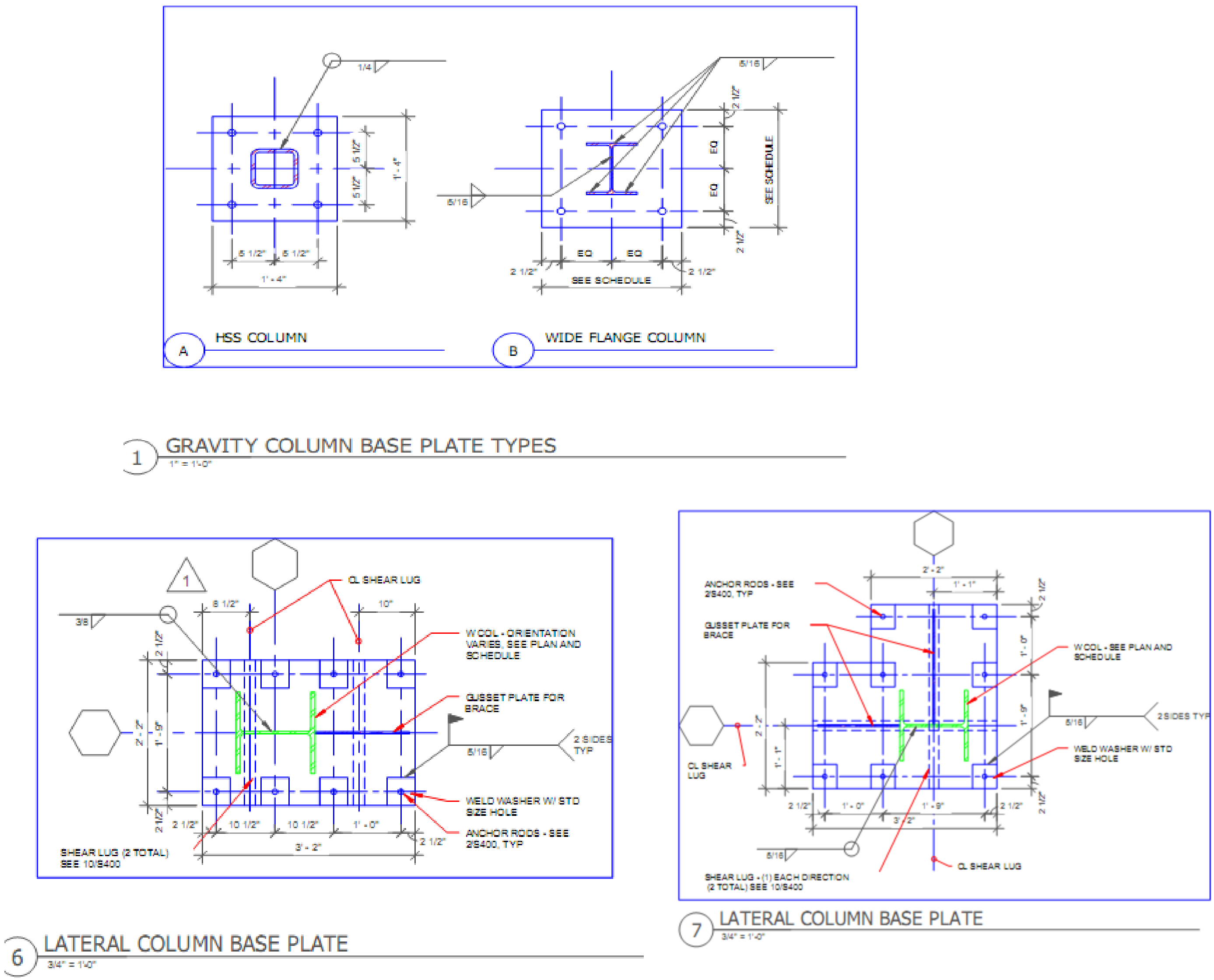


Figure 3.21: Gravity and Lateral base plate to foundation connection detail

Overturning:

The drawings in Figure 3.21 depict the differences in the base plate to caisson connection details for lateral versus gravity columns. The reason for the difference in anchor size, depth, number and layout is because of the overturning moment caused by the lateral loading on the structure. As shown in Figures 3.18 & 3.19 at each braced frame location there will be one side of the frame columns in

compression and the other in tension. Depending on the lateral loading direction there will also be a moment of approximately 20,000-23000 foot kips applied to the base of the columns, this load (moment) would be distributed among the columns which are participating in the loaded direction similar to the manner in which the lateral load is distributed to the braced frames.
The uplift force seen in the columns that are in tension would be negated by the gravity forces in the columns imposed by dead and live loading of the structure as well as the connected weight of the 30”-78” Ø and up to 79’ deep caissons; therefore overturning issues would not be a concern or issue.

Member Checks:

The bracing member compared in Table 3.14 is checked for strength and size using the hand calculation and the value given on the construction documents. One column in the same braced frame between levels 3 and 5 is also checked for compression, lateral stability and size. To compare and evaluate the members in the design documents the gravity loads applied to the columns, beams and HSS members and any moments that are applied to the columns also have to be considered. After determining the gravity loads, the loads will be applied to a simple 2D SAP model to get the member forces to be added to the lateral analysis.

These calculations can be found in Appendix F at the end of this report.

Conclusions:

Based on the calculations and comparisons in this report the lateral force resisting system is designed for strength more than for drift considerations. This conclusion seems completely plausible since two of the levels are relatively small compared to the rest of the structure and are only minimally exposed on one side. There are five other main levels above ground and a smaller penthouse level on the roof. The height of these levels compared to the length and width of the structure is approximately 1:2 making the building relatively short, almost symmetrically square and stocky. These features would indicate that the structures lateral deformations should be less than code standards as compared to taller and thinner structures and therefore the bracing elements would be designed more for compression and buckling.

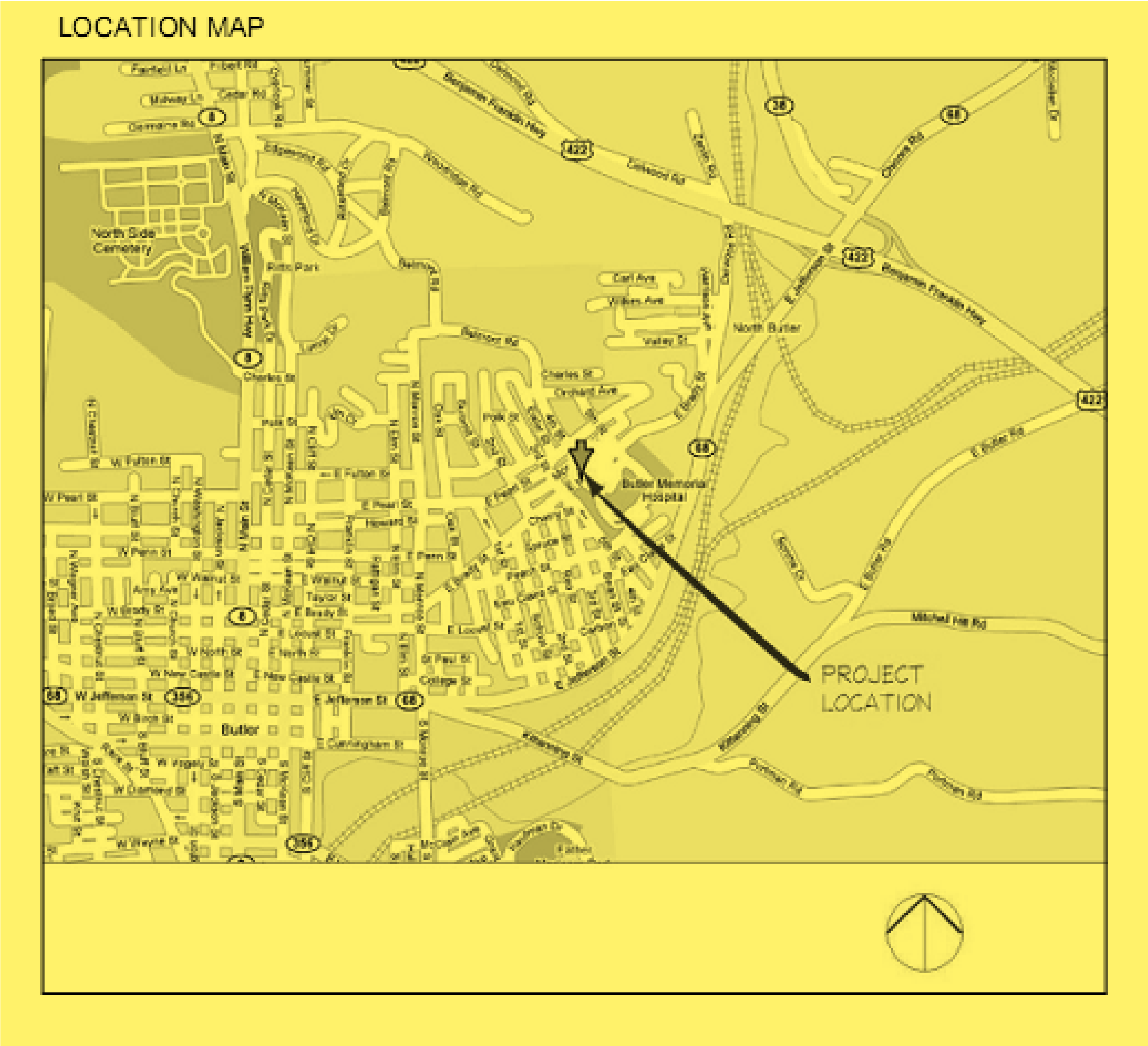
Hand and SAP calculated drift values compiled in Table 3.13 on page 26 for code vs. calculated values shows that the story and total drifts are approximately 3.5-7.5 times less than code standards indicating that a smaller profile could have been used to control building drift.

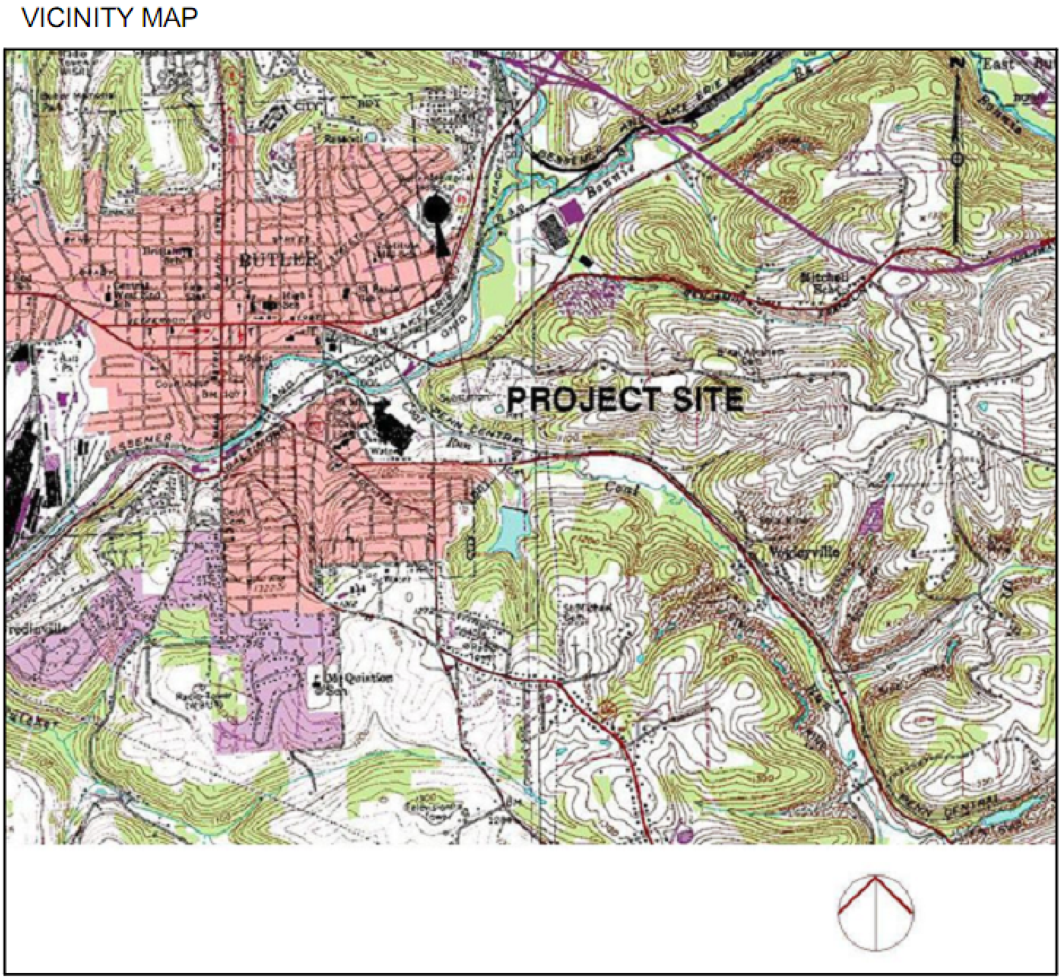
It was also shown that the construction document data for the lateral system and bracing members was oversized as compared to the hand calculations by a factor of 40-50%. The bracing member that was checked shows a service wind load (unfactored) of 130kips on the construction documents while the hand calculated values are 101kips factored.

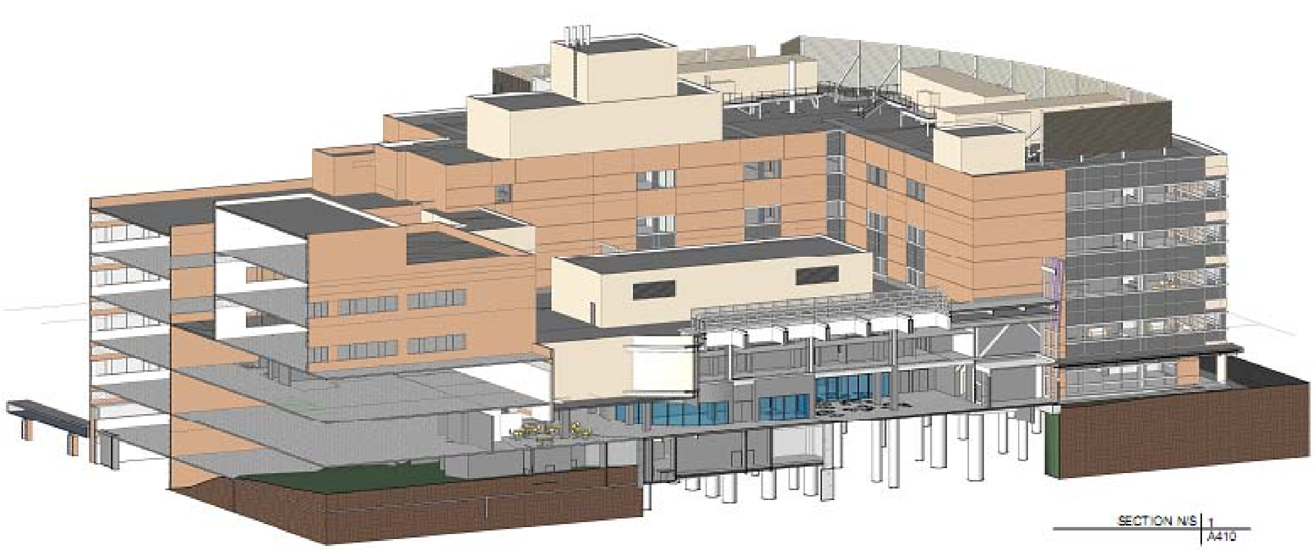
This discrepancy in values and subsequent member sizes could be from multiple reasons. First some of the assumptions and simplifications of the wind values may have been different than design values and led to lower than designed for wind loads. Secondly only one wind combination was checked from ASCE 7-05, a more detailed analysis may have proved that another wind combination might control. Another consideration might be that the designer did not solely use code minimum standards to size the members or increased them for other reasons.

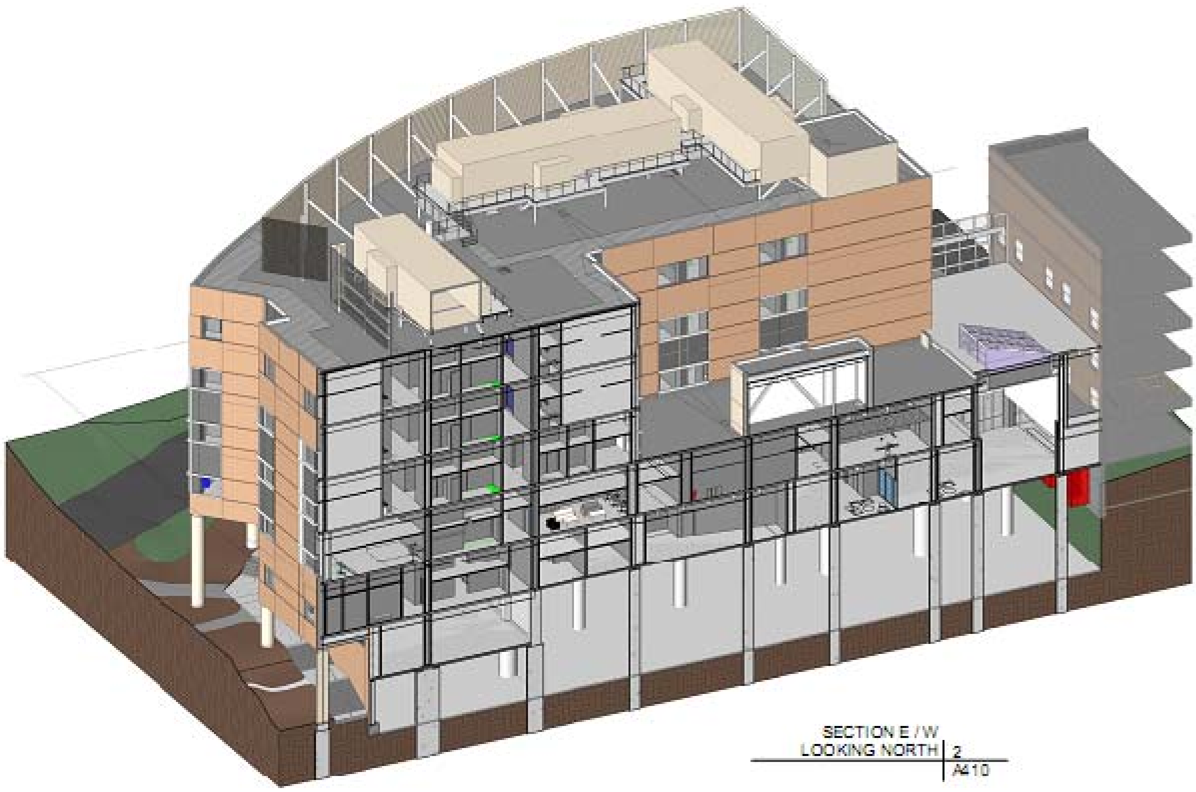
Appendix: A

View looking from magnetic north


LOCATION MAP
North Side Cemetery
Ritts Park
North Butler
Butler Memorial Hospital
Butler
PROJECT LOCATION


VICINITY MAP
PROJECT SITE
BUTLER


SECTION N/S
1
A410



SECTION E / W
LOOKING NORTH 2
A410

MAIN FACADE
NORTHWEST CORNER 3
A400

Appendix B: Wind

Topographic Factor, K_{zt} – Method 2

Figure 6-4 (cont'd)

