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Structural Technical Report 3
Lateral System Analysis and Confirmation Design

Introduction

This report is a description and analysis of the lateral force resisting system of the College of Business Administration building that is being built on the Northern Arizona University campus. This four story classroom building with mechanical mezzanine is located in Flagstaff Arizona. It is to become the new home for the College of Business Administration (CBA) as well as become the focal point of a growing campus. The CBA has been designed to attract students and faculty with its eye catching architectural features as well as its state of the art technology systems.

This report is intended to be a detailed analysis of the lateral system of the CBA building. The report includes calculations of wind and seismic loads which will be used to determine if the lateral design of the CBA is sufficient. This report clearly explains how the lateral loads were distributed by relative stiffness of members as well as how to use the forces found to determine the design of the individual members. With help from the PCA design aides, an extensive shear wall check for shear, flexure, and overturning has been included with this report. The shear wall check was more of a design since the size and reinforcing of the wall are not known at this point.

After an exhaustive analysis of the College of Business Administration building, the loads found on the lateral force resisting system were found to be different than those given in the structural drawings from the engineer. It was found that overturning will not be an issue with the CBA, the dead loads on the walls and columns are greater than the uplift force caused by seismic loads. Calculations showed that only the bottom two levels of the south shear wall needed more than the minimum vertical reinforcing to provide enough flexural moment capacity.

Description of the Structural System

The College of Business Administration building is to house the college as well as serve as a classroom building on the Northern Arizona University campus. The CBA utilizes the use of precast concrete as the main structural system for the building. This 110,000 square foot building is made up of 4 above ground stories as well as an upper mezzanine which houses the mechanical equipment. The roof system is mostly comprised of structural steel.

Foundation

The foundation of the College of Business Administration consists of caissons, grade beams, and continuous footings. The caissons are located beneath the columns and range in size from 2'6" diameter to 7' diameter with the largest located beneath the central columns along column line C. In addition to the caissons, the CBA utilizes grade beams and continuous footings. The caissons will be the most important when looking at the lateral system, as they will help to avoid overturning of the structure.

Floor Framing

The ground floor is composed of a 4" slab on grade on top of 4" of aggregate base course fill. The 2nd, 3rd, and 4th floors are composed of 10" hollow core planks spanning 36 feet with a 3" concrete topping. In the upper floors, the hollow core planks will bear on precast concrete beams. The beams span in the north-south direction and are inverted t-beams in interior bays and l-shaped around the perimeter. There are also rectangular beams that span in the east-west direction along column line C. These rectangular beams are part of braced frames which will be explored later in this report. All of the columns throughout the building are 24" square precast columns. Refer to the Appendix for a typical layout of the columns.

Roof Framing

The roof of the College of Business Administration building is constructed using structural steel. A mixture of W shaped members and open web joists are used. Due to the upper mezzanine, there are roofs at two different levels which both slope towards the edge of the building. The lower roof is broken into two sections since the mezzanine is through the middle of the building. The joists are covered with 1-1/2 " deep painted steel deck on the lower roofs. The upper roof has W30x116 beams spanning in the N-S direction. The E-W direction has four rows of steel I beams. This upper roof has a 3-1/2" deep acoustical steel deck running in the N-S direction.

Lateral Load Resisting System

The lateral load resisting system of the College of Business Administration incorporates shear walls, braced frames, and moment frames. The shear walls are precast walls that are assumed to be 8in thick for this assignment. Some assumptions were made in the geometry of the building to make the calculations easier. The moment frames along column lines A and B were treated as they were parallel to the south face of the building, this should only cause very minimal changes in the results. Other assumptions that may have been made have been documented within the calculations located in the appendix of this report. There are four shear walls in the North-South direction and one in the East West direction. Due to the architecture, shear walls could not be placed throughout the building so moment and braced frames were incorporated in the lateral design. There are three braced frames that are all the same and located along column line C. These frames are made up of the 24" square concrete columns, 24"x 26" precast beams at the floor levels, a W24x 68 at the roof level and 8in steel pipes as braces. Also used for lateral support, are moment frames along column lines A, B, 4 and 5. Refer to lateral member plan in the attached appendix. These moment frames are 24" precast concrete columns with W shaped steel at the roof level.

Gravity and Lateral Loads

Lateral Loads:

The lateral loads are determined by using ASCE 7-02 and given data.

- Soil Site Class C
- $S_s = 0.46$
- $S_1 = 0.13$

SEISMIC

Level, x	w_x	h_x	$w_x h_x^k$	C_{vx}	F_x	V_x	M_x
	(kips)	(ft)			(kips)	(kips)	(ft-kips)
Roof	723	60	41,983	0.1137	92		5,502
5	750	54.5	39,589	0.129	86	92	4,713
4	2765	42	112,702	0.366	246	178	10,339
3	2765	28	75,378	0.245	165	424	4,610
2	2765	14	37,899	0.123	83	589	1,159
1						672	
	Σ = 11222		Σ = 317,829	Σ = 1.000	Σ = 672		Σ = 26,323

- 3 Second Wind Gust = 90 MPH
- Exposure C
- Importance Factor I = 1.15

WIND				
Level	PLF	F_x	V_x	M_x
Roof	201	50.7	0	2937.8
5	423.2	106.6	50.7	5119.0
4	372.3	93.8	157.3	3377.5
3	342.4	86.3	251.2	2070.8
2	318.9	80.4	337.4	964.4
1	0	0.0	417.8	0.0
		Σ =		Σ =
		417.8		14469.5

Story Forces	
Roof	92
5th	86
4th	246
3rd	165
2nd	83
Base	-

The tables above show findings from a seismic analysis as well as from a wind analysis. It has been concluded that the seismic forces will control the design of the lateral force resisting systems. The Table to the left shows the story forces that will be used to analyze the lateral force resisting system.

Gravity Loads

Roof:

- Roof Live Load = 31.5 PSF (Based on 45 PSF Ground Snow Load).
- Roof Dead Load = 20 PSF.

Floors:

- Floor Live Load = 100 PSF (Reducible).
- Floor Dead Load = 138 PSF

Distribution of Lateral Forces

The seismic loads calculated above were distributed to the various lateral force resisting members by proportional stiffness. Each member was individually modeled in Ram Advanse 6.0. The stiffness of each member was calculated by determining the displacement caused by a one kip force, and taking the reciprocal. This was done multiple times in order to compile stiffnesses at each level for each individual member. Since load follows stiffness, proportions of load at each level were found by determining the proportion of stiffness each member had to the total amount at that level. Using these proportions, the direct shear in each wall and frame were found at each floor level.

In addition to direct shear, the members experience eccentric shear since the story forces act at the center of rigidity not the center of mass. This will cause a twisting action throughout the building. To determine the magnitude of the torsion shear in the walls, the center of rigidity must be found at each level. After finding the CoR, The torsional shear can be found and added to the direct shear.

<u>Analysis</u>

As part of the structural drawings, a page was included which identified the braced frame and shear wall loads. This page has been included in the appendix of this report to provide a comparison to the loads found be the aforementioned procedure. Some of the loads shown in the drawings seems to be similar to the loads calculated in this report, whereas others are off by quite a lot. There are many reasons for this. The dimensions of the east, west, grid 4 & 5 shear walls were modeled as rectangles which is not exactly what they are. This could have caused some of the discrepancies in the numbers. Also, the auditorium which is part of the building was not considered since it is only two stories tall and may be separated from the main building by an expansion joint. The seismic loads found are larger than those usually seen on the east coast, but the CBA is located in a higher risk area.

<u>Member Checks</u>

Once the shears are found, the amount of reinforcing needed in the wall. A detailed calculation to find the amount of vertical and horizontal reinforcing needed can be found in the appendix of the report. The calculation followed the design methods presented in the PCS book "*Simplified Design – Reinforced Concrete Buildings of Moderate Size and Height*." A design was done for shear and was followed by a design for flexure. The table shown illustrates what reinforcing is needed for both the vertical and horizontal directions in each floor level. As part of the calculation, an overturning check was performed. The check showed that the wall will not overturn due to the self weight of the wall and the dead loads of the floor which frame into the columns. Since the size and reinforcing of the shear wall is not known, the check was performed like a design problem.

REINFORCING PER FLOOR	VERTICAL	HORIZONTAL
1ST FLOOR	#4 @ 10"	#5 @ 5"
2ND FLOOR	#4 @ 10"	#5 @ 10"
3RD FLOOR	#4 @ 10"	#4 @ 10"
4TH FLOOR	#4 @ 10"	#4 @ 10"
5TH FLOOR	#4 @ 10"	#4 @ 10"

Summary

The lateral system of the College of Business Administration building was designed using shear walls, braced frames and moment frames. While the gravity system has only three different size beams and is very typical, the lateral design is much more involved. The report above describes a procedure in which the lateral loads are found and distributed to the lateral force resisting members by stiffness. A short check found the building will not overturn even under the high seismic loads that it could experience in its lifetime. The forces found by the analysis prescribed we not shown to be the same as the loads shown in the drawings. The differences may have occurred because the building was not modeled exactly the way it was designed. Other reasons for differences could be that the seismic loads found were not entirely correct or that the building was designed using a different method than what was used in this report.