Equations:

$$K_{zt} = (1 + K_1 K_2 K_3)^2$$

K_1 determined from table below

$$K_2 = \left(1 - \frac{|x|}{\mu L_h}\right)$$

$$K_3 = e^{-\gamma z / L_h}$$

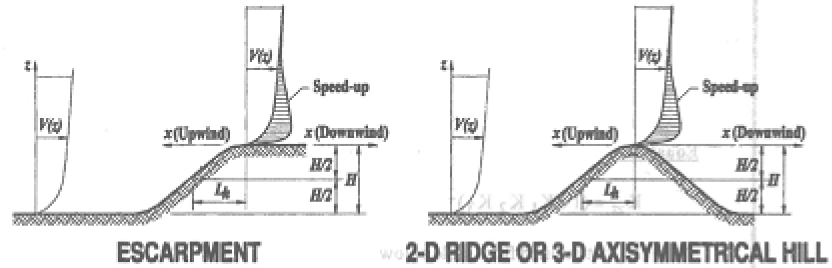
Parameters for Speed-Up Over Hills and Escarpments

Hill Shape	$K_1/(H/L_h)$ Exposure B	C	D	γ	μ Upwind of Crest	Downwind of Crest
2-dimensional ridges (or valleys with negative H in $K_1/(H/L_h)$)	1.30	1.45	1.55	3	1.5	1.5
2-dimensional escarpments	0.75	0.85	0.95	2.5	1.5	4
3-dimensional axisym. hill	0.95	1.05	1.15	4	1.5	1.5

46 ASCE 7-05

Topographic Factor, K_{zt} – Method 2

Figure 6-4



Topographic Multipliers for Exposure C										
	K_1 Multiplier				K_2 Multiplier			K_3 Multiplier		
H/L_h	2-D Ridge	2-D Escarp.	3-D Axisym. Hill	x/L_h	2-D Escarp.	All Other Cases	z/L_h	2-D Ridge	2-D Escarp.	3-D Axisym. Hill
0.20	0.29	0.17	0.21	0.00	1.00	1.00	0.00	1.00	1.00	1.00
0.25	0.36	0.21	0.26	0.50	0.88	0.67	0.10	0.74	0.78	0.67
0.30	0.43	0.26	0.32	1.00	0.75	0.33	0.20	0.55	0.61	0.45
0.35	0.51	0.30	0.37	1.50	0.63	0.00	0.30	0.41	0.47	0.30
0.40	0.58	0.34	0.42	2.00	0.50	0.00	0.40	0.30	0.37	0.20
0.45	0.65	0.38	0.47	2.50	0.38	0.00	0.50	0.22	0.29	0.14
0.50	0.72	0.43	0.53	3.00	0.25	0.00	0.60	0.17	0.22	0.09
				3.50	0.13	0.00	0.70	0.12	0.17	0.06
				4.00	0.00	0.00	0.80	0.09	0.14	0.04
							0.90	0.07	0.11	0.03
							1.00	0.05	0.08	0.02
							1.50	0.01	0.02	0.00
							2.00	0.00	0.00	0.00

Notes:

1. For values of H/L_h, x/L_h and z/L_h other than those shown, linear interpolation is permitted.
2. For $H/L_h > 0.5$, assume $H/L_h = 0.5$ for evaluating K_1 and substitute 2H for L_h for evaluating K_2 and K_3.
3. Multipliers are based on the assumption that wind approaches the hill or escarpment along the direction of maximum slope.
4. Notation:
   - H: Height of hill or escarpment relative to the upwind terrain, in feet (meters).
   - L_h: Distance upwind of crest to where the difference in ground elevation is half the height of hill or escarpment, in feet (meters).
   - K_1: Factor to account for shape of topographic feature and maximum speed-up effect.
   - K_2: Factor to account for reduction in speed-up with distance upwind or downwind of crest.
   - K_3: Factor to account for reduction in speed-up with height above local terrain.
   - x: Distance (upwind or downwind) from the crest to the building site, in feet (meters).
   - z: Height above local ground level, in feet (meters).
   - μ: Horizontal attenuation factor.
   - γ: Height attenuation factor.

JIM ROTUNNO TECH III WIND CALCULATIONS

9-26-09

Gust effect Factor: Rigid Structure

$$G = 0.925\left(\frac{1+1.7\,g_Q I_{\bar{z}} Q}{1+1.7\,g_v I_{\bar{z}}}\right) \Rightarrow I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6}$$

$\bar{z} = 0.6h = 0.6(122') = 73.2' > 30' = \bar{z}_{min}$

$c = 0.20$ (Table 6-2)

$$I_{\bar{z}} = 0.2\left(\frac{33}{73.2}\right)^{1/6}$$

$0.2(0.876) = 0.1752$

$g_Q = 3.4$, $g_v = 3.5$ } 6.5.8.1

$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}}$$

B = horizontal dimension normal to wind direction

E-W 198'

N-S 210'

$h = 122'$

$$L_{\bar{z}} = l\left(\frac{\bar{z}}{70}\right)^{\bar{\epsilon}} \Rightarrow \bar{z} = 73.2$$

$= 500\left(\frac{73.2}{70}\right)^{1/5}$ $\quad \bar{\epsilon} = 1/5.0$ } Table 6-2

$= 744.5'$ $\quad l = 500$

$$(N\text{-}S)\text{wind}\quad Q = \sqrt{\frac{1}{1+0.63\left(\frac{198+122}{744.5}\right)^{0.63}}} = \sqrt{\frac{1}{1.372}}$$

$= 0.854$

$$(E\text{-}W)\text{wind}\quad Q = \sqrt{\frac{1}{1+0.63\left(\frac{210+122}{744.5}\right)^{0.63}}} = \sqrt{\frac{1}{1.3788}}$$

$= 0.851$

(E-W)

$$G = 0.925\left(\frac{1+1.7(3.4)(0.1752)(0.851)}{1+1.7(3.4)(0.1752)}\right)$$

$= 0.856$

(N-S)

$$G = 0.925\left(\frac{1+1.7(3.4)(0.1752)(0.854)}{1+1.7(3.4)(0.1752)}\right)$$

$= 0.857$

TECH III   Wind Calculations

ASCE 7-05 §6.5.12.2.1 MWFRS   11-13-09

Velocity pressure (q_z) evaluated @ height z ⇒ height above ground

$q_z = 0.00256\, K_z\, K_{zt}\, K_d\, V^2\, I$ units (lb/ft^2)

K_z varies Table 6-3

K_{zt} varies Figure 6-4

$K_d = 0.85$

$V^2 = 90^2$

$I = 1.15$

Exposure B   CASE I

$K_{zt} = (1 + K_1 K_2 K_3)^2$

K_{z1}	@ 14'-8"	= 0.85	K_{zt1}	= 1.62	from documents
K_{z2}	29'-4"	0.975	K_{zt2}	= 1.552	↑
K_{z3}	44'-0"	1.06	K_{zt3}	= 1.484	
K_{z5}	58'-8"	1.125	K_{zt5}	= 1.415	linear interpolation
K_{z6}	73'-4"	1.183	K_{zt6}	= 1.347	
K_{z7}	88'-0"	1.234	K_{zt7}	= 1.279	
K_{z8}	102'-8"	1.267	K_{zt8}	= 1.217	
K_{z9}	121'-0"	1.315	K_{zt9}	= 1.141	↓
K_{z10}	135'-0"	1.348	K_{zt10}	= 1.09	from documents

$K_1 = 0.75/(H/L_h)$ $H/L_h \leq 0.5$

$K_2 = \left(1 - \frac{|x|}{\mu L_h}\right)$

$K_3 = e^{-\gamma z/L_h}$

$\gamma = 2.5$ $\mu = 4$

$q_{z1} = 0.00256(0.85)(1.62)(0.85)(90^2)(1.15) = 27.91$

$_2 = 0.00256(0.975)(1.552)(0.85)(90^2)(1.15) = 30.67$

$_3 = 0.00256(1.06)(1.484)(0.85)(90^2)(1.15) = 31.88$

$_5 = 0.00256(1.125)(1.415)(0.85)(90^2)(1.15) = 32.27$

$_6 = 0.00256(1.183)(1.347)(0.85)(90^2)(1.15) = 32.3$

$_7 = 0.00256(1.234)(1.279)(0.85)(90^2)(1.15) = 31.99$

$_8 = 0.00256(1.267)(1.217)(0.85)(90^2)(1.15) = 31.25$

$_9 = 0.00256(1.315)(1.141)(0.85)(90^2)(1.15) = 30.41$

$_{10} = 0.00256(1.348)(1.09)(0.85)(90^2)(1.15) = 29.78$

TECH III

Wind Calculations
11-13-09

$P_1 = q z_i \, G C_p - q_i (G C_{pi})$ lb/ft²

$= q_i = 30.41$

$G = 0.856$ $(G C_{pi}) = \pm 0.18$

$C_p = 0.8$

P (East - west) windward side

$P_1 = 27.91(0.856)(0.8) - 30.41(-0.18) = 24.59$ lb/ft²

$_2 = 30.67(0.856)(0.8) - 30.41(-0.18) = 26.48$

$_3 = 31.88(0.856)(0.8) + 5.474 = 27.30$

$_5 = 32.27(0.856)(0.8) + 5.474 = 27.57$

$_6 = 32.3(0.856)(0.8) + 5.474 = 27.59$

$_7 = 31.99(0.856)(0.8) + 5.474 = 27.38$

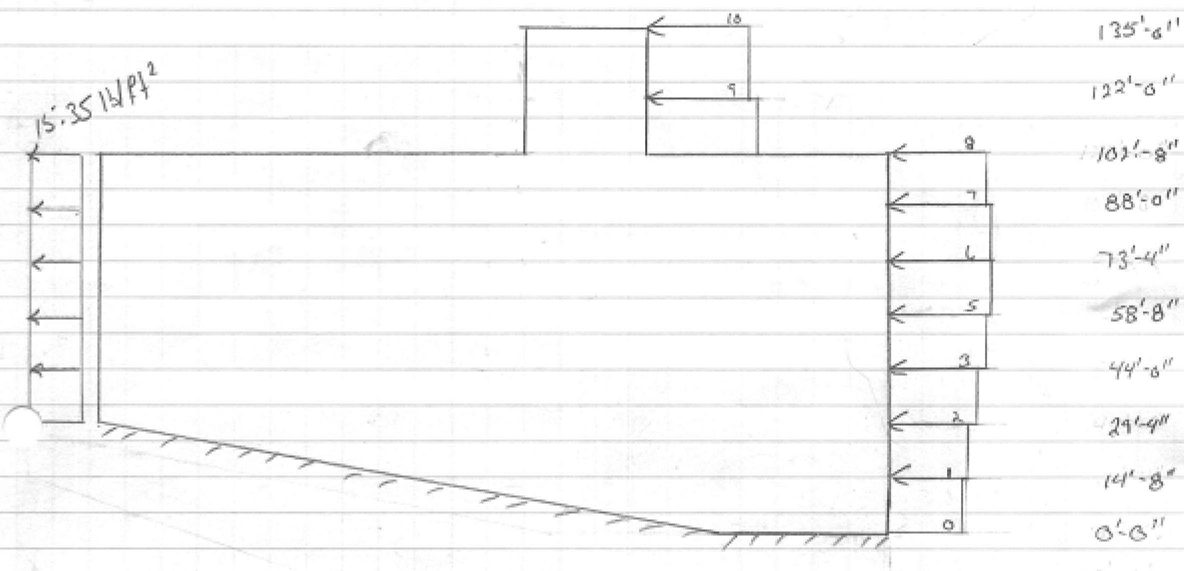
$_8 = 31.25(0.856)(0.8) + 5.474 = 26.87$

$_9 = 30.41(0.856)(0.8) + 5.474 = 26.30$

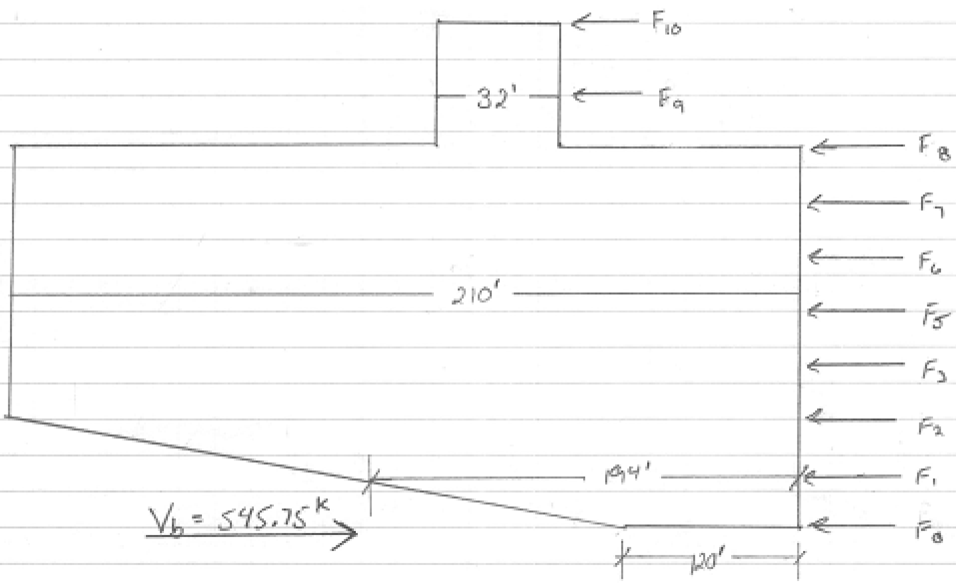
$_{10} = 29.78(0.856)(0.8) + 5.474 = 25.87$

Leeward Side

$P = 30.41(0.856)(0.8) - 30.41(0.18) = 15.35$ lb/ft²



TECH III Wind Calculations
11-13-09



WEST ELEVATION

$F_0 = \frac{1}{2}(14.67')(120')(24.59) = 21.644\ k$

$F_1 = 7.33'(120')(24.59) + 7.33'(157')(26.48) = 52.10\ k$

$F_2 = 7.33'(157')(26.48) + 7.33(199)(27.36) = 69.29\ k$

$F_3 = 7.33(194)(27.30) + 7.33(210)(27.57) = 81.26\ k$

$F_5 = 7.33(210)(27.57) + 7.33(210)(27.59) = 84.91\ k$

$F_6 = 7.33(210)(27.59) + 7.33(210)(27.38) = 84.64\ k$

$F_7 = 7.33(210)(27.38) + 7.33(210)(26.87) = 83.51\ k$

$F_8 = 7.33(210)(26.87) + \frac{19.33}{2}(32')(26.30) = 49.5\ k$

$F_9 = \frac{19.33}{2}(32')(26.3) + \frac{13}{2}(32)(25.87) = 13.52\ k$

$F_{10} = \frac{13}{2}(32)(25.87) = 5.38\ k$

Total = 545.75

TECH III

Wind Calculations
11-13-09

North - South

Same K_z, K_{zt}, k_d V^2 + I as East - West

$q_h = 30.41$

$(GC_{pi}) = -0.18$

$G = 0.857$

q_{zi} is the same as E-W

$P_i = q_{zi}(0.857)(0.8) - 30.41(\pm 0.18)$

$P_1 = 27.91(0.6856) + 5.4738 = 24.60$ lb/ft^2

$P_2 = 30.67(0.6856) + 5.4738 = 26.61$

$P_3 = 31.88 \qquad = 27.33$

$P_5 = 32.27 \qquad = 27.61$

$P_6 = 32.3 \qquad = 27.63$

$P_7 = 32.0 \qquad = 27.43$

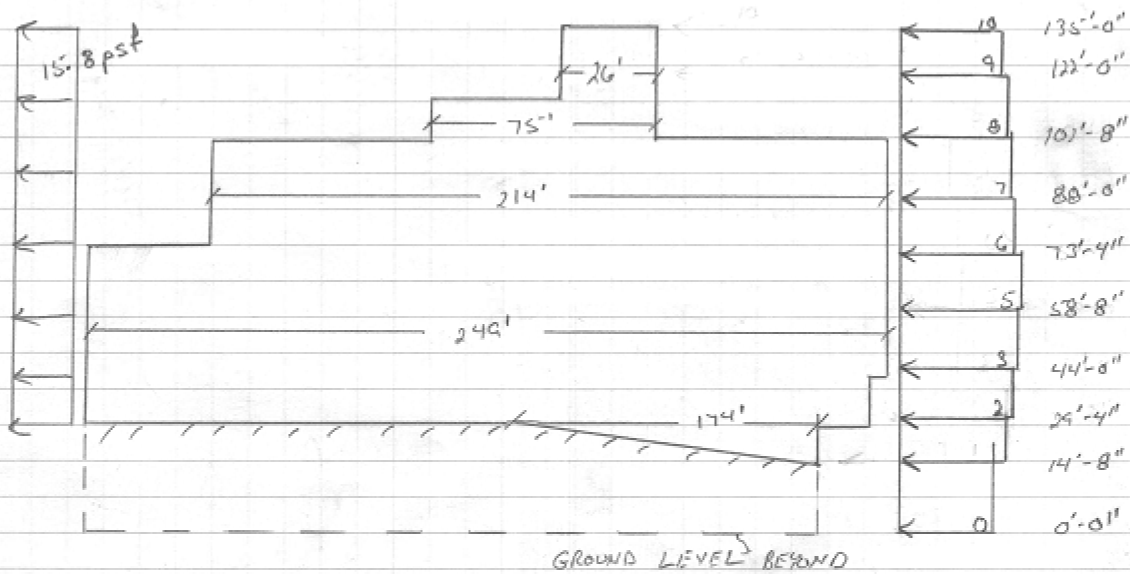
$P_8 = 31.25 \qquad = 26.91$

$P_9 = 30.41 \qquad = 26.34$

$P_{10} = 29.78 \qquad = 25.90$

Leeward side

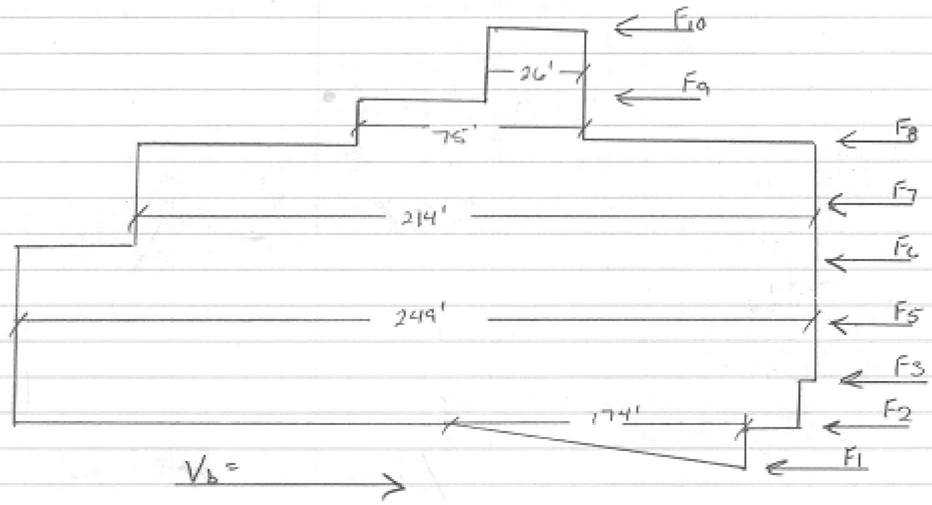
$P = 30.41(0.857)(0.8) - 30.41(\pm 0.18) = 15.8$ lb/ft^2