Appendix

Structural Technical Report 3

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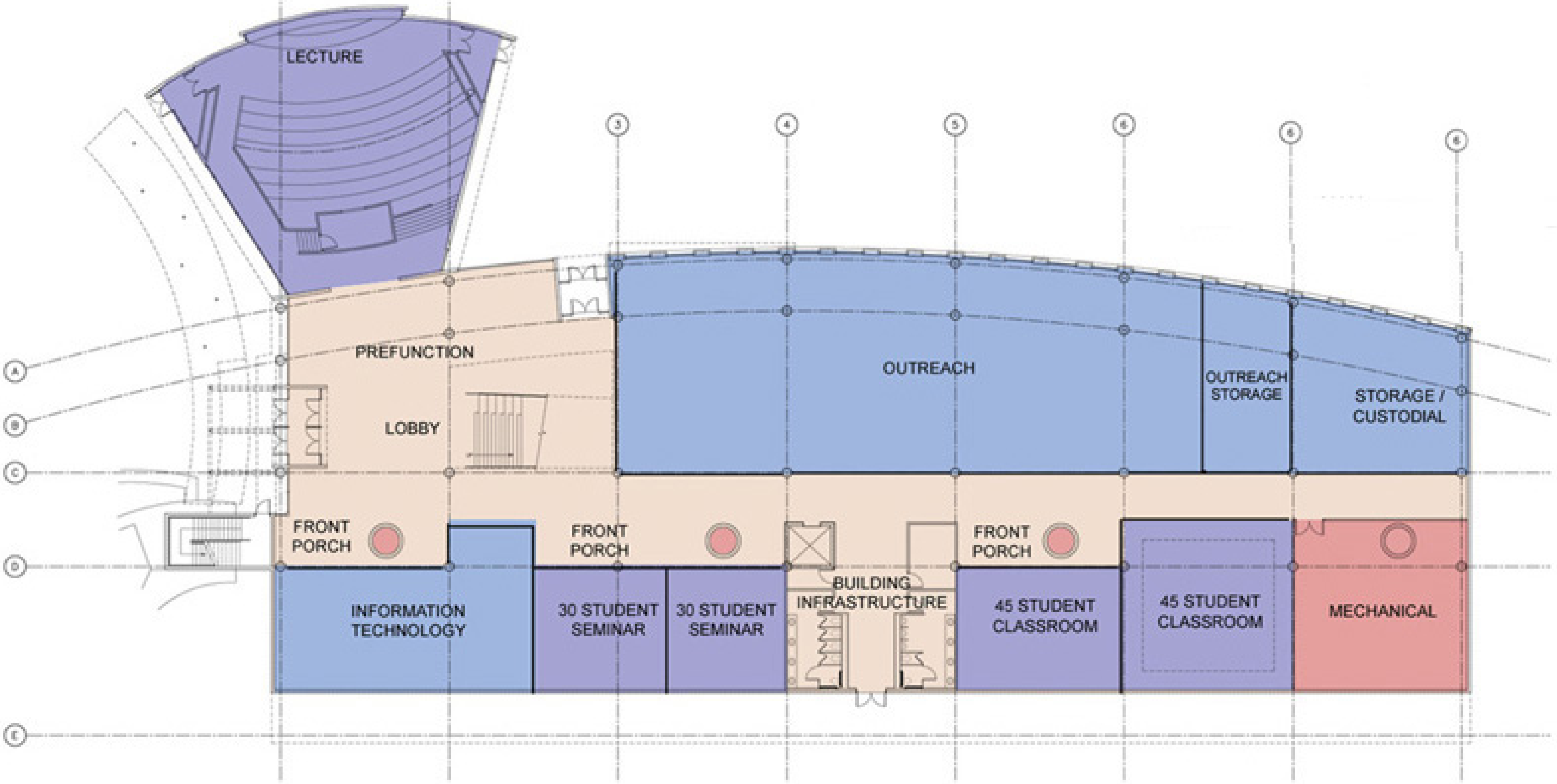
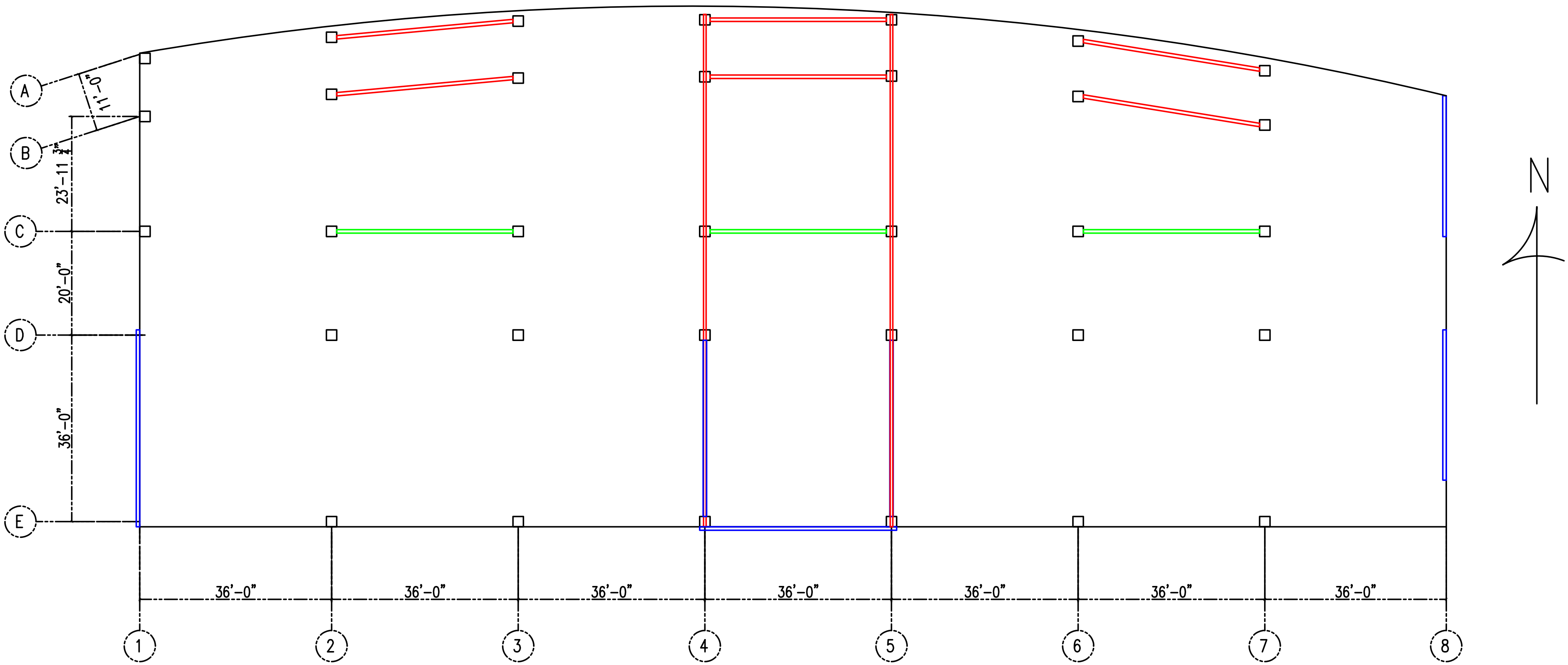
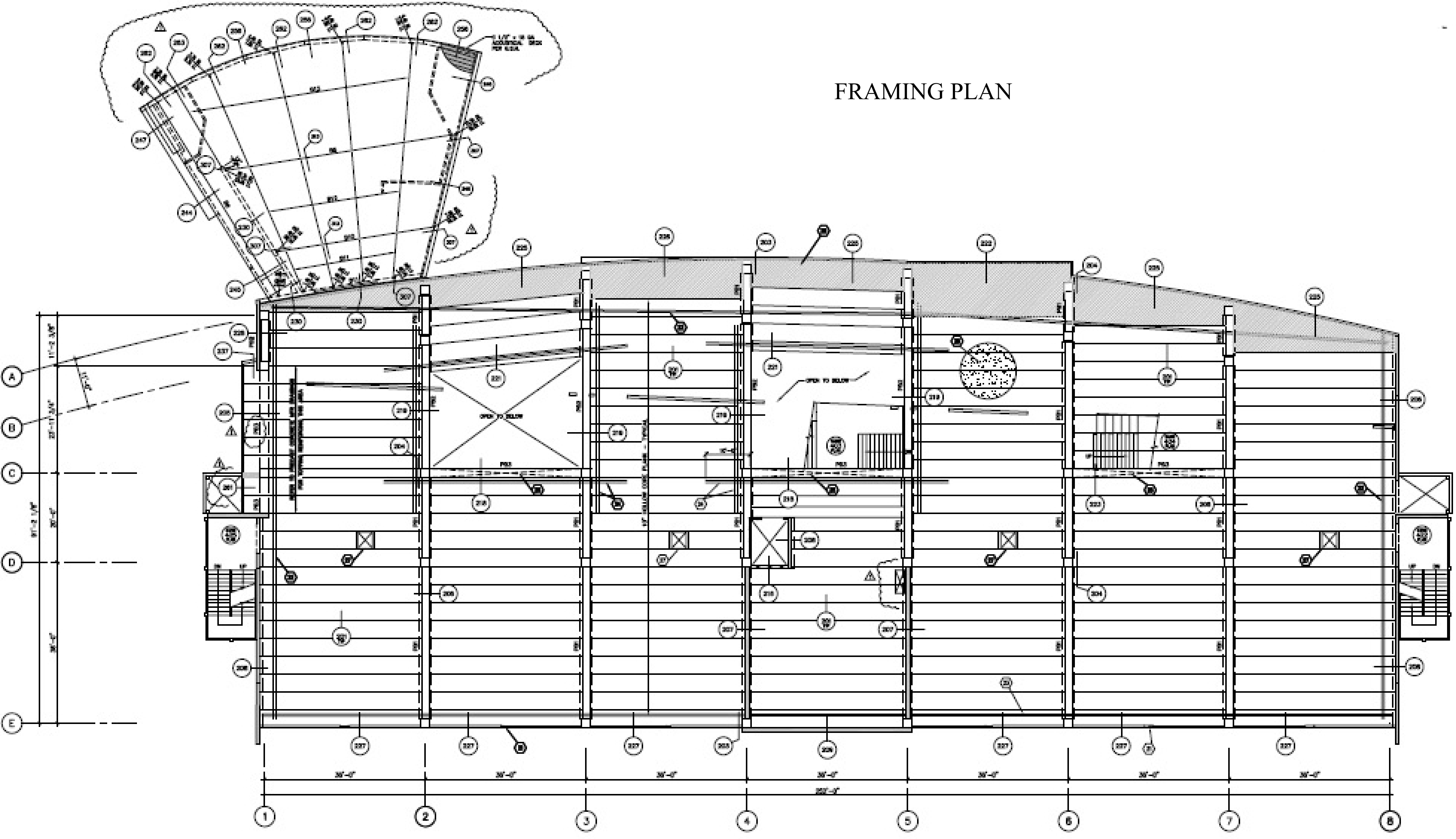

LECTURE
PREFUNCTION
LOBBY
OUTREACH
OUTREACH STORAGE
STORAGE / CUSTODIAL
FRONT PORCH
FRONT PORCH
FRONT PORCH
BUILDING INFRASTRUCTURE
INFORMATION TECHNOLOGY
30 STUDENT SEMINAR
30 STUDENT SEMINAR
45 STUDENT CLASSROOM
45 STUDENT CLASSROOM
MECHANICAL



COLUMN LAYOUT OF COLLEGE OF BUSINESS ADMINISTRATION BUILDING

FRAMING PLAN



Distribution of Forces

Story Forces	
Roof	92
5th	178
4th	424
3rd	589
2nd	672
Base	-

Wall and Frame Stiffness

N-S Direction

	South SW	Grid C BF (x3)	Grid A MF (x3)	Grid B MF (x3)	Sum
Roof	1165.5	703.9	0	235.6	2105
5th Floor	1689.2	1007	238.3	235.7	3170
4th Floor	3164.6	1443	242.3	240.2	5090
3rd Floor	6802.7	2531.6	258.4	268.7	9861
2nd Floor	0	5357.1	291.8	324.7	5974
Base					

South Shear Wall		
	Proportion	Shear
Roof	0.55	50.9
5th Floor	0.53	94.8
4th Floor	0.62	263.6
3rd Floor	0.69	406.3
2nd Floor	0.00	419.5
Base	0.00	-

Grid C Braced Frame		
	Proportion	Shear
Roof	0.33	30.8
5th Floor	0.32	56.5
4th Floor	0.28	120.2
3rd Floor	0.26	151.2
2nd Floor	0.90	226.5
Base	0.00	-

Grid A Moment Frame		
	Proportion	Shear
Roof	0.00	-
5th Floor	0.08	13
4th Floor	0.05	20
3rd Floor	0.03	15
2nd Floor	0.05	12
Base	0.00	0

Grid B Moment Frame		
	Proportion	Shear
Roof	0.11	10.3
5th Floor	0.07	13.2
4th Floor	0.05	20.0
3rd Floor	0.03	16.0
2nd Floor	0.05	13.7
Base	0.00	-

E-W Direction

	East SW1	East SW2	West SW	Grid 4 SW	Grid 5 SW	Grid 4 MF	Grid 5 MF	Sum
Roof	414.3	597	597	597	597	44.8	44.8	2892
5th Floor	685.9	793	793	793	793	45.6	45.6	3949
4th Floor	1383.1	1543.2	1543.2	1543.2	1543.2	50.5	50.5	7657
3rd Floor	3690	3663	3663	3663	3663	59.9	59.9	18462
2nd Floor	12454.9	8771.9	8771.9	8771.9	8771.9	74.2	74.2	47691
Base								

East Shear Wall 1		
	Proportion	Shear
Roof	0.14	13.18
5th Floor	0.17	30.92
4th Floor	0.18	76.59
3rd Floor	0.20	117.72
2nd Floor	0.26	175.50
Base	0.00	-

East Shear Wall 2		
	Proportion	Shear
Roof	0.21	19.0
5th Floor	0.20	35.7
4th Floor	0.20	85.5
3rd Floor	0.20	116.9
2nd Floor	0.18	123.6
Base	0.00	-

West Shear Wall		
	Proportion	Shear
Roof	0.21	19.0
5th Floor	0.20	35.7
4th Floor	0.20	85.5
3rd Floor	0.20	116.9
2nd Floor	0.18	123.6
Base	0.00	-

Grid 4 Shear Wall		
	Proportion	Shear
Roof	0.21	19.0
5th Floor	0.20	35.7
4th Floor	0.20	85.5
3rd Floor	0.20	116.9
2nd Floor	0.18	123.6
Base	0.00	-

Grid 5 Shear Wall		
	Proportion	Shear
Roof	0.21	19.0
5th Floor	0.20	35.7
4th Floor	0.20	85.5
3rd Floor	0.20	116.9
2nd Floor	0.18	123.6
Base	0.00	-

Grid 4 Moment Frame		
	Proportion	Shear
Roof	0.02	1.4
5th Floor	0.01	2.1
4th Floor	0.01	2.8
3rd Floor	0.00	1.9
2nd Floor	0.00	1.0
Base	0.00	-

Grid 5 Moment Frame		
	Proportion	Shear
Roof	0.02	1.4
5th Floor	0.01	2.1
4th Floor	0.01	2.8
3rd Floor	0.00	1.9
2nd Floor	0.00	1.0
Base	0.00	-

2nd Floor

Center of Rigidity

Wall/Frame	R	y	Rx
South SW	0	0	0
Grid C BF (x3)	5357	56	299998
Grid A MF (x3)	292	97	28305
Grid B MF (x3)	325	86	27924
Wall/Frame	R	x	Ry
East SW1	12455	0	0
East SW2	8772	0	0
West SW	8772	252	2210519
Grid 4 SW	8772	108	947365
Grid 5 SW	8772	144	1263154
Grid 4 MF	74	108	8014
Grid 5 MF	74	144	10685

Center of Rigidity N-S Direction = **59.6** (From South Side)

Center of Rigidity E-W Direction = **93.1** (From West Side)

Torsional Moment due to Eccentricities

T (E-W) = 22113

Controlling T **22113**

T (N-S) = 7146

Wall/Frame	R	x	Rx^2	$Rx/\Sigma Rx^2$	Torsional Shear
South SW	0	59.6	0	0.000	0.00
Grid C BF (x3)	5357	3.6	70724	0.000	1.00
Grid A MF (x3)	292	37.4	407428	0.000	0.56
Grid B MF (x3)	325	26.4	225730	0.000	0.44
Wall/Frame	R	x			0.00
East SW1	12455	93.1	107940270	0.003	59.48
East SW2	8772	93.1	76021586	0.002	41.89
West SW	8772	158.9	221500309	0.003	71.51
Grid 4 SW	8772	14.9	1949022	0.000	6.71
Grid 5 SW	8772	50.9	22731697	0.001	22.91
Grid 4 MF	74	14.9	16486	0.000	0.06
Grid 5 MF	74	50.9	192284	0.000	0.19

3rd Floor

Center of Rigidity

Wall/Frame	R	y	Rx
South SW	6802.7	0	0
Grid C BF (x3)	2531.6	56	141769.6
Grid A MF (x3)	258.4	97	25064.8
Grid B MF (x3)	268.7	86	23108.2
Wall/Frame	R	x	Ry
East SW1	3690	0	0
East SW2	3663	0	0
West SW	3663	252	923076
Grid 4 SW	3663	108	395604
Grid 5 SW	3663	144	527472
Grid 4 MF	59.9	108	6469.2
Grid 5 MF	59.9	144	8625.6

Center of Rigidity N-S Direction = **19.3** (From South Side)

Center of Rigidity E-W Direction = **100.8** (From West Side)

Torsional Moment due to Eccentricities

T (E-W) = 14833.31

Controlling T **17516.14**

T (N-S) = 17516.14

Wall/Frame	R	x	Rx^2	$Rx/\Sigma Rx^2$	Torsional Shear
South SW	6802.7	19.3	2523765.073	0.0008	13.2
Grid C BF (x3)	2531.6	36.7	3416996.513	0.0005	9.3
Grid A MF (x3)	258.4	77.7	1561593.329	0.0001	2.0
Grid B MF (x3)	268.7	66.7	1196807.181	0.0001	1.8
Wall/Frame	R	x			0.0
East SW1	3690	100.8	37504744.45	0.0021	37.4
East SW2	3663	100.8	37230319.49	0.0021	37.1
West SW	3663	151.2	83723614.34	0.0032	55.7
Grid 4 SW	3663	7.2	189041.2832	0.0002	2.6
Grid 5 SW	3663	43.2	6830940.548	0.0009	15.9
Grid 4 MF	59.9	7.2	3091.338483	0.0000	0.0
Grid 5 MF	59.9	43.2	111704.4332	0.0000	0.3

4th Floor

Center of Rigidity

Wall/Frame	R	y	Rx
South SW	3165	0	0
Grid C BF (x3)	1443	56	80808
Grid A MF (x3)	242	97	23503
Grid B MF (x3)	240	86	20657
Wall/Frame	R	x	Ry
East SW1	1383	0	0
East SW2	1543	0	0
West SW	1543	252	388886
Grid 4 SW	1543	108	166666
Grid 5 SW	1543	144	222221
Grid 4 MF	51	108	5454
Grid 5 MF	51	144	7272

Center of Rigidity N-S Direction = **24.6** (From South Side)

Center of Rigidity E-W Direction = **103.2** (From West Side)