TECH III Wind Calculations

11-13-09

North - South



$F_0 = 0$

$F_1 = \frac{1}{2}(174')(\frac{1}{2})(14.67')(24.60) = 15.69\text{ kips}$

$F_2 = 7.33'(\frac{3}{4})(174')(24.60) + 7.33'(249)(26.61) = 72.10$

$F_3 = 7.33(249)(26.61) + 7.33(249)(27.33) = 98.45$

$F_5 = 7.33(249)(27.33) + 7.33(249)(27.61) = 100.27$

$F_6 = 7.33(249)(27.61) + 7.33(214)(27.63) = 93.73$

$F_7 = 7.33(214)(27.63) + 7.33(214)(27.43) = 86.37$

$F_8 = 7.33(214)(27.43) + \frac{19.33}{2}(75')(26.91) = 62.53$

$F_9 = \frac{19.33}{2}(75')(26.91) + \frac{13}{2}(26')(26.34) = 23.96$

$F_{10} = \frac{13}{2}(26')(26.34) = 4.45$

Appendix C: Snow

JIM ROTUNNO TECH I + III SNOW LOAD CALCULATIONS

USING ASCE 7-05 CHAPTER 7

Flat roof snow load (p_f)

$p_f = 0.7\, C_e\, C_t\, I\, p_g$

p_g ⇒ Butler, PA Figure 7-1 = ground snow load
= 25 psf

C_e ⇒ Table 7-2 terrain category C roof partially exposed
= 1.0

C_t ⇒ Table 7-3
= 1.0

I = Table 7-4 Category IV
= 1.2

$p_f = 0.7(1.0)(1.0)(1.2)(25) = 21$ psf

7.3 where p_g exceeds 20 lb/ft²

$p_f = 20I$

$= \boxed{24 \text{ psf}}$ not 21 psf

Appendix D: Seismic calculations

JIM ROTUNNO TECH I + III SEISMIC ANALYSIS

Occupancy Category IV

Determine the design spectral responce acceleration

$S_{DS} = \frac{2}{3} S_{MS}$

$S_{MS} = F_a S_S \Rightarrow F_a$ = site coefficient Table 11.4.1

Site Class C

$S_S = .12 \quad S_1 = 0.046 \Rightarrow 0.0055$

$F_a = 1.2$

$S_{MS} = 1.2(0.12) = 0.144$

$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3}(0.144) = 0.096$

$S_{D1} = \frac{2}{3} S_{M1} \Rightarrow S_{M1} = F_v S_1$

$= 1.7(0.046) = 0.0782$

$S_{D1} = \frac{2}{3}(0.0782) = 0.0521$

Importance Factor 1.5

SDC ⇒ Seismic design category = A ⇒ Table 11.6.1 occupancy IV

$S_{DS} < 0.167$

From Table 11.6.2

occupancy IV

$S_{D1} < 0.067 \therefore A$

Calculate the seismic base shear

$V = C_S W \qquad W$ = Total dead load for seismic load determination

C_S = Seismic Responce coefficient ASCE 7-05 §12.8.1

$= S_{DS} / {}^{R}/_{I}$

$= 0.096 / {}^{3.25}/_{1.5} = 0.0443$ and $\leq {}^{S_{D1}}/_{T(R/I)}$ for $T \leq T_L$

$T: T_a = C_t h_n^x$ eq. 12.8.7

T_L = § 11.4.5 Fig 22-15

$= 12$

$T = 0.02(135 ft)^{0.75} = 0.792 sec$

$S_{D1}/T(R/I) = 0.0782 / 0.792({}^{3.25}/_{1.5})$

$C_S = 0.0456$

JIM ROTUNNO TECH I & III SEISMIC ANALYSIS

Effective seismic weight W_T as defined 12.7.2

1) 25% LL for for storage areas
2) partitions - minimum of 10 psf
3) total operating weight of permanent equipment use 10psf
4) where the flat roof snow load exceeds 30psf use 20% $p_f < 30$ ∴ Not required
plus the total Dead load

51psf weight of concrete slab + metal deck concrete 3½" ↕ metal 3" ↕

$$115\text{ pcf}\left(\frac{3.5 + 3/2}{12''}\right) + 3\text{ psf} = 50.92\text{ lb/ft}^2 \quad \text{use } \underline{51\text{ psf}}$$

Superimposed Dead Loads ⇒ MEP, partitions, finishes

25psf use 25 psf

289.16 kips columns from column schedule (tabulated using excel)

beams (see next page)

1.0psf operating weight of permanent equipment - 1.0psf of total building area

42psf roof - concrete deck, insulation, EPDM - 42 psf

building square footage from construction documents

L_1 = 20 405 sq. ft
L_2 = 45 545 sq. ft
L_3 = 42 165 sq. ft
L_5 = 31 525 sq. ft
L_6 = 27 720 sq. ft
L_7 = 27 760 sq. ft
Roof ⇒ L_8 = 46000

Exterior wall weight ⇒ brick veneer 120 lb/ft^3

$$120\frac{\text{lb}}{\text{ft}^3}\left(\frac{3.25''}{12''}\right) = 32.5\text{ lb/ft}^2$$

Story height = 14'-8"

JIM ROTUNNO TECH I + III SEISMIC CALCULATIONS

Level by Level weight analysis (Example)

concrete MEP Equip. columns beams brick facade

$W_1 = 51(20\,405) + 25(20\,405) + 1.0(20\,405) + 58.39 + 141.84 + 4.86$

$= 1772.4\ k$

exterior walls are estimated @ 20% glass + 80% brick

average curtain wall weighs 12.5-15 lb/ft² use 15 psf

$0.20(15)$(sq. ft. of wall per floor) $+ 0.80(32.5)$(sq ft. of ext. wall per floor) =

lb of exterior wall per level ⇒ divide this by the level's square footage to get a lb/sq. ft per level then add into excel spreadsheet

$0.20(15) + 0.80(32) = 28.6\ lb/ft^2$ of exterior wall

Beams

weight of beams @ each floor level is tabulated by taking (3) spot checks of average bays and calculating the % steel per floor area and superimposing it to all floors

spot check 1: @ level 3

Typical bay = 30' × 28' = 840 sq. ft

⇒ 4 - 18×40-30' 1. 21×50-28' ⇒ only 1 girder per bay because they are shared between bays

$4(40)(30) + 1(50)(28) = 6200\ lb$

$6200\ lb / 840\ sq.\ ft = 7.38\ lb/ft^2$

2: Typical bay level 2 30'×30' = 900 sq. ft

4 - 16×26 1-24×55

$4(26)(30) + 1(55)(30) = 4770$

$4770/900 = 5.3\ lb/ft^2$

3: Typical bay level 5 30×28 = 840 sq. ft

5 - 18×40-30' 1-24×62-28'

$5(40)(30) + 1(62)(28) = 7736\ lb$

$7736\ lb / 840 = 9.2\ lb/ft^2$

$(7.38 + 5.3 + 9.2)/3 = 7.3$ average

USE $7.0\ lb/ft^2$

JIM ROTUNNO TECH I + III SEISMIC CALCULATIONS

Distributing the total base shear to individual levels
The roof square footage needs to be divide up between levels 3, 5, 7, + 8

3 = 45 545 - 31 525 = 14 020 14020/45545 = 31%
5 = 31 525 - 27760 = 3765 3765/45545 = 8%
7 = 27760 - 2060 = 25700 25700/45545 = 56%
8 = 45545 - 14020 - 3765 - 25700 = 2060 2060/45545 = 5%

From excel take the floor weight totals (summed across) and add the % of roof weight to levels 3 5 + 7 to get total floor level weights

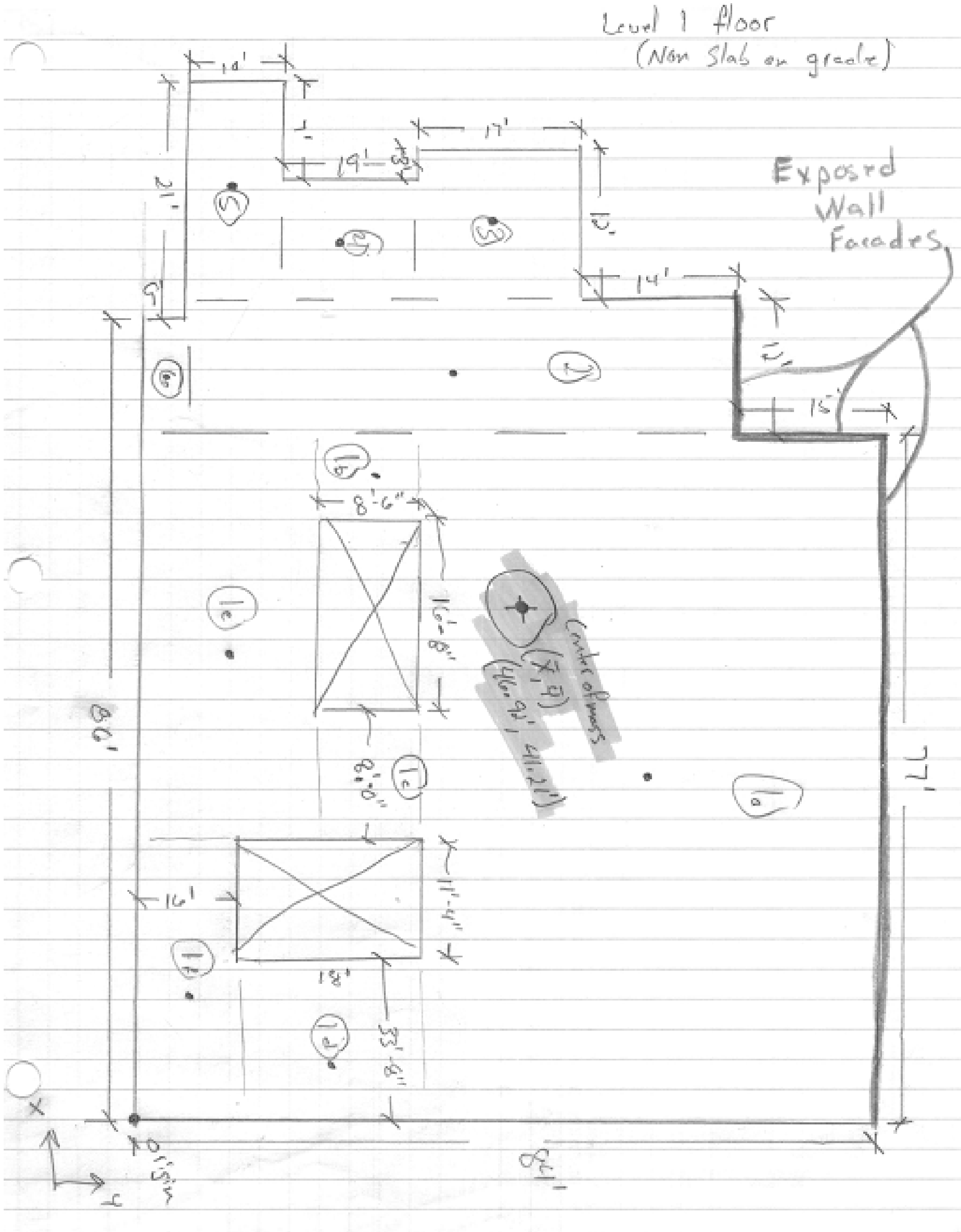
1 = 1772.4k
2 = 3876.4k
3 = 3610.9 + 0.31(1913) = 4203.93
5 = 2689.9 + 0.08(1913) = 2842.94
6 = 2368.0k
7 = 2361.7 + 0.56(1913) = 3432.98
8 = 0.5(1913) = 95.65k
Total = 18676.1 kips

USE THESE LEVEL weights to estimate the % of Total Base Shear that acts at each level.

1 ⇒ 1772.4/22616.5 = 7.84% ⇒ 0.0784(851.58) = 66.76 k
2 ⇒ 3876.4/22616.5 = 17.14% ⇒ 0.1714(851.58) = 145.96 k
3 ⇒ 4203.93/22616.5 = 18.59% ⇒ 0.1859(851.58) = 158.31 k
5 ⇒ 2842.94/22616.5 = 12.57% ⇒ 0.1257(851.58) = 107.04 k
6 ⇒ 2368.0/22616.5 = 10.47% ⇒ 0.1047(851.58) = 89.16 k
7 ⇒ 3432.98/22616.5 = 15.18% ⇒ 0.1518(851.58) = 129.27 k
8 ⇒ 95.65/22616.5 = 0.423% ⇒ 0.00423(851.58) = 3.6 k

		Column Load Summary												
W Shapes	**weight**							**Levels**						
14.667	lbs													
			1		**2**		**3**		**5**		**6**		**7**	
8x40	40	1	0.587	0	0.000	0	0.000	0	0.000	0	0.000	0	0.000	
12x40	40	1	0.587	2	1.173	2	1.173	2	1.173	1	0.587	0	0.000	
12x45	45	0	0.000	2	1.320	2	1.320	0	0.000	0	0.000	0	0.000	
12x50	50	1	0.733	2	1.467	2	1.467	1	0.733	1	0.733	0	0.000	
12x53	53	3	2.332	3	2.332	3	2.332	0	0.000	0	0.000	0	0.000	
12x58	58	0	0.000	0	0.000	0	0.000	2	1.701	2	1.701	0	0.000	
12x65	65	2	1.907	1	0.953	1	0.953	0	0.000	0	0.000	0	0.000	
12x72	72	1	1.056	1	1.056	1	1.056	0	0.000	0	0.000	0	0.000	
12x87	87	0	0.000	1	1.276	1	1.276	0	0.000	0	0.000	0	0.000	
12x96	96	2	2.816	1	1.408	1	1.408	0	0.000	0	0.000	0	0.000	
14x43	43	0	0.000	0	0.000	0	0.000	1	0.631	0	0.000	9	5.676	
14x48	48	0	0.000	0	0.000	0	0.000	3	2.112	4	2.816	11	7.744	
14x53	53	0	0.000	0	0.000	0	0.000	3	2.332	4	3.109	5	3.887	
14x61	61	2	1.789	0	0.000	3	2.684	17	15.210	17	15.210	8	7.157	
14x68	68	2	1.995	0	0.000	2	1.995	10	9.974	9	8.976	3	2.992	
14x74	74	4	4.341	0	0.000	4	4.341	1	1.085	1	1.085	0	0.000	
14x82	82	7	8.419	0	0.000	9	10.824	0	0.000	0	0.000	2	2.405	
14x90	90	7	9.240	18	23.761	18	23.761	4	5.280	4	5.280	0	0.000	
14x99	99	0	0.000	1	1.452	1	1.452	0	0.000	0	0.000	0	0.000	
14x109	109	4	6.395	9	14.388	7	11.191	1	1.599	0	0.000	0	0.000	
14x120	120	3	5.280	0	0.000	1	1.760	0	0.000	0	0.000	0	0.000	
14x132	132	2	3.872	0	0.000	0	0.000	0	0.000	0	0.000	0	0.000	
14x145	145	1	2.127	0	0.000	0	0.000	0	0.000	0	0.000	0	0.000	
14x159	159	1	2.332	0	0.000	0	0.000	0	0.000	0	0.000	0	0.000	
14x176	176	1	2.581	0	0.000	0	0.000	0	0.000	0	0.000		0.000	
			58.389	kips	50.586	kips	68.994	kips	41.830	kips	39.498	kips	29.862	kips
		Total column weight=			289.160	kips								

Appendix E: Frame Stiffness and Load Distribution Calculations



Level 1 TECH III JIM ROTUNNO

Area and Center of Mass Calculation 11-14-09

$\bar{y}$ or $\bar{x}$ $= \Sigma \bar{y}_i (A_i) / \Sigma A_i$

$\bar{y}_{1a} = 16' + 18' + 50'/2 = 59'$

$A_{1a} = (50')(77') = 3850$

$\bar{y}_{1b} = 29.75'$

$A_{1b} = 7.333'(8.5') = 62.334$

$\bar{y}_{1c} = 29.75'$

$A_{1c} = 8'(8.5') = 68$

$\bar{y}_{1d} = 25'$

$A_{1d} = 18'(33.67') = 606.06$

$\bar{y}_{1e} = 12.75'$

$A_{1e} = (32)(25.5') = 816$

$\bar{y}_{1f} = 8'$

$A_{1f} = 16'(45') = 720$

$\bar{y}_2 = 9 + 30 = 39'$

$A_2 = 12'(60) = 720$

$\bar{y}_3 = 9+10+19+17/2 = 46.5$

$A_3 = 12(17) = 204$

$\bar{y}_4 = 9+10+19/2 = 28.5$

$A_4 = 9(19) = 171$

$\bar{y}_5 = 9 + 10/2 = 14$

$A_5 = 10(16) = 160$

$\bar{y}_6 = 4.5'$

$A_6 = 9(9) = 81$

$$\bar{y} = \frac{307386.937}{7458.394} = 41.21'$$

$\bar{x}_{1a} = 77/2 = 38.5$

$A_{1a} = 77(50) = 3850$

$\bar{x}_{1b} = 33.67 + 11.33 + 8 + 16.67 + 7.333/2 = 73.335'$

$\bar{x}_{1c} = 33.67 + 11.33 + 8/2 = 49'$

$\bar{x}_{1d} = 33.67/2 = 16.835'$

$\bar{x}_{1e} = 45 + 32/2 = 61'$

$\bar{x}_{1f} = 45/2 = 22.5'$

$\bar{x}_2 = 77 + 12/2 = 83'$

$\bar{x}_3 = 77 + 12 + 12/2 = 95'$

$\bar{x}_4 = 77 + 12 + 9/2 = 93.5'$

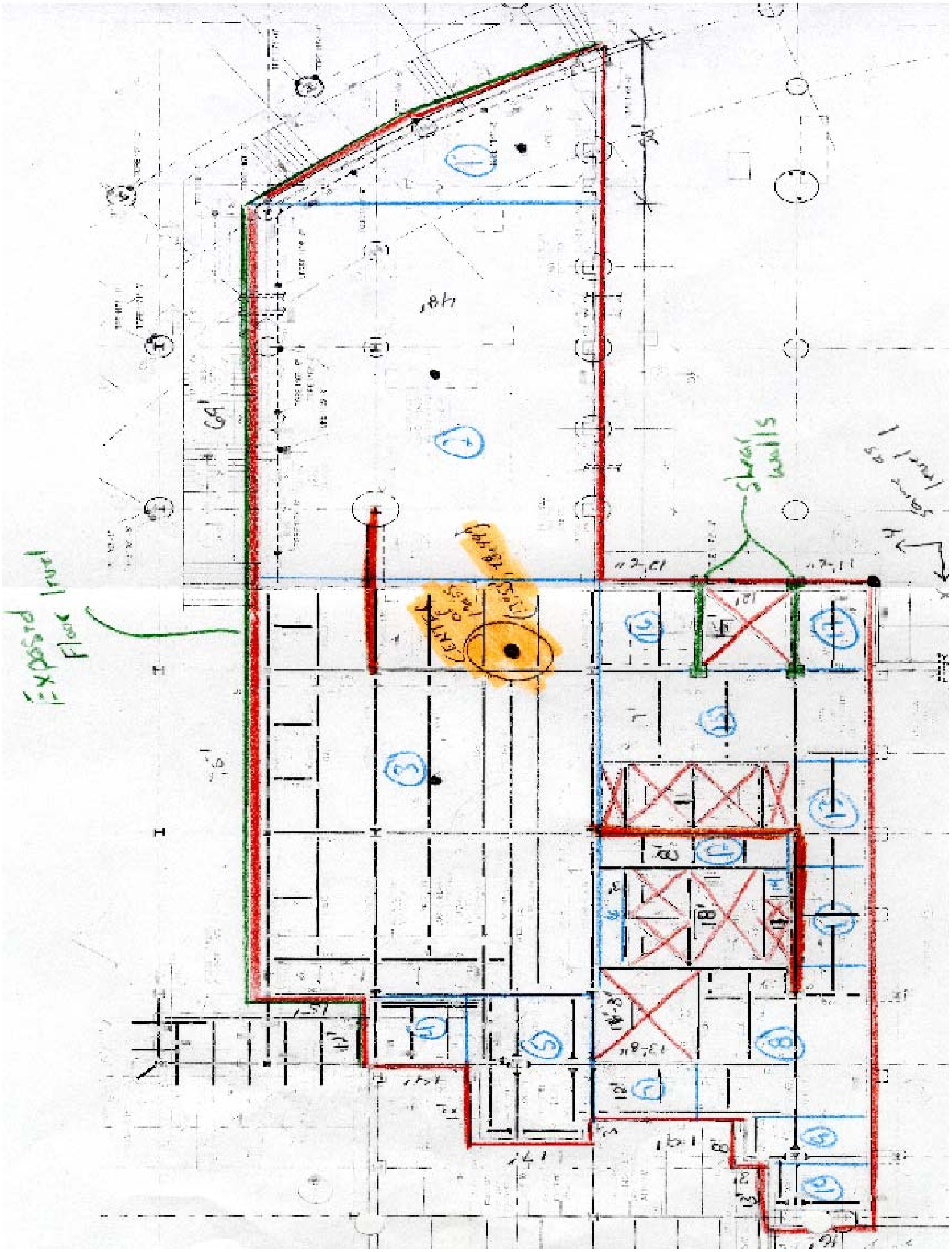
$\bar{x}_5 = 77 + 12 + 21/2 = 99.5'$

$\bar{x}_6 = 86' - 9/2 = 81.5'$

$$\bar{x} = \frac{349957.284}{7458.394} = 46.92'$$

CENTER OF MASS

(46.92', 41.21')


Exposed Floor level
Shear walls

CENTER OF MASS CALC. TECH III LEVEL 2

$\bar{y}_1 \approx 11.5' + 12' + 12.5' + \frac{1}{2}(48)' = 52'$

$A_1 \approx \frac{1}{2}(48)(30) = 720$

$\bar{y}_2 = 11.5 + 12 + 12.5 + \frac{48}{2} = 60'$

$A_2 = 48(69) = 3312$

$\bar{y}_3 = \bar{y}_2 = 60'$

$A_3 = 75(48) = 3600$

$\bar{y}_4 = 84 - 15 - \frac{14}{2} = 62'$

$A_4 = 12(14) = 168$

$\bar{y}_5 = 84 - 29 - \frac{17}{2} = 46.5'$

$A_5 = 17(24) = 408$

$\bar{y}_6 = 84 - 48 - \frac{3}{2} = 34.5'$

$A_6 = 3(18) = 54$

$\bar{y}_7 = 84 - 48 - \frac{13.6}{2} = 29.167'$

$A_7 = 164$

$\bar{y}_8 = (84 - 29 - 30.67)/2 = 12.167'$

$A_8 = (12.167/2)(28.67) = 349.79$

$\bar{y}_9 = (16+3)/2 = 9.5'$

$A_9 = 19(8) = 152$

$\bar{y}_{10} = 8'$

$A_{10} = 13(16) = 208$

$\bar{y}_{11} = \frac{11.5}{2} = 5.75'$

$A_{11} = 11.5(18) = 207$

$\bar{y}_{12} = 11.5 + \frac{24.5}{2} = 23.75'$

$A_{12} = 8(24.5) = 196$

$\bar{y}_{13} = 5.75'$

$A_{13} = 11.5(8+11) = 218.5$

$\bar{y}_{14} = 11.5 + \frac{3}{2} = 13'$

$A_{14} = 5(3) = 15$

$\bar{y}_{15} = \frac{36}{2} = 18'$

$A_{15} = 17(36) = 612$

$\bar{y}_{16} = 11.5 + 12 + \frac{12.5}{2} = 29.75'$

$A_{16} = 12.5(15) = 187.5$

$\bar{y}_{17} = 5.75'$

$A_{17} \; 11.5(15) = 172.5$

$$\bar{y} = \frac{520446.9079}{10744.29} = 48.44'$$

$\bar{x}_1 = -69 - 10 = -79'$

$\bar{x}_2 = -\frac{69}{2} = -34.5'$

$\bar{x}_3 = \frac{75}{2} = 37.5'$

$\bar{x}_4 = 75 + \frac{12}{2} = 81'$

$\bar{x}_5 = 75 + 12 = 87'$

$\bar{x}_6 = 15 + 17 + 11 + 8 + \frac{18}{2} = 60'$

$\bar{x}_7 = 75 + 24 - 3 - \frac{12}{2} = 90'$

$\bar{x}_8 = 75 + 24 - 3 - ((12 + 16.67)/2) = 81.67'$

$\bar{x}_9 = 75 + 24 + 5 - 4 = 100'$

$\bar{x}_{10} = \bar{x}_9 + 4 + \frac{13}{2} = 110.5'$

$\bar{x}_{11} = 15 + 17 + 11 + 8 + \frac{18}{2} = 60'$

$\bar{x}_{12} = 15 + 17 + 11 + 4 = 47'$

$\bar{x}_{13} = 15 + 17 + \frac{11}{2} = 37.5'$

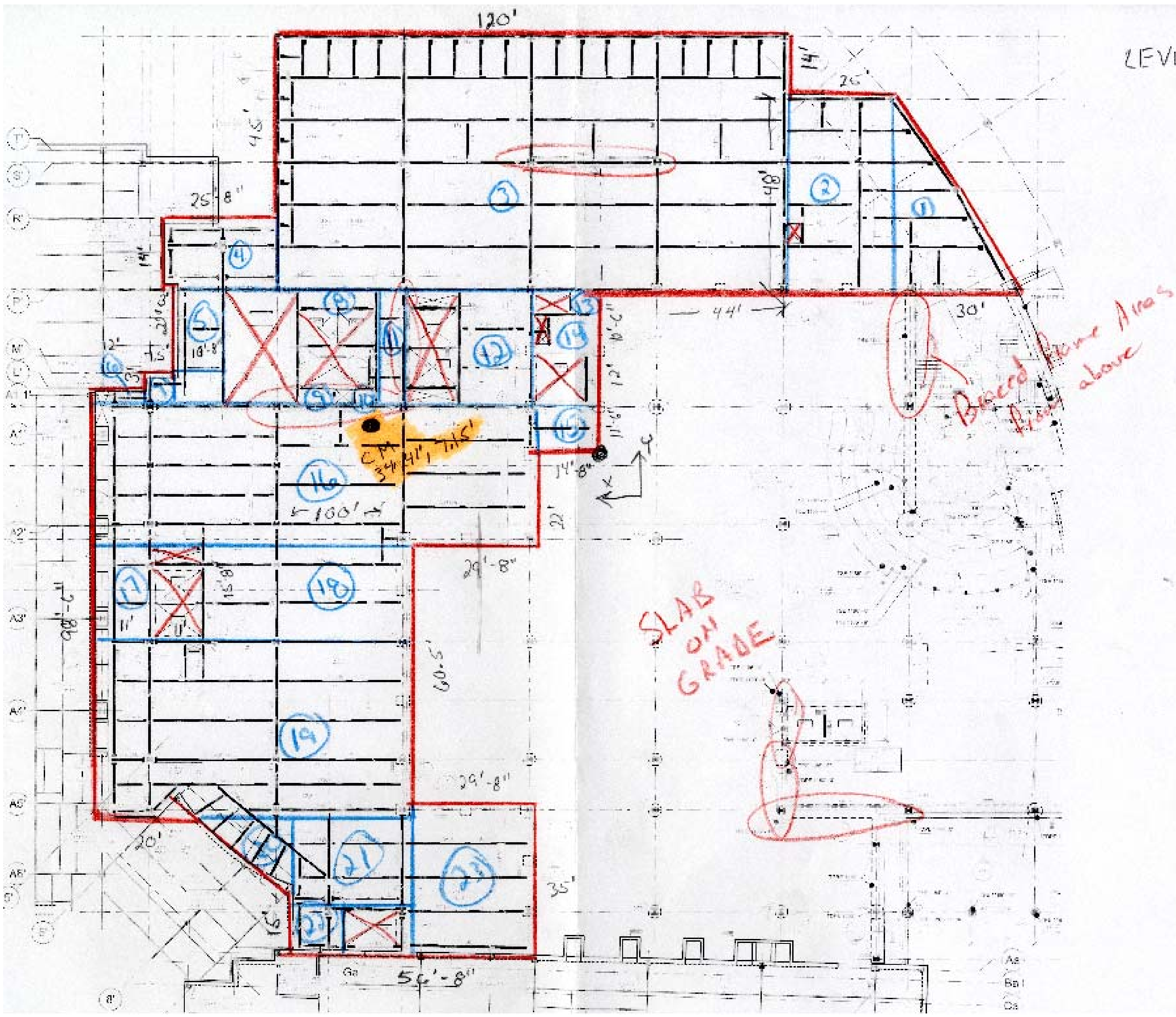
$\bar{x}_{14} = 15 + 17 + 11 + 8 + \frac{5}{2} = 53.5'$

$\bar{x}_{15} = 15 + \frac{17}{2} = 23.5'$

$\bar{x}_{16} = \frac{15}{2} = 7.5'$

$\bar{x}_{17} = \frac{15}{2} = 7.5'$

$$\bar{x} = \frac{145421.5993}{10744.29} = 13.53'$$


120'
LEVE
SLAB ON GRADE
Braced Frame Areas floor above
100'
60.5'
56'-8"

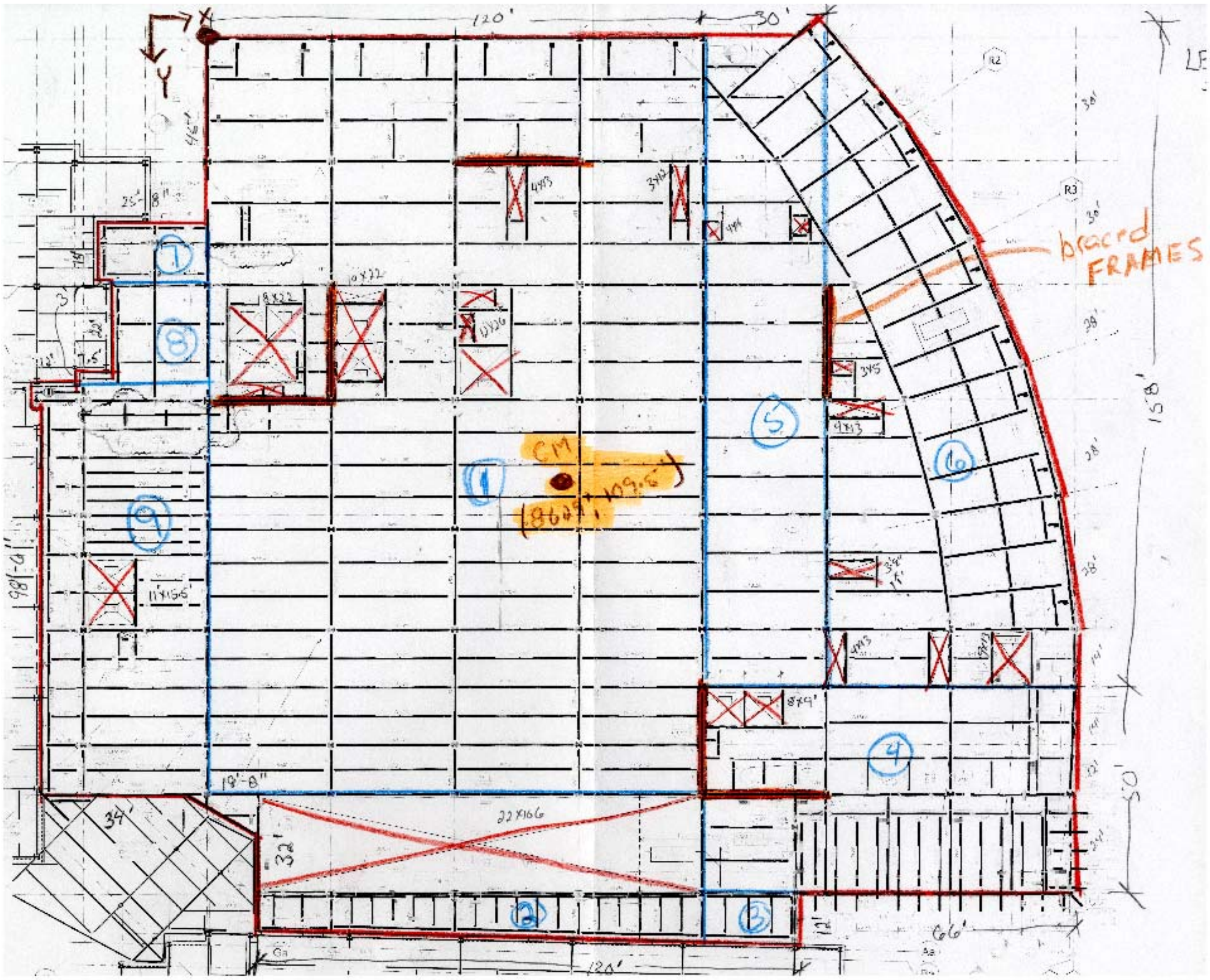
LEVEL 3 TECH III 11-14-09

CENTER OF MASS CALC.

$\bar{y}_1 = 52'$	$A_1 = 720$	$\bar{x}_1 = -79'$
$\bar{y}_2 = 60'$	$A_2 = 1200$	$\bar{x}_2 = -44' - 12.5' = -56.5'$
$\bar{y}_3 = 60'$	$A_3 = 7440$	$\bar{x}_3 = 120/2 - 44 = 16'$
$\bar{y}_4 = 7'$	$A_4 = 359.324$	$\bar{x}_4 = 60 + 16 + 25.67/2 = 88.83'$
$\bar{y}_5 = 11.5 + 12.25 = 23.75'$	$A_5 = 261.332$	$\bar{x}_5 = 89.33'$
$\bar{y}_6 = 1.5'$	$A_6 = 36$	$\bar{x}_6 = 89.33 + 10.67/2 + 7.5 + 6 = 108.16'$
$\bar{y}_7 = 3'$	$A_7 = 22.5$	$\bar{x}_7 = 108.16 - 6 - 7.5/2 = 98.413'$
$\bar{y}_8 = 34.5'$	$A_8 = 54$	$\bar{x}_8 = 60'$
$\bar{y}_9 = 1.5'$	$A_9 = 13.5$	$\bar{x}_9 = 62.5'$
$\bar{y}_{10} = 27.125'$	$A_{10} = 13.5$	$\bar{x}_{10} = 51'$
$\bar{y}_{11} = 23.75'$	$A_{11} = 207$	$\bar{x}_{11} = 23.75'$
$\bar{y}_{12} = 18'$	$A_{12} = 416.5$	$\bar{x}_{12} = 23.75'$
$\bar{y}_{13} = 34.5'$	$A_{13} = 20$	$\bar{x}_{13} = 3.33'$
$\bar{y}_{14} = 28.25'$	$A_{14} = 112.78$	$\bar{x}_{14} = 5.833'$
$\bar{y}_{15} = 5.75$	$A_{15} = 168.67$	$\bar{x}_{15} = 7.33'$
$\bar{y}_{16} = (33.5/2) - 11.5 = -5.25'$	$A_{16} = 3350$	$\bar{x}_{16} = 64.67'$
$\bar{y}_{17} = -22 - 15.67/2 = -29.83'$	$A_{17} = 172.33$	$\bar{x}_{17} = 114.67 - 11/2 = 109.17'$
$\bar{y}_{18} = -29.83'$	$A_{18} = 757.22$	$\bar{x}_{18} = 61.17'$
$\bar{y}_{19} = -60.08'$	$A_{19} = 3153.28$	$\bar{x}_{19} = 79.5'$
$\bar{y}_{20} = -89.17'$	$A_{20} = 200$	$\bar{x}_{20} = 81.33'$
$\bar{y}_{21} = -92.5'$	$A_{21} = 606.67$	$\bar{y}_{21} = 59.5'$
$\bar{y}_{22} = -110'$	$A_{22} = 225$	$\bar{x}_{22} = 63.83'$
$\bar{y}_{23} = -100'$	$A_{23} = 1038.33$	$\bar{x}_{23} = 29.5'$
	20 597.936	

$$\bar{y} = \frac{146\,892.9641}{20\,547.936} = 7.15'$$

$$\bar{x} = \frac{707090.7863}{20547.936} = 34.41'$$


120'
30'
158'
braced FRAMES
CM
98'-4"
19'-8"
22x166
120'
66'

LEVEL 345 TECH III 11-14-09

CENTER OF MASS CALCULATION (Subtracting voids from Areas)

$\bar{y}_1 = [22080(92') - 71.5(396) - 71.5(220) - 71.5(312) - 36.5(52) - 36(36)]/A_1 = 93.136'$

$A_1 = (158+26)120 - \text{voids} = 22080 - 1016 = 21064$

$\bar{x}_1 = [22080(60') - 75(52) - 112.5(36) - 14.5(396) - 35(220) - 66(312)]/A_1 = 60.90'$

$\bar{y}_2 = 214'$ $A_2 = 1560$ $\bar{x}_2 = 83.67'$

$\bar{y}_3 = 214'$ $A_3 = 288$ $\bar{x}_3 = 136.67'$

$\bar{y}_4 = 183'(4500) - 162(144)/A_4$, $800172/4356 = 183.7'$

$A_4 = 50(90) - \text{voids} = 4500 - 144 = 4356$

$\bar{x}_4 = 165(4500) - 128(144)/A_4 = 170.45'$

$\bar{y}_5 = (158/2)(30 \times 158) - 45'(32')/A_5$, $79.23'$

$A_5 = 30(158) - 2(16) = 4708$

$\bar{x}_5 = (135'(4740) - 122(16) - 148(16))/4708 = 135'$

$\bar{y}_6 = [105(\tfrac{1}{2} \times 60 \times 158) - 151(52) - 151(52) - 151(117) - 130(40) - 96(52) - 51.5(15)]/A_6 = 96.05'$

$A_6 = 4740$

$\bar{x}_6 = (170(4740) - 152(52) - 178(52) - 195(117) - 155.5(40) - 156(52) - 161.6(15))/A_6 = 158.06'$