Torsional Moment due to Eccentricities

T (E-W) = 9650.2

Controlling T **10366.3**

T (N-S) = 10366.3

Wall/Frame	R	x	Rx^2	$Rx/\Sigma Rx^2$	Torsional Shear
South SW	3165	24.6	1907506	0.0011	11.0
Grid C BF (x3)	1443	31.4	1427162	0.0006	6.4
Grid A MF (x3)	242	72.4	1271790	0.0002	2.5
Grid B MF (x3)	240	61.4	906983	0.0002	2.1
Wall/Frame	R	x			0.0
East SW1	1383	103.2	14741785	0.0019	20.1
East SW2	1543	103.2	16448212	0.0022	22.5
West SW	1543	148.8	34150276	0.0031	32.4
Grid 4 SW	1543	4.8	34964	0.0001	1.0
Grid 5 SW	1543	40.8	2563831	0.0009	8.9
Grid 4 MF	51	4.8	1144	0.0000	0.0
Grid 5 MF	51	40.8	83899	0.0000	0.3

5th Floor

Center of Rigidity

Wall/Frame	R	y	Rx
South SW	1689.2	0	0
Grid C BF (x3)	1007	56	56392
Grid A MF (x3)	238.3	97	23115.1
Grid B MF (x3)	235.7	86	20270.2
Wall/Frame	R	x	Ry
East SW1	685.9	0	0
East SW2	793	0	0
West SW	793	252	199836
Grid 4 SW	793	108	85644
Grid 5 SW	793	144	114192
Grid 4 MF	45.6	108	4924.8
Grid 5 MF	45.6	144	6566.4

Center of Rigidity N-S Direction = **31.5** (From South Side)

Center of Rigidity E-W Direction = **104.1** (From West Side)

Torsional Moment due to Eccentricities

T (E-W) = 3895.4

Controlling T **3895.4**

T (N-S) = 3119.7

Wall/Frame	R	x	Rx^2	$Rx/\Sigma Rx^2$	Torsional Shear
South SW	1689.2	31.5	1673290.112	0.0014	5.3
Grid C BF (x3)	1007	24.5	605759.8862	0.0006	2.5
Grid A MF (x3)	238.3	65.5	1023193.899	0.0004	1.6
Grid B MF (x3)	235.7	54.5	700768.8265	0.0003	1.3
Wall/Frame	R	x			0.0
East SW1	685.9	104.1	7435206.167	0.0018	7.2
East SW2	793	104.1	8596178	0.0021	8.3
West SW	793	147.9	17342731.17	0.0030	11.8
Grid 4 SW	793	3.9	11964.78764	0.0001	0.3
Grid 5 SW	793	39.9	1261472.383	0.0008	3.2
Grid 4 MF	45.6	3.9	688.0130094	0.0000	0.0
Grid 5 MF	45.6	39.9	72538.63894	0.0000	0.2

Roof

Center of Rigidity

Wall/Frame	R	y	Rx
South SW	1166	0	0
Grid C BF (x3)	704	56	39418
Grid A MF (x3)	0	97	0
Grid B MF (x3)	236	86	20262
Wall/Frame	R	x	Ry
East SW1	414	0	0
East SW2	597	0	0
West SW	597	252	150444
Grid 4 SW	597	108	64476
Grid 5 SW	597	144	85968
Grid 4 MF	45	108	4838
Grid 5 MF	45	144	6451

Center of Rigidity N-S Direction = **28.4** (From South Side)

Center of Rigidity E-W Direction = **107.9** (From West Side)

Torsional Moment due to Eccentricities

T (E-W) = 1660.7

Controlling T **1899.7**

T (N-S) = 1899.7

Wall/Frame	R	x	Rx^2	$Rx/\Sigma Rx^2$	Torsional Shear
South SW	1165.5	28.4	936840.6062	0.0012	2.3
Grid C BF (x3)	703.9	27.6	538087.2904	0.0007	1.4
Grid A MF (x3)	0	68.6	0	0.0000	0.0
Grid B MF (x3)	235.6	57.6	782979.9608	0.0005	0.9
Wall/Frame	R	x			0.0
East SW1	414.3	107.9	4827828.851	0.0016	3.1
East SW2	597	107.9	6956827.96	0.0024	4.5
West SW	597	144.1	12388169.02	0.0032	6.0
Grid 4 SW	597	0.1	1.555179431	0.0000	0.0
Grid 5 SW	597	36.1	775907.4203	0.0008	1.5
Grid 4 MF	44.8	0.1	0.116703582	0.0000	0.0
Grid 5 MF	44.8	36.1	58225.54845	0.0001	0.1

South Shear Wall			
	Concentric Shear	Eccentric Shear	Total Shear
Roof	51	2.3	53.2
5th Floor	95	5.3	100.2
4th Floor	264	11.0	274.6
3rd Floor	406	13.2	419.5
2nd Floor	419	0.0	419.5

Grid C Braced Frame (Each)			
	Concentric Shear	Eccentric Shear	Total Shear
Roof	10	0.5	10.7
5th Floor	19	0.8	19.7
4th Floor	40	6.4	46.5
3rd Floor	50	3.1	53.5
2nd Floor	75	0.3	75.8

Grid A Moment Frame (Each)			
	Concentric Shear	Eccentric Shear	Total Shear
Roof	0	0.0	0.0
5th Floor	4	0.5	5.0
4th Floor	7	0.8	7.6
3rd Floor	5	0.7	5.8
2nd Floor	4	0.2	4.3

Grid B Moment Frame (Each)			
	Concentric Shear	Eccentric Shear	Total Shear
Roof	3	0.3	3.7
5th Floor	4	0.4	4.8
4th Floor	7	0.7	7.4
3rd Floor	5	0.6	6.0
2nd Floor	5	0.1	4.7

East Shear Wall 1			
	Concentric Shear	Eccentric Shear	Total Shear
Roof	13	3.1	16.3
5th Floor	31	7.2	38.1
4th Floor	77	20.1	96.7
3rd Floor	118	37.4	155.1
2nd Floor	175	59.5	235.0

East Shear Wall 2			
	Concentric Shear	Eccentric Shear	Total Shear
Roof	19.0	4.5	23.5
5th Floor	35.7	8.3	44.0
4th Floor	85.5	22.5	107.9
3rd Floor	116.9	37.1	154.0
2nd Floor	123.6	41.9	165.5

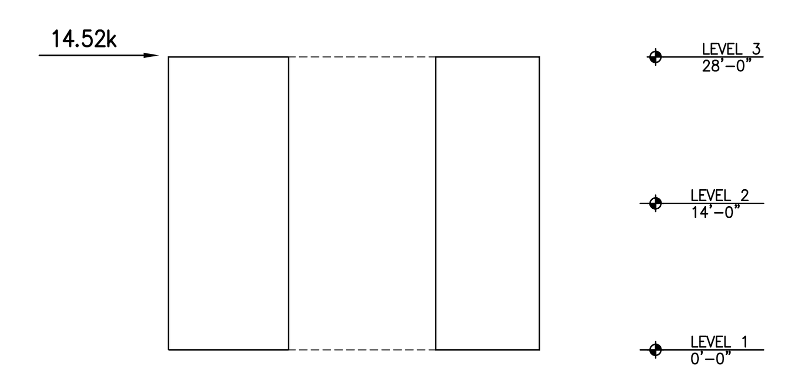
West Shear Wall			
	Concentric Shear	Eccentric Shear	Total Shear
Roof	19.0	6.0	25.0
5th Floor	35.7	11.8	47.5
4th Floor	85.5	32.4	117.8
3rd Floor	116.9	55.7	172.5
2nd Floor	123.6	71.5	195.1

Grid 4 Shear Wall			
	Concentric Shear	Eccentric Shear	Total Shear
Roof	19.0	0.0	19.0
5th Floor	35.7	0.3	36.1
4th Floor	85.5	1.0	86.5
3rd Floor	116.9	2.6	119.5
2nd Floor	123.6	6.7	130.3