$\bar{y}_7 = 45 + 14/2 = 52'$ $A_7 = 359.324$ $\bar{x}_7 = -12.833'$

$\bar{y}_8 = 45 + 14 + 11 = 70'$ $A_8 = 498.66$ $\bar{x}_8 = -11.333'$

$\bar{y}_9 = 133.25(3349) - 136.25(170.5)/A_9 = 133.09'$

$A_9 = 3349 - 170.5 = 3178.5$

$\bar{x}_9 = -17(3349) - (-22.5)(170.5)/A_9 = -16.71'$

$\bar{y} = \sum \bar{y}_i A_i / \sum A_i$

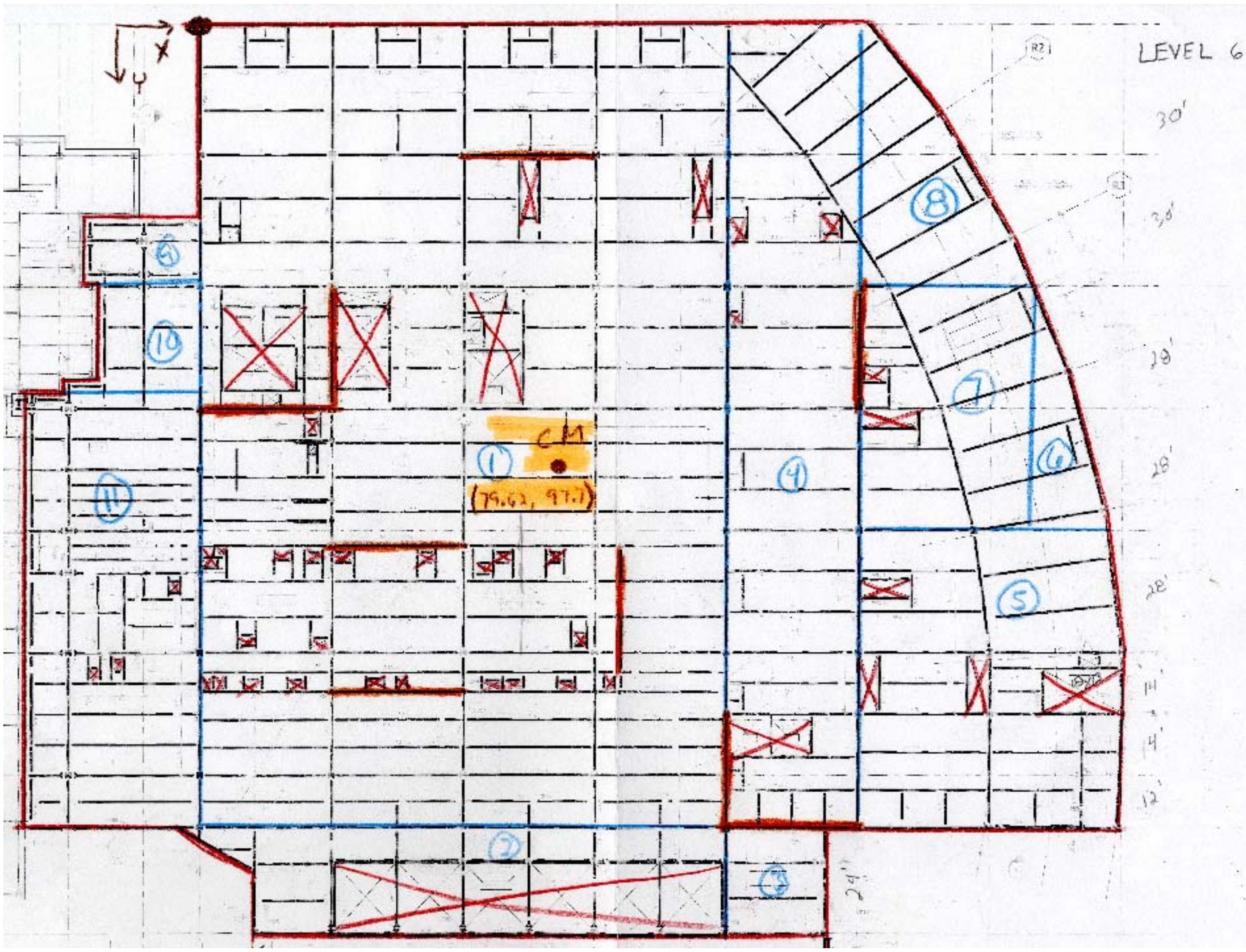
$= \dfrac{4462395.357}{40752.484}$

$= 109.5'$

$\bar{x} = \sum \bar{x}_i A_i / \sum A_i$

$= \dfrac{3516573.106}{40752.484}$

$= 86.29'$


LEVEL 6
CM
(79.62, 97.1)
30'
28'
28'
28'
14'
14'
12'
24'

LEVEL 6 TECH III 11-14-09

CENTER OF MASS CALCULATION (by Subtracting Voids from Areas) similar to level 4

$\bar{y}_1 = (22080(92') - 71.5(396) - 71.5(220) - 71.5(312) - 36.5(52) - 36(36) - 10(120)(6) - 3(142)(6) - 10(149)(6))/A_1$
$= 93.44'$

$A_1 = 22080 - \text{Voids}$
$22080 - 396 - 220 - 312 - 52 - 36 - 23(2\times3) = 20926$

$\bar{x}_1 = [60(22080) - 75(52) - 112.5(36) - 14.5(396) - 35(220) - 66(312) - 2(2')6 - 2(8)6 - 2(21)6 - 2(27)(6) - 32(6) - 37(6) - 42(6) - 52(6) - 2(65)6 - 2(68)6 - 3(83)6 - 94(6)]/A_1$
$= 61.05'$

$\bar{y}_2 = 170(2568) - 174(16\times88)/A_2$
$= 168'$

$A_2 = 1184$

$\bar{x}_2 = 75'(2568) - 75(16\times88)/A_2$
$= 73.5'$

$\bar{y}_3 = 170'$ $A_3 = 540$ $\bar{x}_3 = 131.25'$

$\bar{y}_4 = 79.23'$ $A_4 = 4708$ $\bar{x}_4 = 135'$

$\bar{y}_5 = (128(3480) - 151(52) - 151(52) - 130(40) - 151(234) - 90(52) - 81.5(15))/A_5 = 112.35'$

$A_5 = 3480 - 52 - 52 - 40 - 234$
$= 3102$

$\bar{x}_5 = (180(3480) - 152(52) - 178(52) - 210.5(234) - 155.5(40))/A_5$
$= 178.37'$

$\bar{y}_6 = 106.67'$ $A_6 = 560$ $\bar{x}_6 = 200'$

$\bar{y}_7 = (88(2240) - 90(52) - 77(15))/A_7$
$= 88.03'$

$A_7 = 2240 - 52 - 15$
$= 2173$

$\bar{x}_7 = (170'(2240) - 155.5(40) - 157.5(15))/A_7$
$= 171.33'$

$\bar{y}_8 = 50'$ $A_8 = 1200$ $\bar{x}_8 = 26.67'$

$\bar{y}_9 = 52'$ $A_9 = 359.324$ $\bar{x}_9 = -12.833$

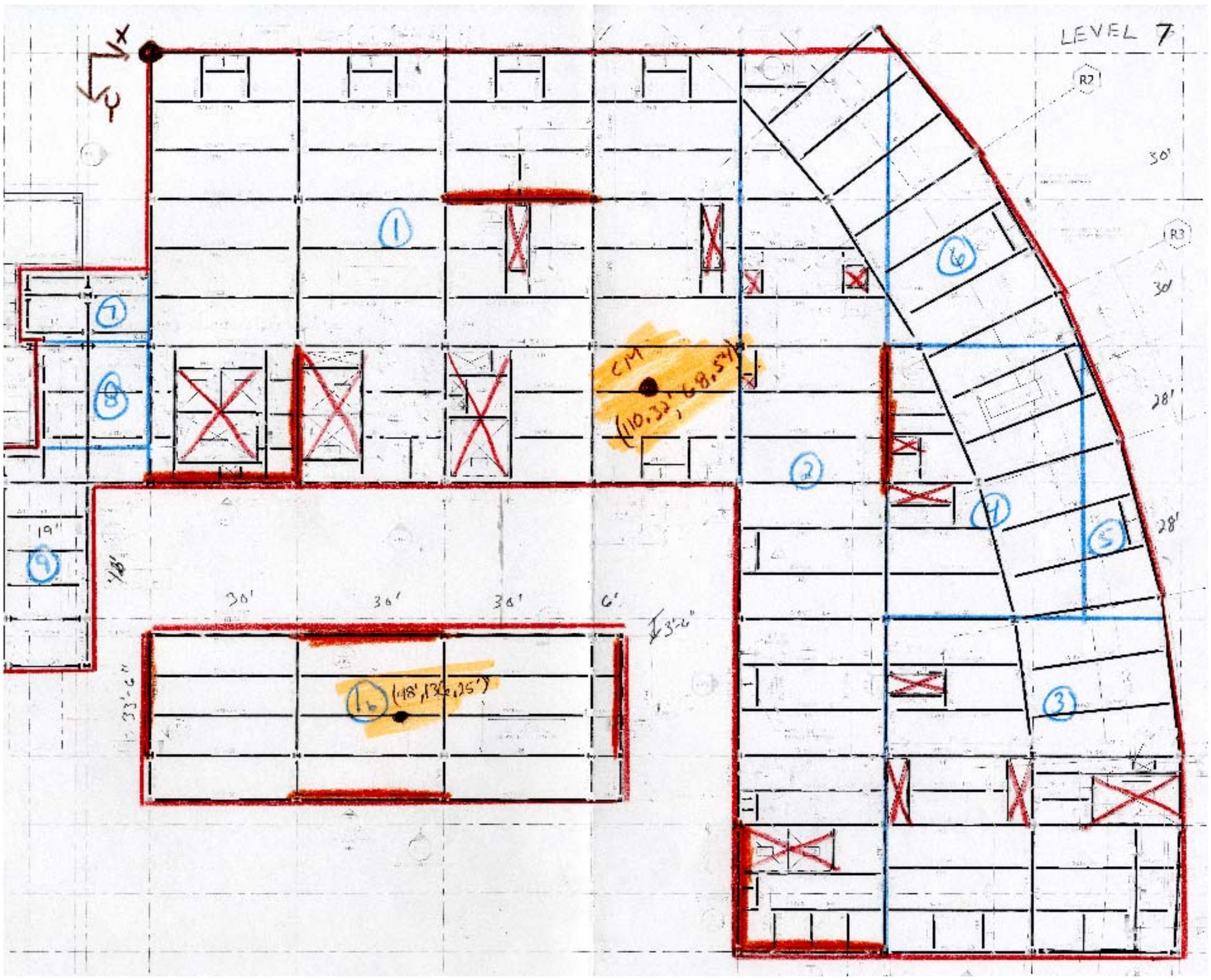
$\bar{y}_{10} = 70'$ $A_{10} = 498.66$ $\bar{x}_{10} = -11.333$

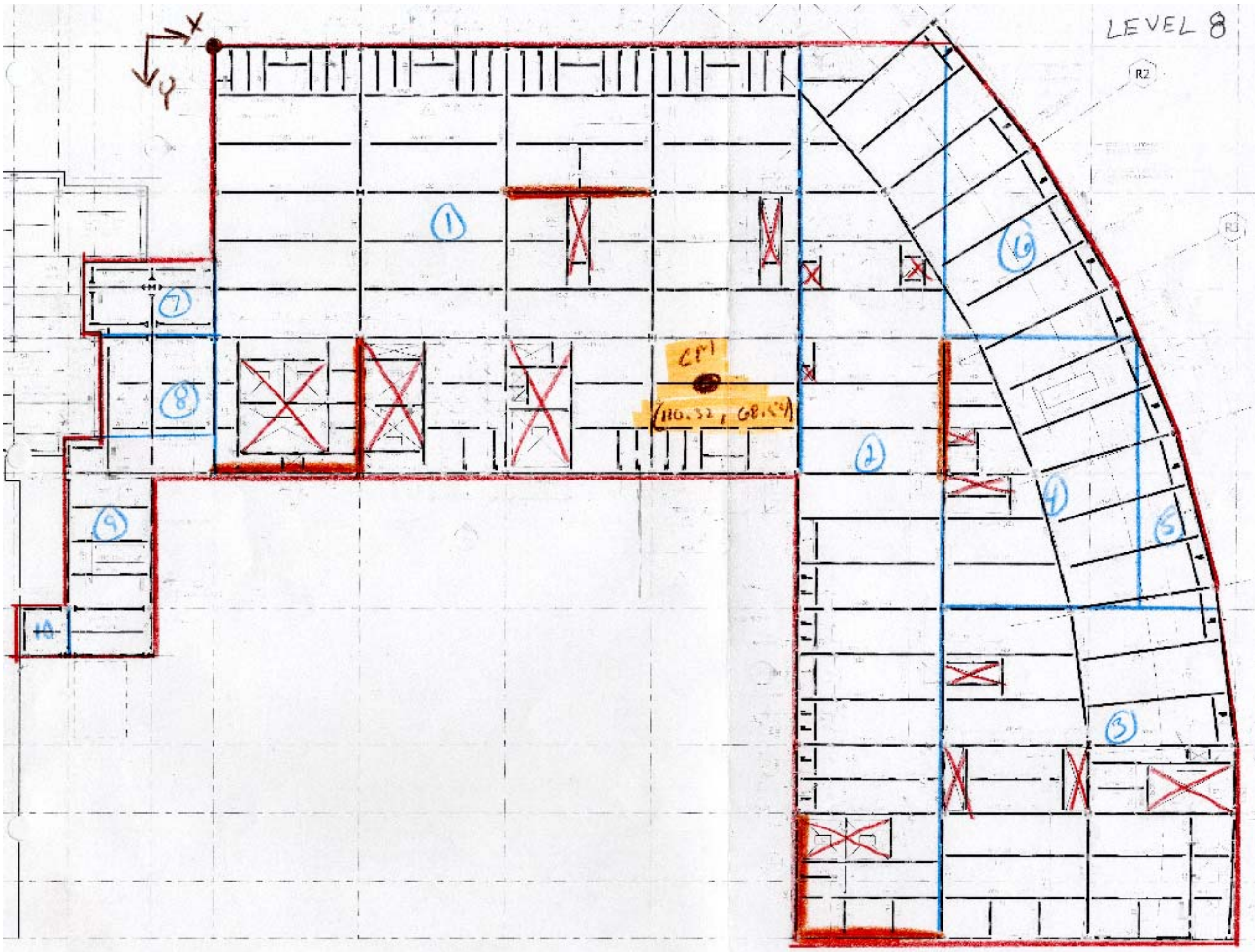
$\bar{y}_{11} = 133.25'$ $A_{11} = 3349$ $\bar{x}_{11} = -17$

$\overline{38599.984}$

$$\bar{y} = \frac{3771090.868}{38599.984} = 97.7'$$

$$\bar{x} = \frac{3073423.611}{38599.984} = 79.62'$$


LEVEL 7
CM
(110.32', 68.54')
(148', 136', 25')
30'
36'
36'
6'
19"
33'-6"
30'
30'
28'
28'


LEVEL 8
X
Y
CM
(110.32, 68.59)
1
2
3
4
5
6
7
8
9
10

LEVEL 7 ~~9~~ 8   TECH III   11-14-09

CENTER OF MASS CALCULATION

$\bar{y}_1 = (44'(10560) - 71.5(396) - 71.5(220) - 71.5(312) - 36.5(52) - 36(36))/A_1 = 41.397'$

$A_1 = 10560 - 396 - 220 - 312 - 52 - 36 = 9544$

$\bar{x}_1 = (60(10560) - 76(52) - 112.5(36) - 14.5(396) - 35(220) - 66(312))/A = 61.99'$

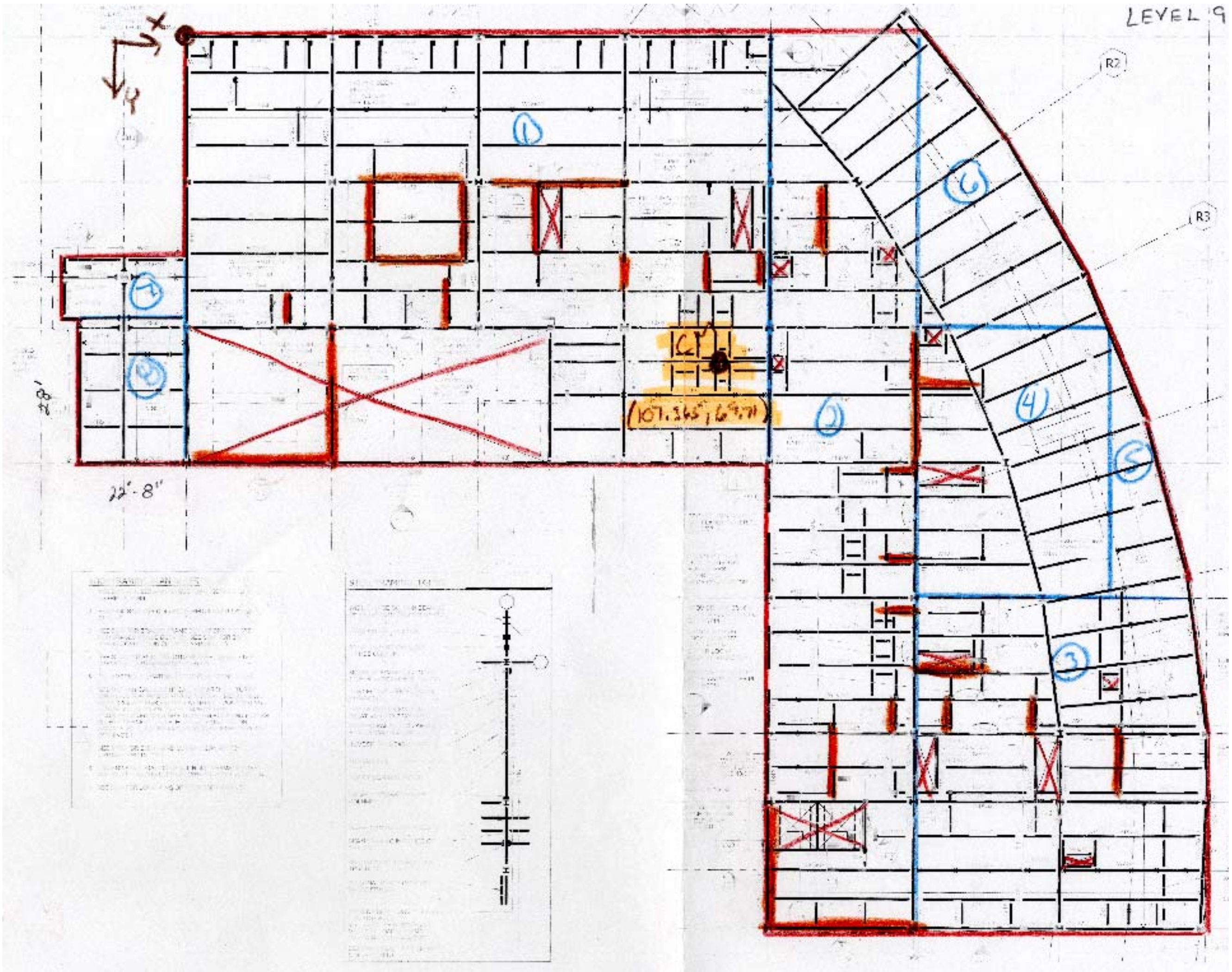
$\bar{y}_2 = 79.23'$	$A_2 = 4708$	$\bar{x}_2 = 135'$
$\bar{y}_3 = 112.36'$	$A_3 = 3102$	$\bar{x}_3 = 178.37'$
$\bar{y}_4 = 88.03'$	$A_4 = 2173$	$\bar{x}_4 = 171.33'$
$\bar{y}_5 = 106.67'$	$A_5 = 560$	$\bar{x}_5 = 200'$
$\bar{y}_6 = 50'$	$A_6 = 1200$	$\bar{x}_6 = 26.67'$
$\bar{y}_7 = 52'$	$A_7 = 359.324$	$\bar{x}_7 = -12.833'$
$\bar{y}_8 = 70'$	$A_8 = 498.66$	$\bar{x}_8 = -11.333'$
$\bar{y}_9 = 164'$	$A_9 = 874$	$\bar{x}_9 = 21'$
$\bar{y}_{10} = 122'$	$A_{10} = 164$	$\bar{x}_{10} = -34.33'$
	23122.984	