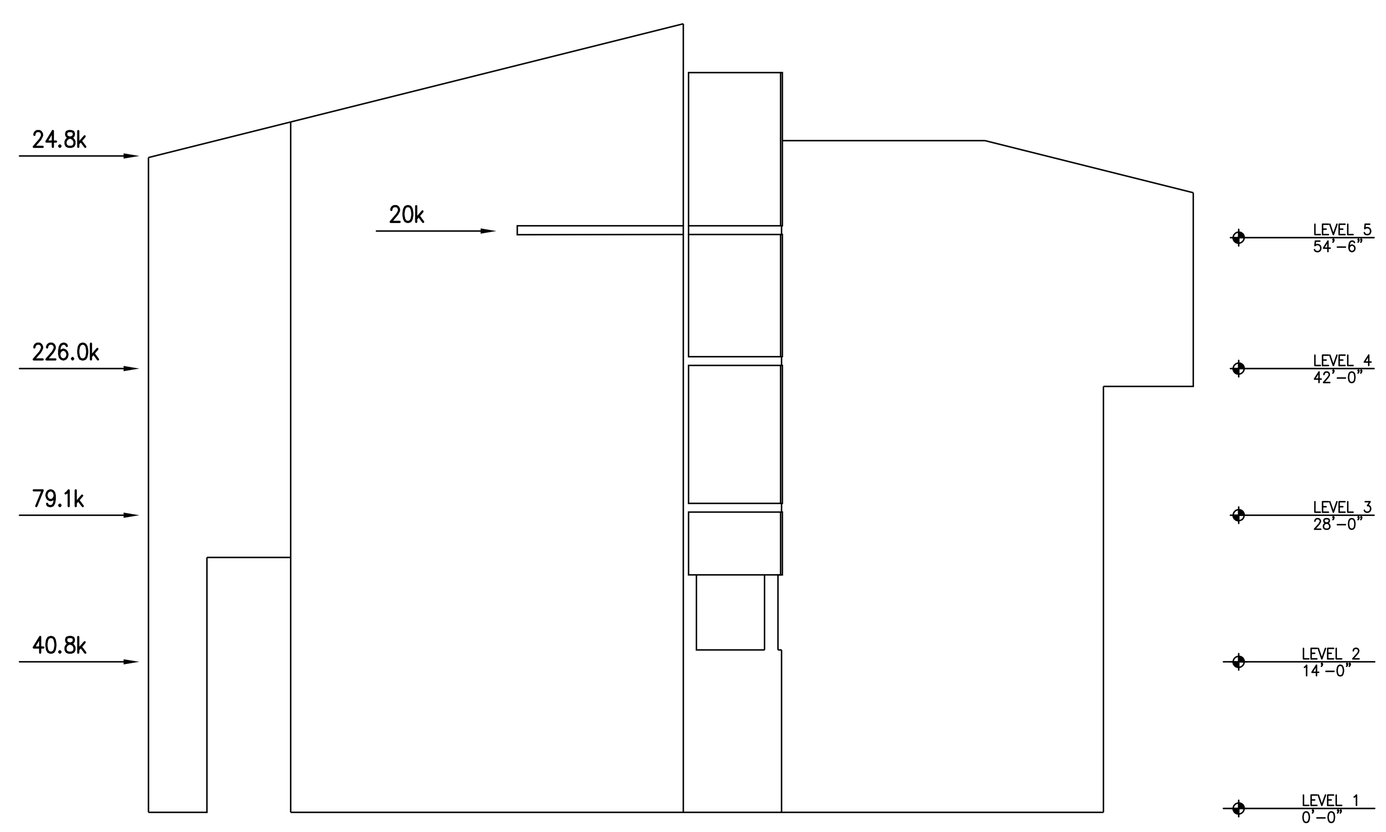
Grid 5 Shear Wall			
	Concentric Shear	Eccentric Shear	Total Shear
Roof	19.0	1.5	20.5
5th Floor	35.7	3.2	38.9
4th Floor	85.5	8.9	94.3
3rd Floor	116.9	15.9	132.8
2nd Floor	123.6	22.9	146.5

Grid 4 Moment Frame (Each)			
	Concentric Shear	Eccentric Shear	Total Shear
Roof	0.5	0.0	0.5
5th Floor	0.7	0.0	0.7
4th Floor	0.9	0.0	0.9
3rd Floor	0.6	0.0	0.7
2nd Floor	0.3	0.0	0.4

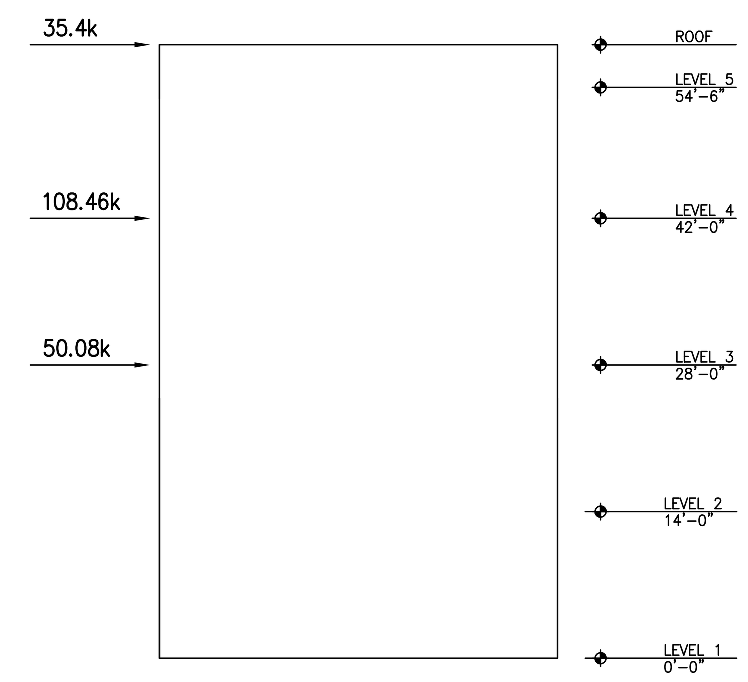
Grid 4 Moment Frame (Each)			
	Concentric Shear	Eccentric Shear	Total Shear
Roof	0.5	0.0	0.5
5th Floor	0.7	0.1	0.7
4th Floor	0.9	0.1	1.0
3rd Floor	0.6	0.1	0.7
2nd Floor	0.3	0.1	0.4



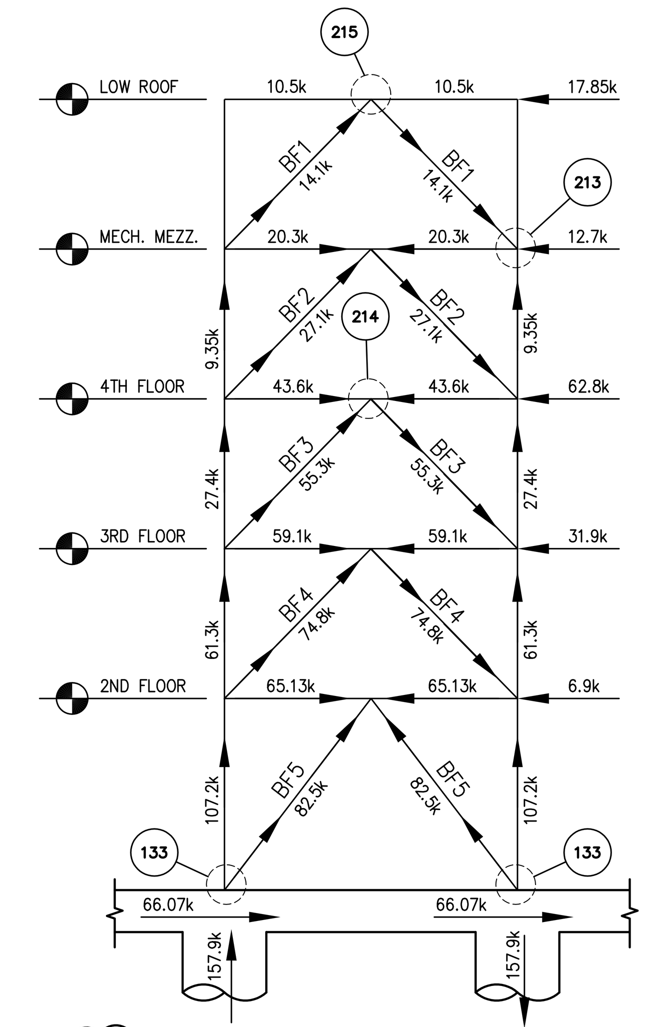
LECTURE HALL – SOUTH SHEAR WALL



EAST SHEAR WALL



SOUTH SHEAR WALL



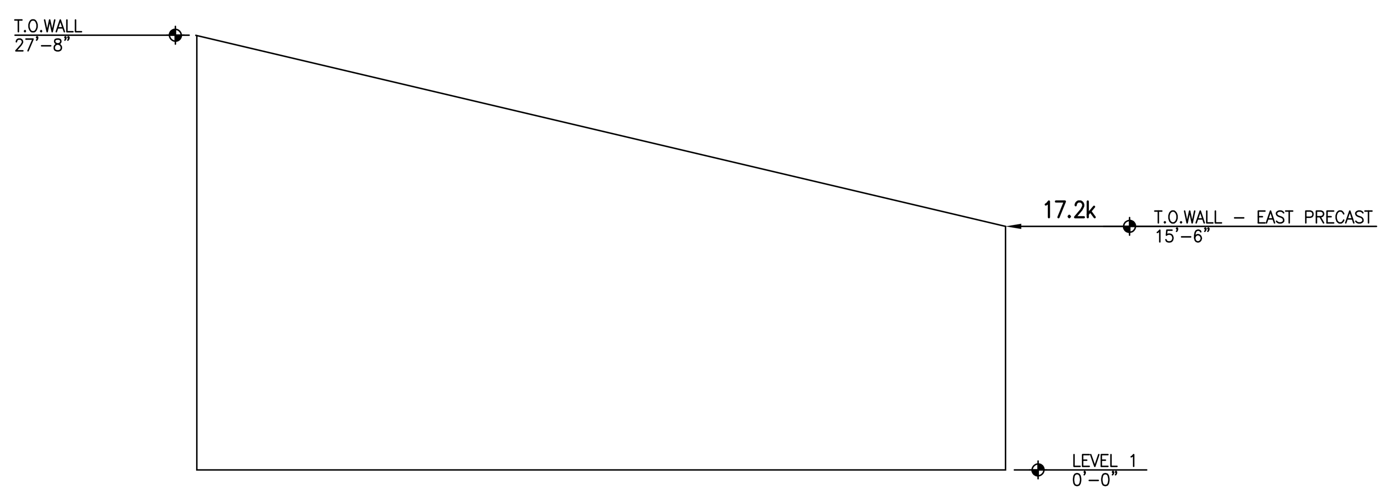
NOTE: ALL LOADS ARE SERVICE LOADS FROM SEISMIC

TYPICAL BRACED FRAME

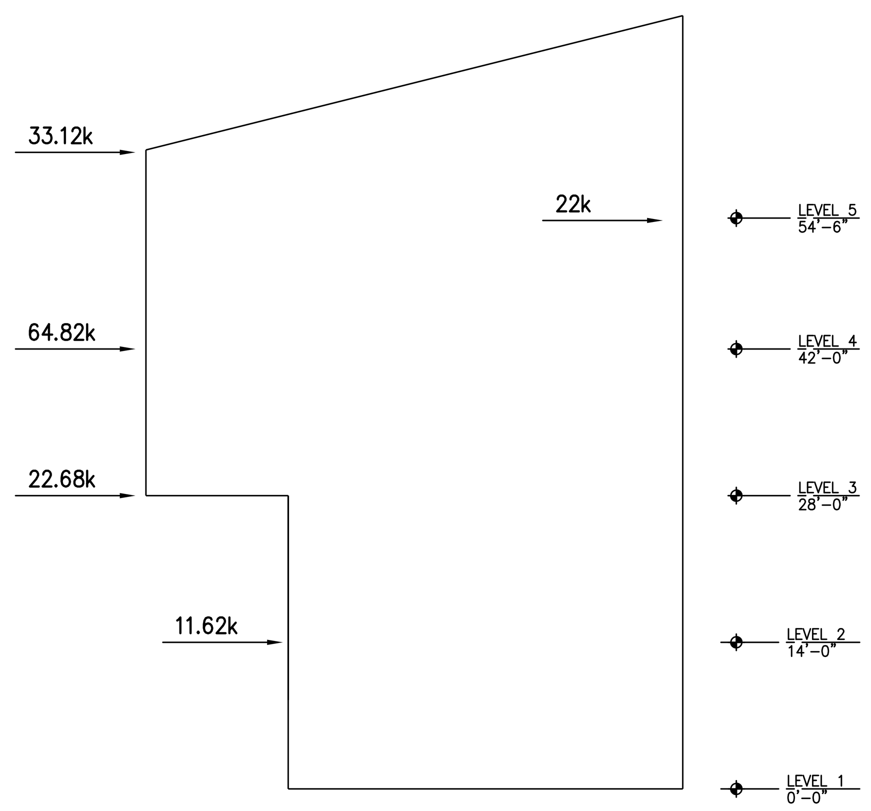
BRACED FRAME (BF) SCHEDULE

MARK	SIZE	COMPRESSION FORCE (KIPS)	TENSION FORCE (KIPS)	REMARKS
BF1	8" STD PIPE	13.0	12.5	---
BF2	8" STD PIPE	18.5	13.0	---
BF3	8" STD PIPE	43.5	32.0	---
BF4	8" EXTRA STRONG PIPE	62.0	43.0	---
BF5	8" EXTRA STRONG PIPE	80.0	54.0	---

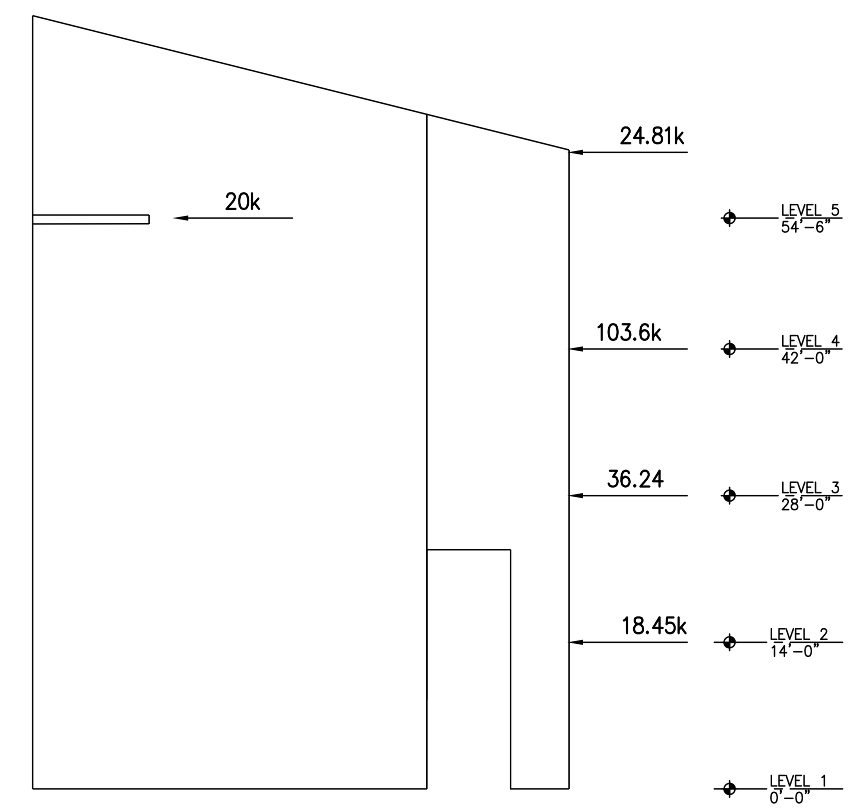
NOTE:
ALL FORCES ARE WORKING STRESS.