$\bar{y} = \dfrac{1584816.946}{23122.984} = 68.54'$

$\bar{x} = \dfrac{2550953.551}{23122.984} = 110.32'$

$\bar{y}_b = 136.25$

$\bar{x}_b = 48'$


LEVEL 9
X
Y
1
2
3
4
5
6
7
8
CT
(107.365', 69.71)
28'
22'-8"
R2
R3

LEVEL 9 TECH III 11-14-09

CENTER of MASS Calculations

$\bar{y}_1 = (44'(10560) - 36.5(52) - 36(36) - 54(2100))/A_1 = 41.573'$ $A_1 = 10560 - 52 - 36 - 2100 = 8372$ $\bar{x}_1 = (60(10560) - 75(52) - 112.5(36) - 37.5(2100))/A_1 = 65.32'$

$\bar{y}_2 = 79.23'$ $A_2 = 4708$ $\bar{x}_2 = 135'$

$\bar{y}_3 = (118(3480) - 151(52) - 151(52) - 90(52) - 136(6) - 145(8))/A_3 = 117.31'$ $A_3 = 3480 - 52 - 52 - 52 - 6 - 8 = 3310$ $\bar{x}_3 = (180(3480) - 130(52) - 151(52) - 151(52) - 136(6) - 168(8))/A_3 = 181.8$

$\bar{y}_4 = 88.03'$ $A_4 = 2173$ $\bar{x}_4 = 171.33'$

$\bar{y}_5 = 106.67'$ $A_5 = 560$ $\bar{x}_5 = 200'$

$\bar{y}_6 = 50'$ $A_6 = 1200$ $\bar{x}_6 = 26.67'$

$\bar{y}_7 = 52'$ $A_7 = 359.324$ $\bar{x}_7 = -12.833'$

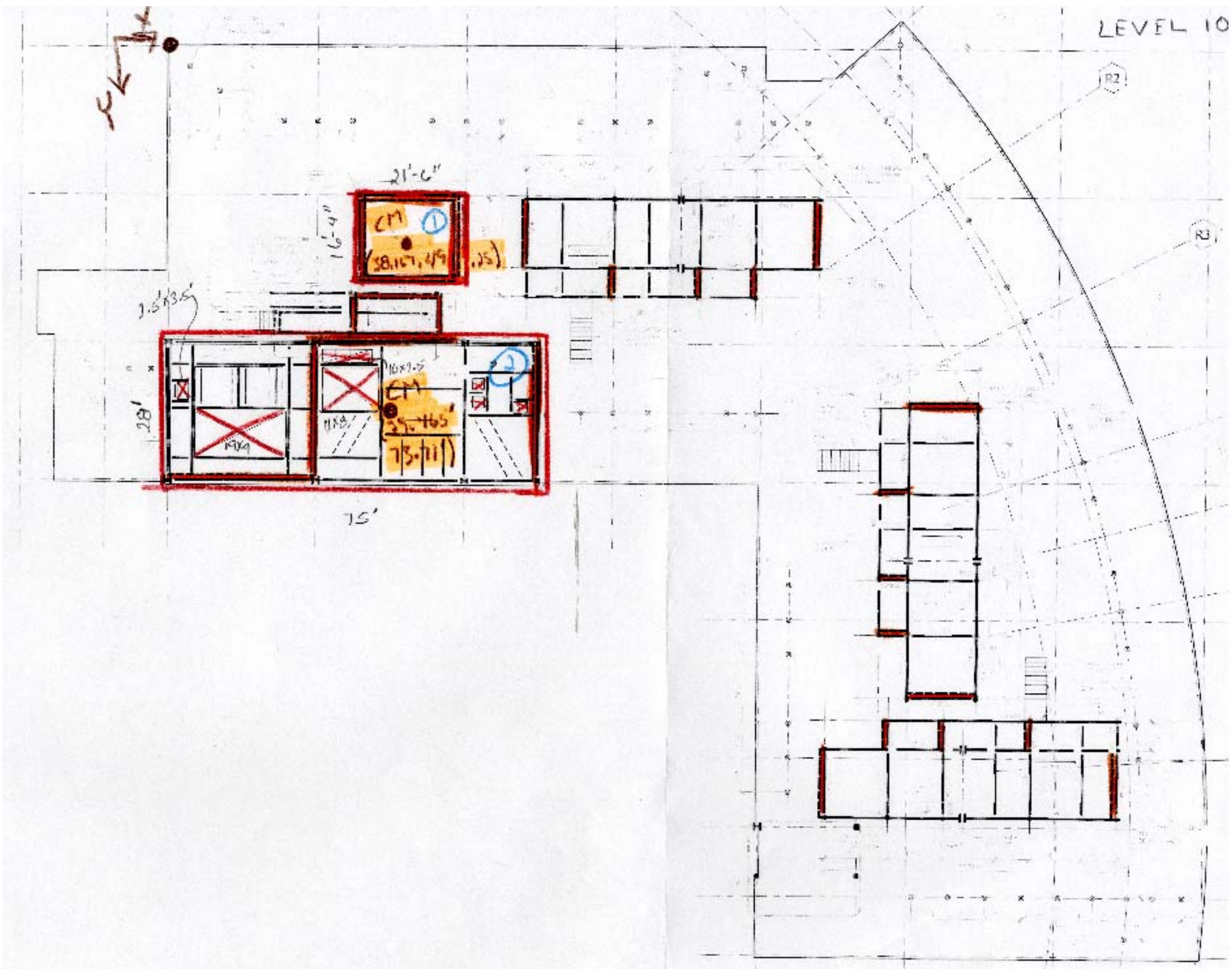
$\bar{y}_8 = 74'$ $A_8 = 634.66$ $\bar{x}_8 = -11.333'$

$\Sigma A = 21316.984$

$\bar{y} = \dfrac{1486034.174}{21316.984} = 69.71'$ $\bar{x} = \dfrac{2288697.323}{21316.984} = 107.365'$

LEVEL 10 $\bar{y}_1 = 38.167'$ $\bar{x} = 49.25$

$\bar{y}_2 = (74(2100) - 72(8.75) - 70(88) - 63(25) - 2(72)(6) - 70(6) - 78.5(171))/A_2 = 73.71'$ $A_2 = 2100 - 8.75 - 88 - 25 - 6 - 6 - 171 = 1795.25$ $\bar{x}_2 = (37.5(2100) - 4(8.75) - 15(171) - 36(25) - 36.5(88) - 63(6)(2) - 72(6))/A_2 = 39.465'$


LEVEL 10
21'-6"
16'-4"
CM
28'
75'
CM

AT LEVEL 3 TECH III 11-19-09

CENTER OF RIGIDITY

$K = AE/L$ assuming only braces are participating

$k_1 \Rightarrow$ HSS 8x8x3/8 $\Rightarrow A = 10.4 in^2$ $L = 251.75''$

$= 10.4(29000)/251.75$

$= 1198.01$ k/in

$k_2 = k_1$

$k_3 \Rightarrow$ HSS 10x10x5/8 $\Rightarrow A = 21.0 in^2$ $L = 251.75''$

$= 21(29000)/251.75$

$= 2419.066$ k/in

$k_4 \Rightarrow$ HSS 10x10x3/8 $A = 13.2 in^2$ $L = 243.31''$

$= 13.2(29000)/243.31$

$= 1573.3$ k/in

$k_5 \Rightarrow$ HSS 10x10x1/2 $A = 17.2 in^2$ $L = 243.31''$

$= 17.2(29000)/243.31$

$= 2050.06$ k/in

$k_6 \Rightarrow$ HSS 10x10x1/2 $A = 17.2 in^2$ $L = 235.184''$

$= 17.2(29000)/235.184$

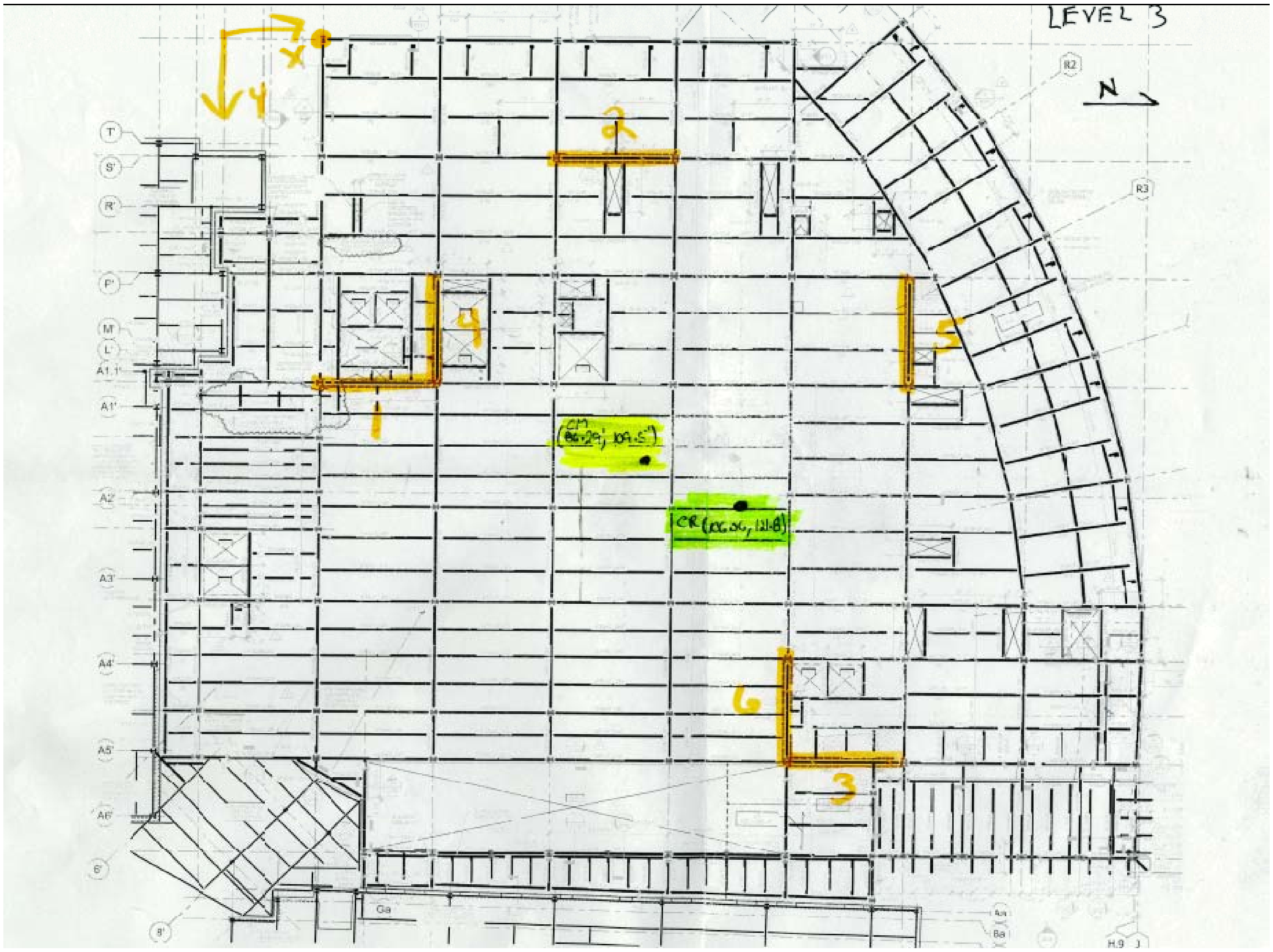
$= 2120.89$ k/in

$$\bar{X} = \frac{\sum k_{iy} X_i}{\sum k_{iy}} \qquad \bar{Y} = \frac{\sum k_{ix} Y_i}{\sum k_{ix}}$$

$$\bar{X} = \frac{k_4(30') + k_5(150') + k_6(120')}{k_4 + k_5 + k_6} \qquad \bar{Y} = \frac{k_1(88') + k_1(30') + k_3(184')}{k_1 + k_2 + k_3}$$

$$= \frac{1573.3(30') + 2050.06(150') + 2120.89(120')}{1573.3 + 2050.06 + 2120.89} \qquad = \frac{1198.01(88) + 1198.01(30) + 2419.066(184)}{1198(2) + 2419.066}$$

$$= 106.06 \qquad = 121.8$$


LEVEL 3
N
x
y
1
2
3
4
5
6
CM (86.29, 109.5')
CR (106.06, 121.8)

AT LEVEL 5 TECH III 11-15-09

CENTER OF RIGIDITY

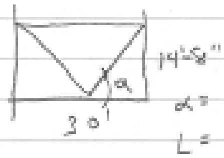
Assuming loading perpendicular to frame direction (strong axis)

$K = AE/L \qquad \delta = PL/AE$

$k_1 \Rightarrow$ HSS 8x8x3/8 $\Rightarrow A = 10.4 \quad L = 251.75''$

$K_1 = 10.4(29000)/251.75$

$= 1198.01 \text{ k/in}$



$\alpha = 44.36°$

$L = 20.98' = 251.75''$

$K_2 = K_1$

$k_3 \Rightarrow$ HSS 10x10x3/8 $\Rightarrow A = 13.2 \text{ in}^2 \quad L = 251.75''$

$= 13.2(29000)/251.75$

$= 1520.556 \text{ k/in}$

$k_4 \Rightarrow$ HSS 10x10x3/8 $\Rightarrow A = 13.2 \text{ in}^2 \quad L = 243.31''$

$= 13.2(29000)/243.31$

$= 1573.3 \text{ k/in}$

$k_5 \Rightarrow$ HSS10x10x1/2 $\Rightarrow A = 17.2 \text{ in}^2 \quad L = 243.31''$

$= 17.2(29000)/243.31$

2050.06 k/in

$k_6 \Rightarrow$ HSS10x10x1/2 $\Rightarrow A = 17.2 \text{ in}^2 \quad L = 235.184''$

$17.2(29000)/235.184$

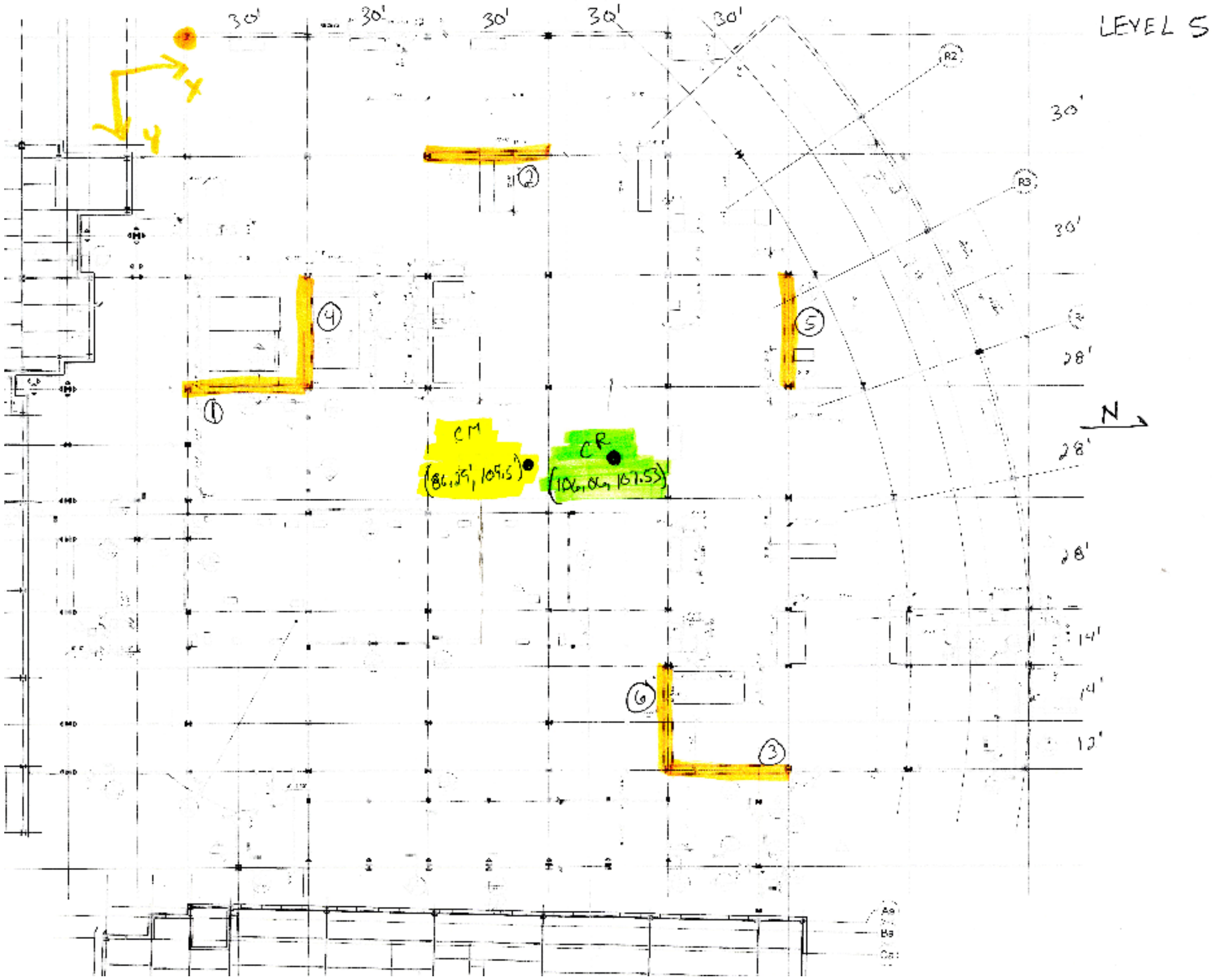
2120.89 k/in

$$\bar{x} = \frac{\sum_i k_{iy} x_i}{\sum_i k_{iy}} \qquad \bar{y} = \frac{\sum_i k_{ix} y_i}{\sum_i k_{ix}}$$

$$\bar{x} = \frac{K_4(30') + k_5(150') + k_6(120')}{k_4 + k_5 + k_6} \qquad \bar{y} = \frac{k_1(88') + k_2(30') + k_3(184')}{k_1 + k_2 + k_3}$$

$$= \frac{1573.3(30) + 2050.06(150) + 2120.89(120)}{1573.3 + 2050.06 + 2120.89} \qquad = \frac{1198.01(88) + 1198(30) + 1520.556(184)}{2(1198.01) + 1520.556}$$

$$= 106.86' \qquad = 107.53'$$


LEVEL 5
30'
30'
30'
30'
30'
30'
30'
28'
28'
28'
14'
14'
12'
N
x
y
CM
(86.59', 109.5')
CR
(106.06, 107.53)
1
2
3
4
5
6

AT LEVEL 6 TECH III 11-15-09

CENTER of RIGIDITY

$K_1 \Rightarrow$ HSS $8 \times 8 \times 5/16$ $A = 8.76\ in^2$ $L = 251.75''$

$= 8.76(29000)/(251.75'')$

$= 1009.10\ k/in$

$K_2 \Rightarrow$ HSS $8 \times 8 \times 3/8$ $A = 10.4\ in^2$ $L = 251.75''$

$= 10.4(29000)/251.75$

$= 1198.01\ k/in$

$K_3 = K_2 = 1198.01\ k/in$

$K_4 \Rightarrow$ HSS $8 \times 8 \times 3/8$ $A = 10.4\ in^2$ $L = 243.31''$

$= 10.4(29000)/243.31$

$= 1239.57\ k/in$

$K_5 \Rightarrow$ HSS $10 \times 10 \times 3/8$ $A = 13.2\ in^2$ $L = 243.31''$

$= 13.2(29000)/243.31$

$= 1573.3\ k/in$

$K_6 \Rightarrow$ HSS $10 \times 10 \times 3/8$ $A = 13.2\ in^2$ $L = 235.184''$

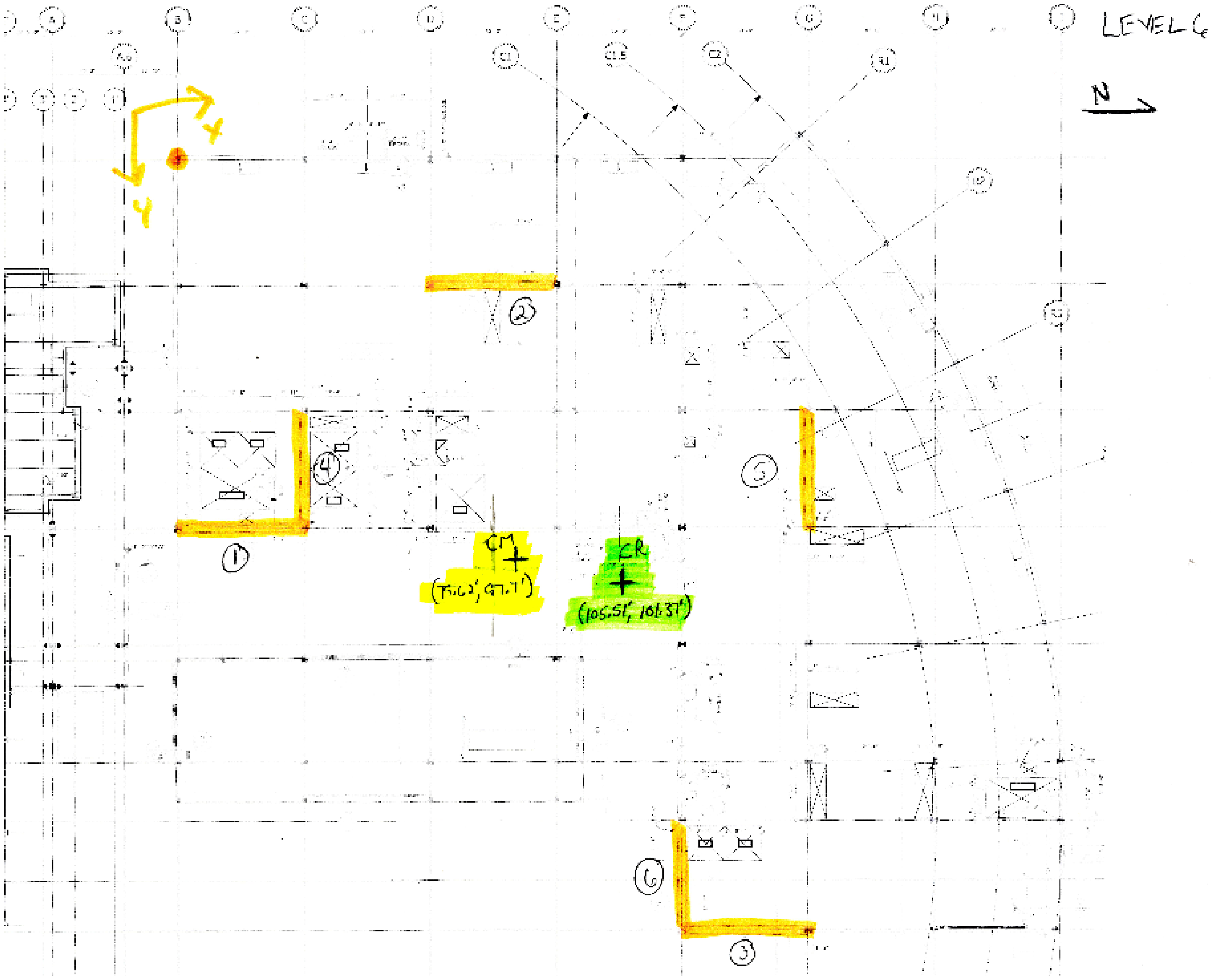
$= 13.2(29000)/235.184$

$= 1627.66\ k/in$

$$\bar{X} = \frac{k_4(30') + k_5(150') + k_6(120')}{k_4 + k_5 + k_6} \qquad \bar{Y} = \frac{k_1(88') + k_2(30') + k_3(184')}{k_1 + k_2\ k_3}$$

$$= \frac{1239.57(30) + 1573.3(150) + 1627.66(120)}{1239.57 + 1573.3 + 1627.66} \qquad = \frac{1009.1(88) + 1198.01(30) + 1198.01(184)}{1009.1 + 2(1198.01)}$$

$$= 105.51' \qquad = 101.37'$$


LEVEL 6
N
X
Y
CM
(75.62', 97.7')
CR
(105.51', 101.37')
1
2
3
4
5
6

AT LEVEL 7 + 8 TECH III 11-15-09

CENTER of RIGIDITY

$K_1 \Rightarrow$ HSS 8×8×5/16 $A = 8.76\ in^2$ $L = 251.75''$

$= 8.76(29000)/251.75$

$= 1009.1\ k/in$

$K_2 = k_1 = k_3$

$K_4 \Rightarrow$ HSS 8×8×5/16 $A = 8.76\ in^2$ $L = 243.31''$

$= 8.76(29000)/243.31$

$= 1044.1\ k/in$

$K_5 \Rightarrow$ HSS 8×8×3/8 $A = 10.4\ in^2$ $L = 243.31''$

$= 10.4(29000)/243.31$

$= 1239.571\ k/in$

$K_6 \Rightarrow$ HSS 8×8×5/16 $A = 8.76\ in^2$ $L = 235.184''$

$= 8.76(29000)/235.184$

$= 1080.176\ k/in$

$$\bar{X} = \frac{k_4(30') + k_5(150') + k_6(120')}{k_4 + k_5 + k_6}$$

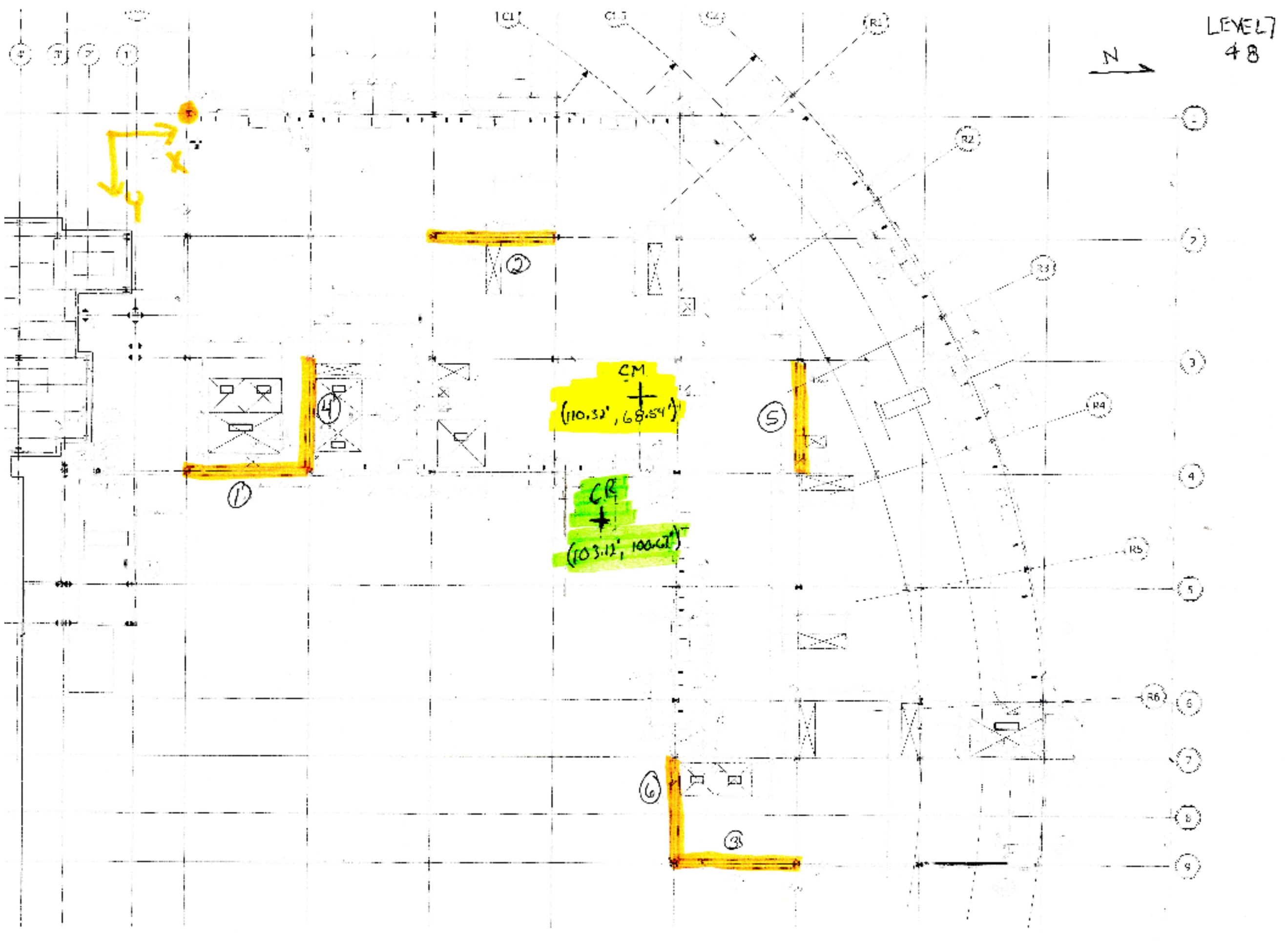
$$= \frac{1044.1(30) + 1239.571(150) + 1080.176(120)}{1044.1 + 1239.571 + 1080.176}$$

$$= 103.12'$$

$$\bar{Y} = \frac{K_1(88') + k_2(30') + k_3(184')}{K_1 + k_2 + k_3}$$

$$= \frac{1009.1(88) + 1009.1(30) + 1009.1(184)}{3(1009.1)}$$

$$= 100.67'$$


LEVEL 7
48
N
X
Y
CM
(110.32', 68.54')
CR
(103.12', 100.62')
1
2
3
4
5
6

DIRECT FORCE INTO EACH FRAME ON LEVEL 3

TECH III

11-A-09

$P_y = 98.45\,k$ Table 3.5

$\Sigma k_y = k_1 + k_2 + k_3$

$= 1198.01 + 1198.01 + 2419.066$

$= 4815.086$

$P_x = 81.26k$ Table 3.3

$\Sigma k_x = k_4 + k_5 + k_6$

$= 1573.3 + 2050.06 + 2120.89$

$= 5744.25$

$$F_1 = \frac{k_1}{\Sigma k_y} P_y = \frac{1198.01(98.45)}{4815.086} = 24.495\,k$$

$$F_2 = F_1 = 24.495\,k$$

$$F_3 = \frac{k_3}{\Sigma k_y} P_y = \frac{2419.066(98.45)}{4815.086} = 49.461\,k$$

$$F_4 = \frac{k_4}{\Sigma k_x} P_x = \frac{1573.3(81.26)}{5744.25} = 22.256\,k$$

$$F_5 = \frac{k_5}{\Sigma k_x} P_x = \frac{2050.06(81.26)}{5744.25} = 29.0\,k$$

$$F_6 = \frac{k_6}{\Sigma k_x} P_x = \frac{2120.89(81.26)}{5744.25} = 30.0\,k$$

DIRECT FORCE INTO EACH FRAME ON EACH LEVEL

TECH III

11-15-09

LEVEL 5 $P_5 = 100.27\ k$ $\sum k_y = k_1 + k_2 + k_3$

$= 1198.1(2) + 1520.556 = 3916.756$

$$F_1 = \frac{k_1}{\sum k_y}(P_5) = \frac{1198.01(100.27)}{3916.756}$$

$= 30.669\ k$ from Table 3.5

$$F_2 = \frac{k_2}{\sum k_y}(P_5) = \frac{1198.01(100.27)}{3916.756} = 30.669\ k$$

$$F_3 = \frac{k_3}{\sum F_y}(P_5) = \frac{1520.556(100.27)}{3916.756} = 38.927\ k$$

$P_x = 84.91\ k$ from Table 3.3 $\sum k_x = k_4 + k_5 + k_6$

$= 5744.25$

$$F_4 = \frac{k_4}{\sum k_x}(P_x) = \frac{1573.3(84.91)}{5744.25}$$

$= 23.256\ k$

$$F_5 = \frac{k_5}{\sum k_x} P_x = \frac{2050.06(84.91)}{5744.25}$$

$= 30.30\ k$

$$F_6 = \frac{k_6}{\sum k_x} P_x = \frac{2120.89(84.91)}{5744.25}$$

$= 31.35\ k$

DIRECT FORCE INTO EACH FRAME ON LEVEL 6 — TECH III — 11-15-09

$P_x = 84.64\,k$ Table 3.3

$\Sigma k_y = k_1 + k_2 + k_3$
$= 2(1198.01) + 1009.1$
$= 3405.12$

$P_y = 93.73$ Table 3.5

$\Sigma k_x = k_4 + k_5 + k_6$
$= 1239.57 + 1573.3 + 1627.66$
$= 4440.53$

$$F_1 = \frac{k_1}{\Sigma k_y} P_y = \frac{1198.01(93.73)}{3405.12} = 32.98\,k$$

$$F_2 = \frac{k_2}{\Sigma k_y} P_y = \frac{1198.01(93.73)}{3405.12} = 32.98\,k$$

$$F_3 = \frac{k_3}{\Sigma k_y} P_y = \frac{1009.1(93.73)}{3405.12} = 27.78$$

$$F_4 = \frac{k_4}{\Sigma k_x} P_x = \frac{1239.57(84.64)}{4440.53} = 23.627\,k$$

$$F_5 = \frac{k_5}{\Sigma k_x} P_x = \frac{1573.3(84.64)}{4440.53} = 29.99\,k$$

$$F_6 = \frac{k_6}{\Sigma k_x} P_x = \frac{1627.66(84.64)}{4440.53} = 31.02\,k$$

DIRECT FORCE INTO EACH FRAME ON LEVEL 7 TECH III 11-15-09

$P_y = 86.37$ Table 3.5

$\Sigma k_y = k_1 + k_2 + k_3$
$= 1009.1(3)$
$= 3027.3$

$P_x = 83.51$ Table 3.3

$\Sigma k_x = k_4 + k_5 + k_6$
$= 1044.1 + 1239.571 + 1080.176$
$= 3363.847$

$$F_1 = \frac{k_1}{\Sigma k_y} P_y = \frac{1009.1(86.37)}{3027.3} = 28.79\,k$$

$$F_2 = \frac{k_2}{\Sigma k_y} P_y = F_1 = 28.79\,k$$

$$F_3 = \frac{k_3}{\Sigma k_y} P_y = F_1 = 28.79\,k$$

$$F_4 = \frac{k_4}{\Sigma k_x} P_x = \frac{1044.1(83.51)}{3363.847} = 25.92\,k$$

$$F_5 = \frac{k_5}{\Sigma k_x} P_x = \frac{1239.571(83.51)}{3363.847} = 30.77\,k$$

$$F_6 = \frac{k_6}{\Sigma k_x} P_x = \frac{1080.176(83.51)}{3363.847} = 26.82\,k$$

DIRECT FORCE INTO EACH FRAME ON LEVEL 8   TECH III   11-15-09

$P_y = 62.53k$ Table 3.5

$\Sigma k_y = k_1 + k_2 + k_3$

$= 1009.1(3)$

$= 3027.3$

$P_x = 49.5k$ Table 3.3

$\Sigma k_x = k_4 + k_5 + k_6$

$= 1044.1 + 1239.571 + 1080.176$

$= 3363.847$

$$F_1 = \frac{k_1}{\Sigma k_y} P_y = \frac{1009.3(62.53)}{3027.3}$$

$$= 20.84\ k$$

$$F_2 = F_1 = F_3$$

$$F_4 = \frac{k_4}{\Sigma k_x} P_x = \frac{1044.1(49.5)}{3363.847}$$

$$= 15.36\ k$$

$$F_5 = \frac{k_5}{\Sigma k_x} P_x = \frac{1239.571(49.5)}{3363.847}$$

$$= 18.24\ k$$

$$F_6 = \frac{k_6}{\Sigma k_x} P_x = \frac{1080.176(49.5)}{3363.847}$$

$$= 15.90\ k$$

FORCE IN EACH FRAME AT EACH LEVEL DUE TO ECCENTRICITY

TECH III

11-19-09

LEVEL 3

$e_x = 106.06' - 86.29 = 19.77'$ $P_y = 98.45\ k$ Table 3.5

$e_y = 121.8' - 109.5' = 12.3'$ $P_x = 81.26\ k$ Table 3.3

d_i = distance from CR to frame i

5% minimum of direct force

$$F_i = \frac{k_{iy}\, d_i\, P_y\, e_x}{\sum k_{jy}\, d_j^2}$$

Y: $F_1 = \dfrac{1198.01(18.06)(98.45)\,19.77}{1198.01(18.06)^2 + 1198.01(76.06)^2 + 2419.066(77.94)^2} = \dfrac{42111487.18}{22016346.97}$

$= \boxed{1.91k} > 5\%$

$F_2 = \dfrac{1198.01(76.06)(98.45)(19.77)}{22016346.97} = \boxed{8.056k}$

$F_3 = \dfrac{2419.066(77.94)(98.45)(19.77)}{22016346.97} = \boxed{16.668k}$

X: $F_4 = \dfrac{1573.3(91.8)(81.26)(12.3)}{1573.3(91.8^2) + 2050.06(28.2^2) + 2120.89(8.2^2)} = \dfrac{144356436.7}{15031475.05}$

$= \boxed{9.604\ k}$

$F_5 = \dfrac{2050.06(28.2)(81.26)(12.3)}{15031475.05} = \boxed{3.844\ k}$

$F_6 = \dfrac{2120.89(8.2)(81.26)(12.3)}{15031475.05} = 1.156$

5% minimum $= \boxed{1.5k}$

FORCE IN EACH FRAME AT EACH LEVEL DUE TO ECCENTRICITY — TECH III — 11-15-09

LEVEL 5

$ex = 106.06 - 86.29 = 19.77'$ $P_y = 100.27\,k$ Table 3.5

$ey = 109.5 - 107.53 = 1.97'$ $P_x = 84.91\,k$ Table 3.3

d_i = distance from CR to frame i

5% minimum of direct force

$$F_i = \frac{k_{iy}\, d_i\, P_y\, ex}{\sum k_{jy}\, d_j^2}$$

y: $F_1 = \dfrac{1198.1(19.53)(100.27)(19.77)}{1198.1(19.53^2) + 1198.1(77.53)^2 + 1520.56(76.47^2)} = \dfrac{46\,384\,512.41}{16\,550\,360.01}$