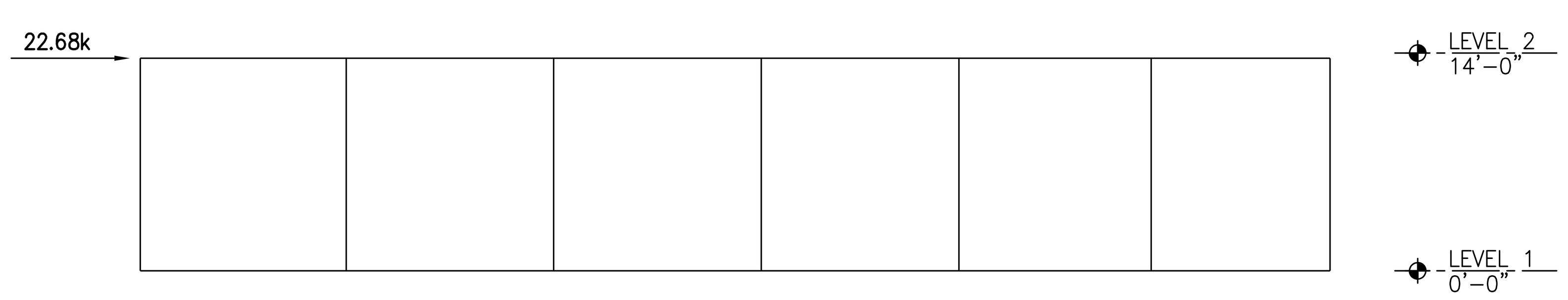
LECTURE HALL – EAST SHEAR WALL



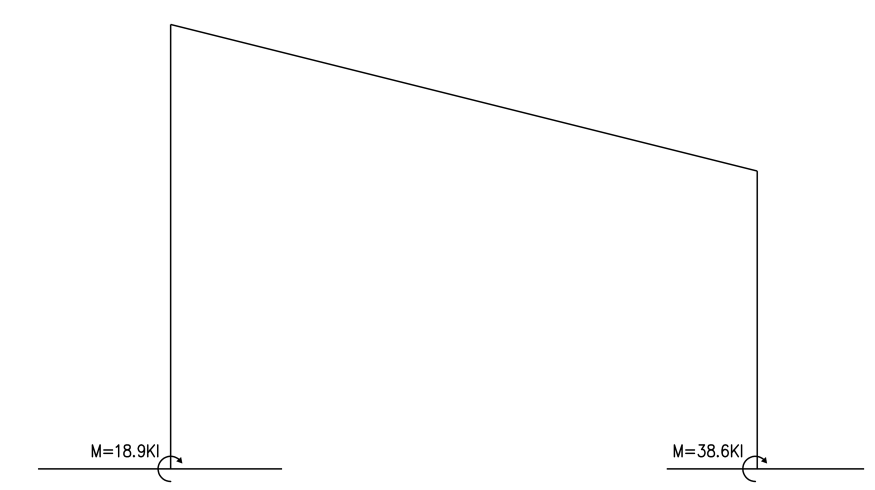
SHEAR WALLS AT GRIDS 4 & 5



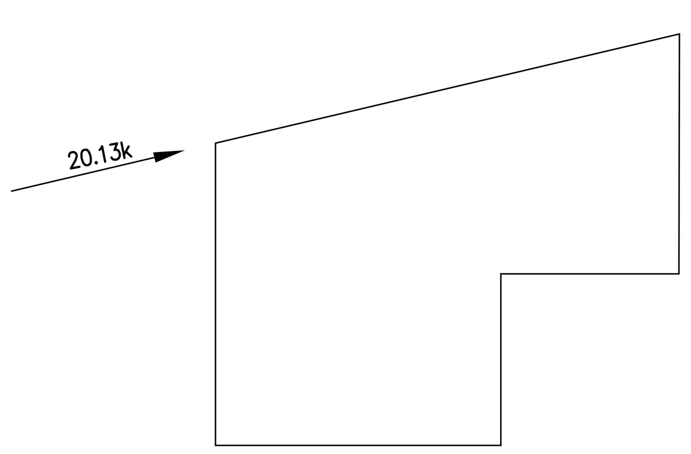
WEST SHEAR WALL



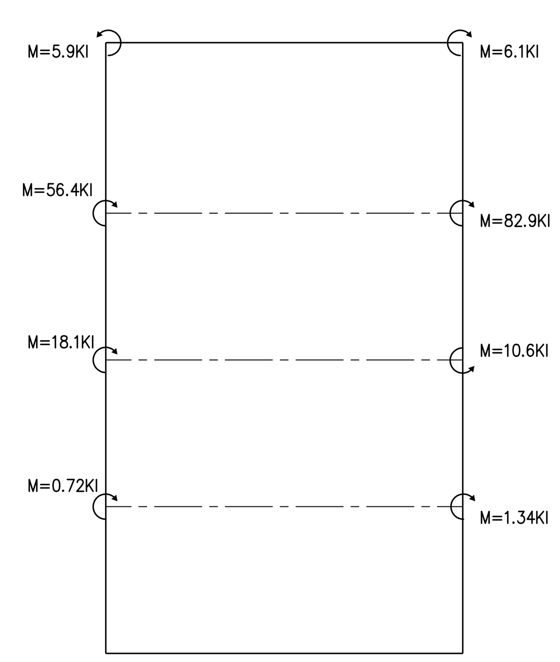
LECTURE HALL – NORTH SHEAR WALL



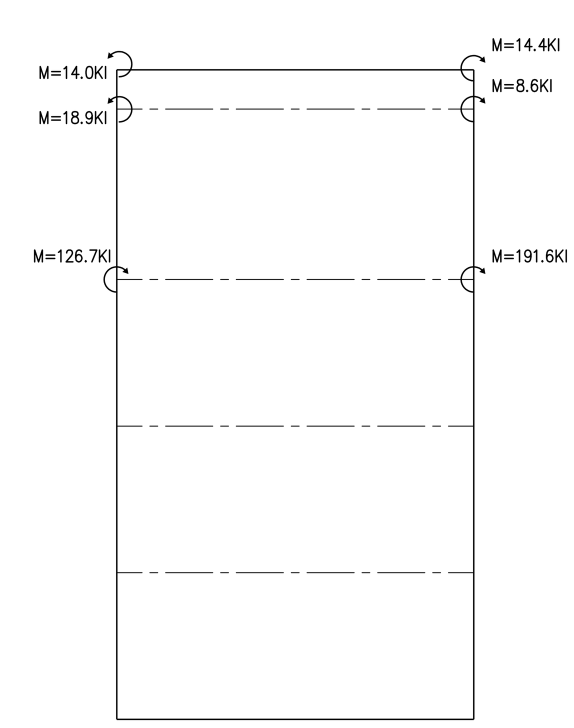
FRAME ALONG GRIDLINE 8



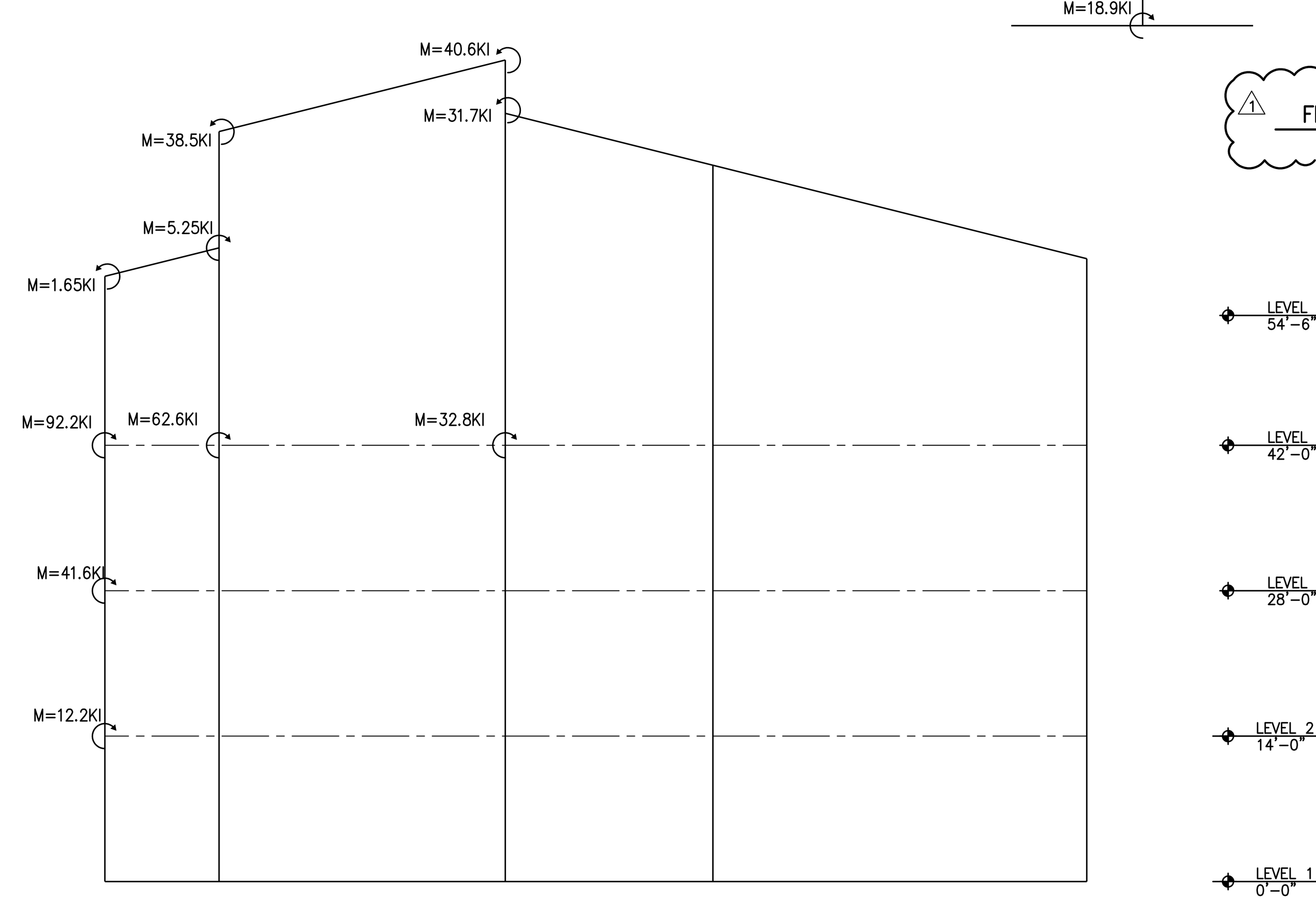
LECTURE HALL – WEST INTERIOR SHEAR WALL



FRAMES ALONG GRIDLINE A



FRAMES ALONG GRIDLINE B



FRAME AT GRIDLINES 4 & 5

FOR ADDITIONAL INFORMATION SHOWN BUT NOT NOTED, SEE GENERAL STRUCTURAL NOTES AND INTERPRETATION OF DRAWINGS ON SHEET S1.1. AND TYPICAL DETAIL SHEETS.

PROJECT NUMBER	03-955	PROJECT MANAGER	SJH
PROJECT ENGINEER	CAP	PROJECT DRAFTER	SSR

Carter::Burgess
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Phoenix, Arizona 85008
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602-912-9390(Fax)

Registered Professional Engineer, 24471 SANDRA J. HERD, 9/27/04, Arizona, U.S.A.

NOTICE OF EXTENDED CERTIFICATION AND APPROVAL PERIOD PROVISION:
This contract allows the owner to certify and approve billing and estimates within 20 days after the billing and estimates are received from the contractor.

NOTICE OF EXTENDED PAYMENT PROVISION:
This contract allows the owner to make payment within 15 days after certification and approval of billing and estimates.

NORTHERN ARIZONA UNIVERSITY
College of Business Administration
McConnell Circle
Flagstaff, Arizona