$= \boxed{2.80}\,k$ 5% minimum = 1.53 k

$F_2 = \dfrac{1198.1(77.53)(100.27)19.77}{16\,550\,360.01} = \boxed{11.13\,k}$

$F_3 = \dfrac{1520.56(76.47)(100.27)(19.77)}{16\,550\,360.01} = \boxed{13.93\,k}$

x: $F_4 = \dfrac{1573.3(76.06)(84.91)(1.97)}{1573.3(76.06^2) + 2050.06(43.94^2) + 2120.89(13.94)^2} = \dfrac{20\,016\,720.77}{13\,471\,973.16}$

$= \boxed{1.49\,k}$ 5% minimum = 1.16k

$F_5 = \dfrac{2050.06(43.94)(84.91)(1.97)}{13\,471\,973.16} = 1.12\,k$ 5% minimum = $\boxed{1.52\,k}$

$F_6 = \dfrac{2120.89(13.94)(84.91)(1.97)}{13\,471\,977.16} = .637\,k$ 5% minimum = $\boxed{1.57\,k}$

FORCE IN EACH FRAME DUE TO ECCENTRICITY — TECH III LEVEL 6 — 11-15-09

$e_x = 105.51' - 79.62' = 25.89'$ — $P_y = 93.73\ k$ Table 3.5

$e_y = 101.37' - 97.7' = 3.67'$ — $P_x = 84.64\ k$ Table 3.3

d_i = distance from CR to frame i

5% minimum of direct force

$$F_i = \frac{k_{iy}\, d_i\, P_y\, e_x}{\sum k_{iy}\, d_j^2}$$

Y:

$$F_1 = \frac{1009.1(13.37)(93.73)(25.89)}{1009.1(13.37^2) + 1198.1(71.37^2) + 1198.1(82.63^2)} = \frac{32739819.51}{14463405.5} = \boxed{2.26\ k}$$

5% minimum = 1.65 k

$$F_2 = \frac{1198.1(71.37)(93.73)(25.89)}{14463405.5} = \boxed{14.35\ k}$$

$$F_3 = \frac{1198.1(82.63)(93.73)(25.89)}{14463405.5} = \boxed{16.61\ k}$$

X:

$$F_4 = \frac{1239.57(75.51)(84.64)(3.67)}{1239.57(75.51^2) + 1573.3(44.49^2) + 1627.66(14.49^2)} = \frac{29074834.15}{10523601.67} = \boxed{2.76\ k}$$

$$F_5 = \frac{1573.3(44.49)(84.64)(3.67)}{10523601.67} = \boxed{2.07\ k}$$

$$F_6 = \frac{1627.66(14.49)(84.64)(3.67)}{10523601.67} = 0.696\ k$$

5% minimum = $\boxed{1.55\ k}$

FORCE IN EACH FRAME AT EACH LEVEL DUE TO ECCENTRICITY

TECH III

11-15-09

LEVEL 7

$e_x = 110.32' - 103.12' = 7.2'$ $\quad P_y = 86.37\ k$ Table 3.5

$e_y = 100.67' - 68.54' = 32.13'$ $\quad P_x = 83.51\ k$ Table 3.3

d_i = distance from CR to frame i

5% minimum of direct force

$$F_i = \frac{k_{iy}\, d_i\, P_y\, e_x}{\sum k_{jy}\, d_j^2}$$

Y: $F_1 = \dfrac{1009.1(12.67)(86.37)(7.2')}{1009.1(12.67^2) + 1009.1(70.67^2) + 1009.1(83.33^2)} = \dfrac{7\,950\,715.934}{12\,208\,764.57}$

$= 0.65\ k$ $\quad$ 5% minimum $= \boxed{11.44\ k}$

$F_2 = \dfrac{1009.1(70.67)(86.37)(7.2)}{12\,208\,764.57} = \boxed{3.63\ k}$

$F_3 = \dfrac{1009.1(83.33)(86.37)(7.2)}{12\,208\,764.57} = \boxed{4.28\ k}$

X: $F_4 = \dfrac{1044.1(73.12)(83.51)(32.13)}{1044.1(73.12^2) + 1239.57(46.88^2) + 1080.18(16.88^2)} = \dfrac{204\,845\,999.9}{8\,614\,342.637}$

$= \boxed{23.78\ k}$

$F_5 = \dfrac{1239.57(46.88)(83.51)(32.13)}{8\,614\,342.637} = \boxed{18.1\ k}$

$F_6 = \dfrac{1080.18(16.88)(83.51)(32.13)}{8\,614\,342.637} = \boxed{5.68\ k}$

FORCE IN EACH FRAME AT EACH LEVEL DUE TO ECCENTRICITY

TECH III

LEVEL 8

11-15-09

$e_x = 7.2'$ $P_y = 62.53$ Table 3.5

$e_y = 32.13'$ $P_x = 49.5$ Table 3.3

d_i = distance from CR to frame i

5% minimum of direct force

y: $F_1 = \frac{1009.1(12.67)(62.53)(7.2)}{12\,208\,764.57} = 0.47\,k$ 5% minimum = $\boxed{1.04\,k}$

$F_2 = \frac{1009.1(70.67)(62.53)(7.2)}{12\,208\,764.57} = \boxed{2.63\,k}$

$F_3 = \frac{1009.1(83.33)(62.53)(7.2)}{12\,208\,764.57} = \boxed{3.1\,k}$

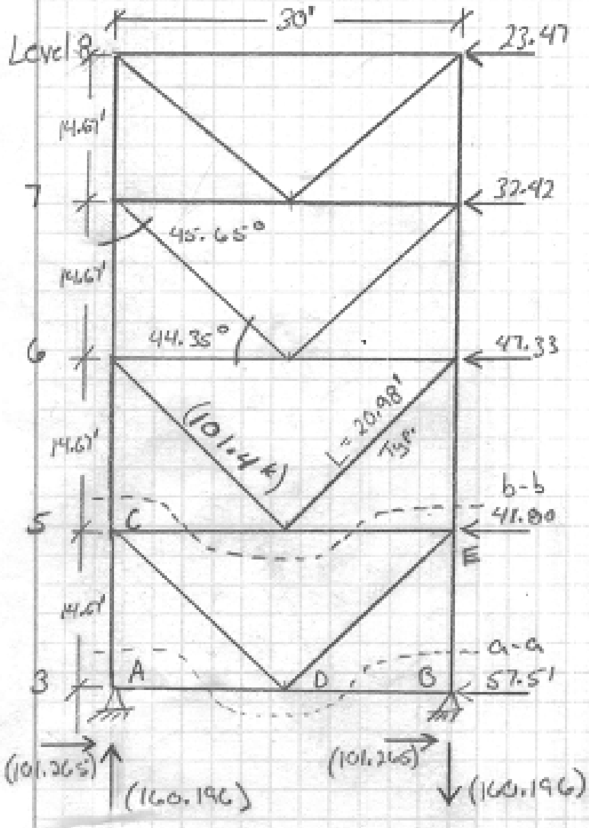
x: $F_4 = \frac{1044.1(73.12)(49.5)(32.13)}{8\,614\,342.637} = \boxed{14.10\,k}$

$F_5 = \frac{1239.57(46.88)(49.5)(32.13)}{8\,614\,342.637} = \boxed{10.73\,k}$

$F_6 = \frac{1080.18(16.88)(49.5)(32.13)}{8\,614\,342.637} = \boxed{3.37\,k}$

Appendix F: Member Spot Checks

FRAME #2 | GRIDLINE 2 D-E | FBD HAND CALCULATIONS



using a-a

$+\circlearrowright \sum M_A = 0$

$R_{By}(30') - 23.47(4)(14.67') - 32.42(3)(14.67') - 47.33(2)(14.67') - 41.80(14.67') = 0$

$R_{By} = 4805.892/30 = 160.196\text{ k}$

$\uparrow \sum F_y = 0$

$R_{Ay} = 160.196\text{ k}$

using b-b

$+\circlearrowright \sum M_C = 0$

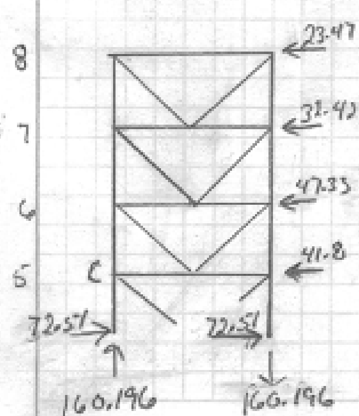
$R_{Ey}(30') = -23.47(3)(14.67') - 32.42(2)(14.67') - 47.33(1)(14.67') = 0$

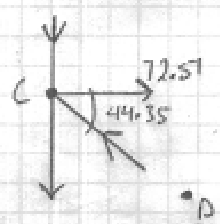
$R_{Ey} = 89.28\text{ k}$

$\rightarrow \sum F_x = 0$

$R_{Cx} = R_{Ex} = (23.47 + 32.42 + 47.33 + 41.8)/2$

$= 72.51$





$F_{CD} \Rightarrow \cos 44.35 = 72.51/x$

$F_{CD} = 72.51/\cos 44.35$

$\boxed{= 101.4}$

GRAVITY LOADS TO BEAMS TO BRACES | MEMBER SPOT CHECKS | 11-27-09

Roof Area contributing to brace frame

Trib. Area = 7.5' x 30'

DL = 48 psf for decking
+3 psf for metal deck

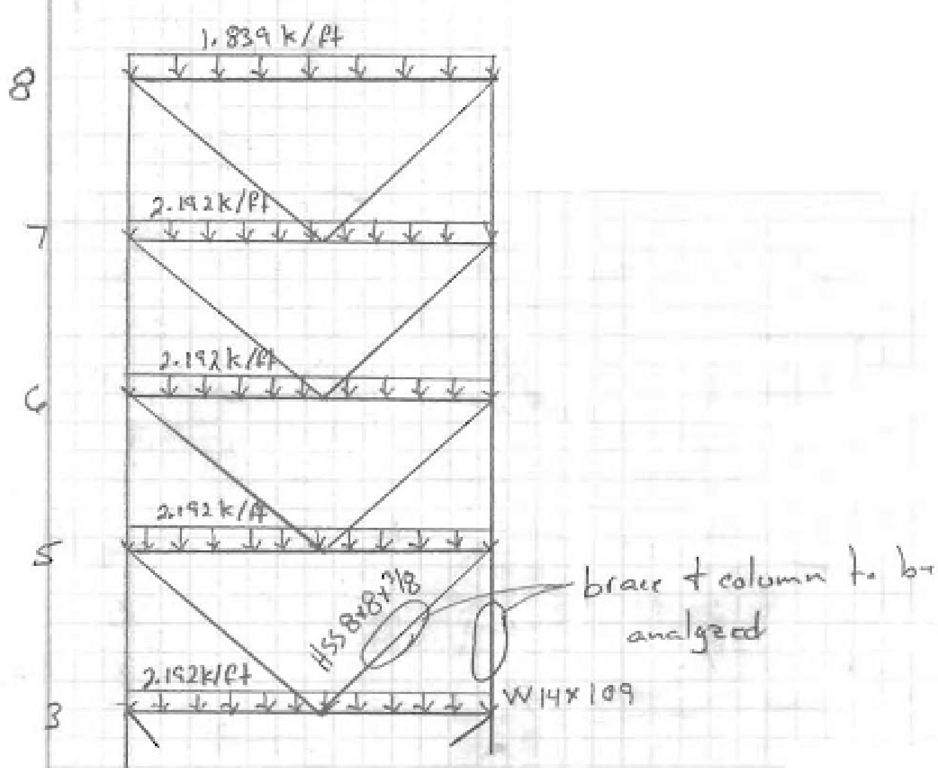
LL = 115 psf

$W_u = (48+3)(7.5)(1.2) + 115(7.5)(1.6)$
$= 1.839\ k/ft$

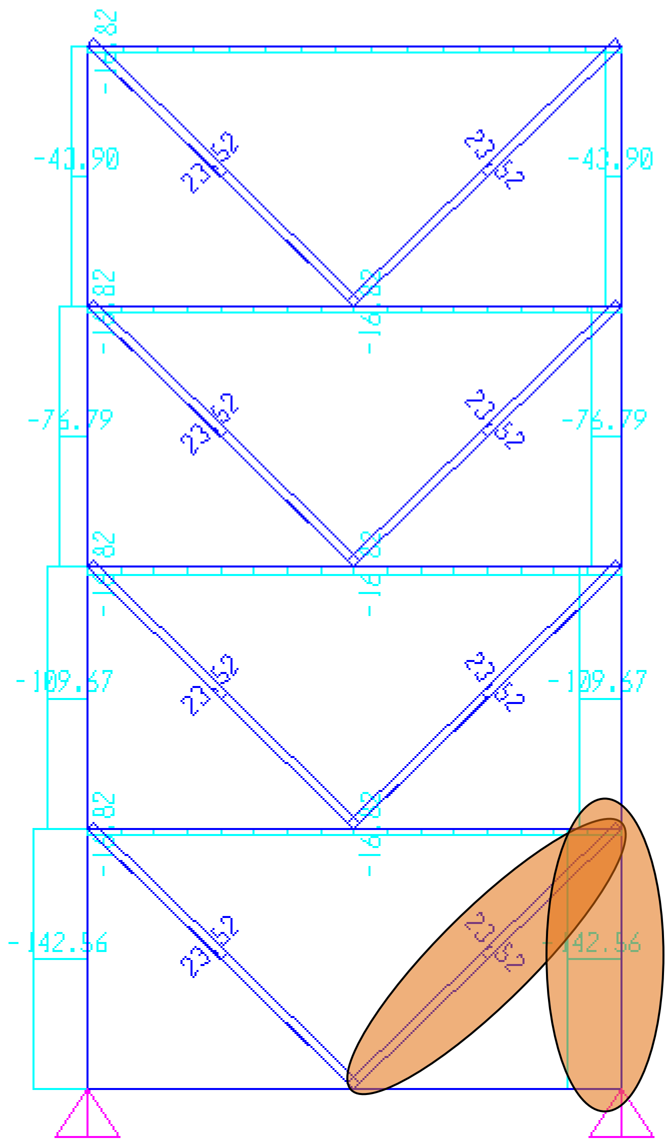
FLoor Area contributing to brace frame
Levels 7, 6 + 5

Tributary Areas = 10' x 30'
DL = 48 + 3 + 25 psf for MEP
LL = 80 psf no reductions

$W_u = 1.2(76)(10') + 1.6(80)(10)$
$= 2.192\ k/ft$



1



Axial loads in columns and braces determined in SAP 2000 for previous page to be added to hand calculated lateral axial load analysis.

BRACE CHECK | TECH III | 11-27-09

LATERAL AXIAL LOAD IN BRACE (HAND CALCULATION)

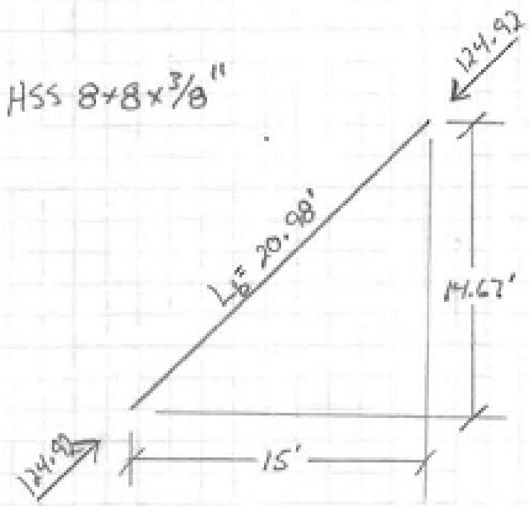
101.4 kips

GRAVITY AXIAL LOAD IN BRACE (FACTORED LIVE + DEAD LOADS)

23.52 kips

Total axial load in brace 124.92 kips

HSS 8x8x3/8"



$F_y = 46$ ksi

$A_g = 10.4 \text{ in}^2$

$r_x = r_y = 3.10$ in

$KL_x/r = 1.0(20.98')(12")/3.1$ in

$= 81.21 < 113$

$\therefore$ use $F_{cr} = 0.658^{(F_y/F_e)}(F_y)$

Limit State $4.71\sqrt{\frac{29000}{46}} = 118.26$

$81.21 < 118.26 \therefore$ inelastic

$F_e = \pi^2 E/(KL_x/r)^2 = \pi^2(29000)/81.21^2 = 43.40$

$F_{cr} = 0.658^{(46/43.4)}(46) = 29.52$ k

$\phi P_n = \phi F_{cr} A_g = 0.9(29.52)(10.4) = 265.68$ kips

ϕP_n from AISC 13 @ $L_b = 21' \Rightarrow 275$k

$\phi P_n \geq P_u$

$265.68 > 124.92 \therefore$ ok

STRESS RATIO

$\frac{P_u}{\phi P_n} = \frac{124.92}{265.68} = 0.47 < 1.0 \therefore$ ok

COLUMN CHECK | TECH III | 11-27-09

W14X109 COMBINED AXIAL LOADS

$P_u \Rightarrow 160.196^k$ From lateral loading

16.34^k From gravity load on columns due to brace tension/compression

$142.56 - [(1.839(30')) + (3 \times 2.192 \times 30')]/2$

4.67^k From gravity load of columns above

$109(14.67') + 74(2)(14.67') + 61(14.67') = 4.67\,k$

From DL + LL for all floors above

Tributary Area $= 30' \times 30' = 900\ ft^2$

Roof $\Rightarrow 1.2(51psf)(900) + 1.6(115)(900) = 220,680\,k$

Floors $\Rightarrow 3[1.2(76)(900) + 1.6(80)900] = 591.840\,k$

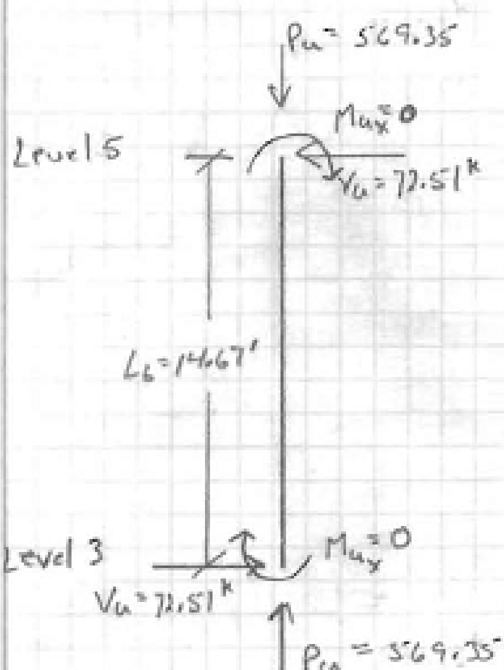
$812.52^k \Rightarrow 220.68 + 591.84 = 812.52\,k$

993.73^k

Load Combinations

$1.2D + 1.6L_r + 0.8W = 374.65$

$1.2D + 1.6W + L + 0.5L_r = 569.35$



$KL = 14.67'(r_x/r_y) = 14.67(1.67)$
$= 24.5'$

ϕP_n From Table 4-1 @ 24.5'
$= 915^k > P_u = 569.35^k \therefore OK$

$\frac{P_u}{\phi P_n} = \frac{569.35}{915} = 0.622 < 1.0$
$\therefore OK$