NORTHERN ARIZONA UNIVERSITY

ISSUED: 8/9/04

FOR: CONSTRUCTION DOCUMENTS

REVISIONS

1 9/27/04 ADDENDUM NO. 1

JOB NUMBER 191579
DRAWN SSR
CHECKED CAP
TITLE BRACED FRAME AND SHEAR WALL LOADS

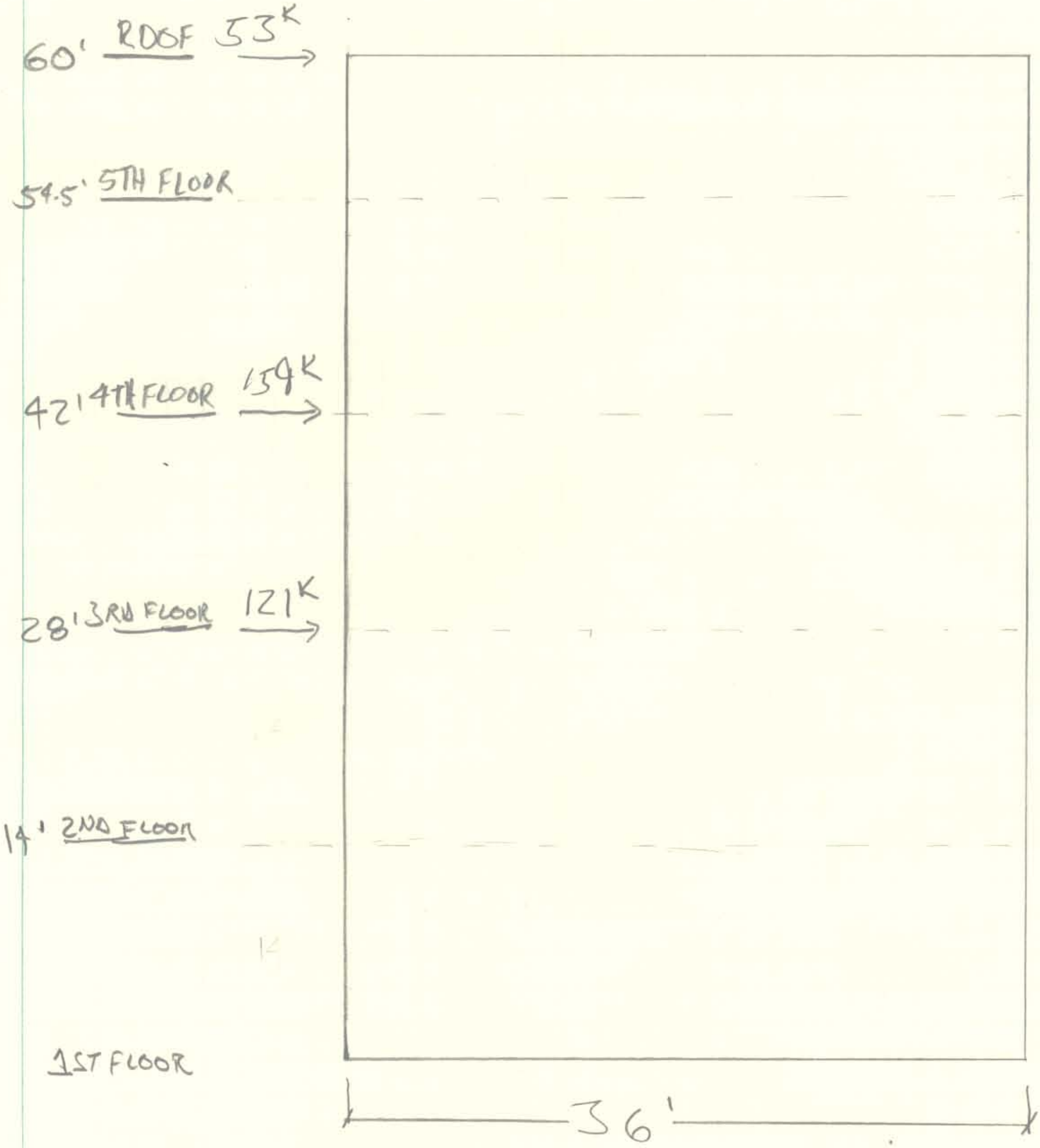
S7.1

Overturning of Entire Building

	Height	Fx	Mx	
Roof	60	92	5520	
5th Floor	54.5	178	9701	
4th Floor	42	424	17808	
3rd Floor	28	589	16492	
2nd Floor	14	672	9408	
		Σ	58929	Overturning Moment
Weight of Building (kips)		9944		
Resisting Moment		487256		

Resisting Moment > Overturning Moment

SOUTH SHEAR WALL



SHEAR WALL CHECK

CHECK SHEAR IN 2ND FLOOR

TOTAL SHEAR = 53 + 159 + 121 = 333 KIPS

$V_u = 1.6(333) = 533$ KIPS

$\phi V_c = (.75)2\sqrt{f'_c}\, hd$ where $d = .8 l_w$ ACI 318 11.10

$\phi V_c = (.75)2\sqrt{5000}\,(8)(.8)(36)(12)/1000$

$\phi V_c = 293$

$\phi V_c < V_u$

∴ PROVIDE SHEAR REINFORCEMENT

$\phi V_n = (.75)10\sqrt{5000}\, hd = 10\sqrt{5000}\,(8)(.8)(36)(12)/1000\,(.75)$

$\phi V_n = 1466^K > V_u = 533^K$

∴ WALL SECTION IS ADEQUATE

$\phi V_s = V_u - \phi V_c = 533 - 293 = 240^K$

$V_s = \frac{\phi V_s}{\phi} = \frac{240}{.75} = 320$

HORIZONTAL REINFORCEMENT

$s = \frac{A_v f_y d}{V_s} = \frac{(.2)(60)(.8)(36)(12)}{320} = 12.96''$

USE MINIMUM SPACING ⇒ #4 @ 10" O.C.

VERTICAL REINFORCEMENT

$\frac{h_w}{l_w} = \frac{60}{36} = 1.667$

$\rho_h = \frac{A_{vh}}{s h} = \frac{.2}{(10)(8)} = 0.0025$

$\rho_v = 0.0025 + .5(2.5 - \frac{h_w}{l_w})(\rho_h - 0.0025)$

$\rho_v = 0.0025$

USE #4 @ 10" O.C.

CHECK SHEAR IN 3RD FLOOR

SAME AS 2ND FLOOR

CHECK SHEAR IN 4TH FLOOR

$V_u = 1.6(53 + 159) = 339.2$

$\phi V_c/2 = 195 < V_u = 339 < \phi V_c$

∴ USE MINIMUM REINFORCING ∴ SAME AS 2ND FLOOR

USE #4 @ 10" O.C. FOR HORIZ. + VERT.

FOR ENTIRE WALL FOR SHEAR REINF.

FLEXURE DESIGN

USE LOAD COMBINATION : .9D + 1.6S

TRIBUTARY FLOOR AREA = 0

WALL DL = $.150(8)(36)(60)(\frac{1}{12}) = 216^{K}$

1ST FLOOR $P_u = .9(216) = 194.4$

$M_u = 1.6(53 \times 60 + 159 \times 42 + 121 \times 28) = 21,193^{'K}$

2nd FLOOR $P_u = 149^{K}$

$M_u = 13,734.4$

3rd FLOOR $P_u = 104^{K}$

$M_u = 6,275.2^{K}$

4th FLOOR $P_u = 58.3^{K}$

$M_u = 1526.4^{K}$

5th FLOOR $P_u = 17.8^{K}$

$M_u = 466.4^{K}$

CHECK MOMENT BASED ON RQD VERTICAL REIN. FROM SHEAR

MOMENT STRENGTH IN 1ST STORY

$A_{ST} = (.24)(36) = 8.64 \text{ in}^2$

$$\omega = \left(\frac{A_{st}}{l_w h}\right)\frac{f_y}{f'_c} = \frac{8.64}{(36)(12)(8)}\left(\frac{60}{5}\right) = 0.03$$

$$\alpha = \left(\frac{P_u}{l_w h f'_c}\right) = \frac{194.4}{(36)(12)(8)(5)} = 0.01125$$

$$\frac{c}{l_w} = \frac{\omega + \alpha}{2\omega + .85\beta_1} = \frac{0.03 + 0.01125}{2(.03) + .85(.8)} = 0.0557$$

$$\phi M_n = \phi\left[.5 A_{st} f_y l_w \left(1 + \frac{P_u}{A_{st} f_y}\right)\left(1 - \frac{c}{l_w}\right)\right]$$

$$= .9\left[.5(8.64)(60)(36)(12)\left(1 + \frac{194.4}{8.64(60)}\right)(1 - .0557)\right]$$

$\phi M_n = 130,850$ in-kips

$\phi M_n = 10,904'^k < M_u = 21,193'^k$

INCREASE AMOUNT OF VERTICAL REINFORCEMENT

USE #5 @ 5" O.C.

$\phi M_n = 25,652'^k > M_u = 21,193'^k$

FOLLOW SAME PROCEDURE TO FIND MOMENT STRENGTH IN EACH STORY

2ND FLOOR TRY #4 @ 10

$\phi M_n = 10,248'^k < M_u = 13,734'^k$

TRY #5 @ 10

$\phi M_n = 13,392'^k > M_u$ $13,734'^k$ OK

USE #5 @ 10" O.C.

OVERTURNING CHECK

$M_{OT} = 53^{K} * 60' + 159^{K} * 42' + 121^{K} * 28'$

$M_{OT} = 13,261^{1K}$

M_R (WALL WT) $= .150\,PCF\,(8/12\,ft)(36\,ft)(60\,ft)(\frac{36}{2}\,ft)$

$M_R = 3888^{1K}$

P(COLUMN) $\Rightarrow$ TRIB AREA OF COLUMN $= (36')(18') = 648^{\#}$

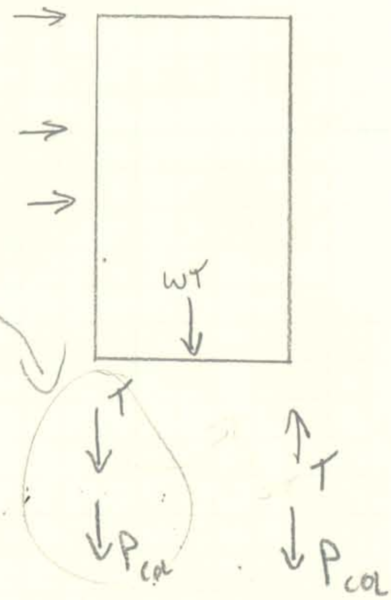
P(COLUMN) $= 648^{\#}(3(138) + 1(26)) = 282^{K}$

$\frac{M_{OT} - M_R}{L_w} = \frac{9373^{1K}}{36'} = 260^{K} = T$

$P_{column} = 282^{K} > T = 260^{K}$ NO OVERTURNING

COLUMN FORCE

$= 542^{K} = F_c$



DRIFT

$\Delta = 0.13669\,in$

$\frac{L}{\Delta} = \frac{60(12)}{.13669} = 5267$

DRIFT $= \frac{L}{5267} < \frac{L}{400}$ OK

3RD FLOOR TRY #4@10

$\phi M_n = 9592^{'k} > M_u = 6{,}275^{'k}$ OK

4TH FLOOR TRY #4@10

$\phi M_n = 8{,}921^{'k} > M_u = 1{,}526^{'k}$ OK

5TH FLOOR TRY #4@10

$\phi M_n = 8{,}322^{'k} > M_u = 466.4^{'k}$ OK

THE REQUIRED SHEAR REINFORCEMENT IS ADEQUATE FOR MOMENT STRENGTH IN FLOORS 3, 4, + 5:

REINFORCING PER FLOOR	VERTICAL	HORIZONTAL
1ST FLOOR	#4 @ 10"	#5 @ 5"
2ND FLOOR	#4 @ 10"	#5 @ 10"
3RD FLOOR	#4 @ 10"	#4 @ 10"
4TH FLOOR	#4 @ 10"	#4 @ 10"
5TH FLOOR	#4 @ 10"	#4 @ 10"