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Structural Technical Report 1
Structural Concepts / Structural Existing Conditions Report

Introduction

This report is a description of the existing conditions of the College of Business Administration building that is being built on the Northern Arizona University campus, as well as a preliminary analysis of the structural system. 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 an introduction to the structural system of the CBA building. The report includes a description of the foundation, floor framing, roof framing, and lateral force resisting systems, as well as loading criteria and design assumptions. Analysis of the structural system with spot checks of a 10" hollow core plank, an inverted t-beam, and a column are provided within this report. Also included in the Appendix of the report are, copies of the hand calculations and typical framing plans in order to provide for better understanding of the procedures followed in analysis.

Codes and Design Standards

- 2000 Edition of the International Building Code.
- ACI 318
- Latest Version of AISC Handbook
- SJI "Standard Specifications"
- CRSI Specifications and Handbook

Gravity and Lateral Loads

Gravity Loads

Roof:

- Roof Live Load = 45 PSF (Ground Snow Load).
- Roof Dead Load = 20 PSF.
- Steel Roof Joists shall be designed for a superimposed load of 300 LB. at any location.

Floors:

- Floor Live Load = 100 PSF (Reducible).
- Floor Dead Load = 138 PSF.
- Partition Load = 20 PSF (10 PSF for Seismic).
- Non-Exit Balcony Live Load = 60 PSF.
- Exit Corridor, Exit Balcony and Stair Live Load = 100 PSF.
- For Precast, Prestressed elements Superimposed Dead Load = 23.5 PSF (Does not include Concrete Topping).

Mechanical Mezzanine:

- Mezzanine Live Load = 60 PSF (Does not include Mechanical Loads).
- Floor Dead Load = 128 PSF = 128 PSF (Does not include House Keeping Pads for Mechanical Units).
- Partition Load = 20 PSF (10 PSF for Seismic).
- For Precast, Prestressed elements Superimposed Dead Load = 13.5 PSF (Does not include Concrete Topping).

Access Floor Loading:

- Access Floor Live Load at Exit Corridors = 100 PSF (Non Reducible).
- Access Floor Live Load at Offices and Classrooms = 50 PSF.
- Partition Load = 20 PSF (10 PSF for Seismic).

Lateral Loads:

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

- 3 Second Wind Gust = 90 MPH
- Exposure C, Soil Site Class C
- $S_s = 0.46$
- $S_1 = 0.13$

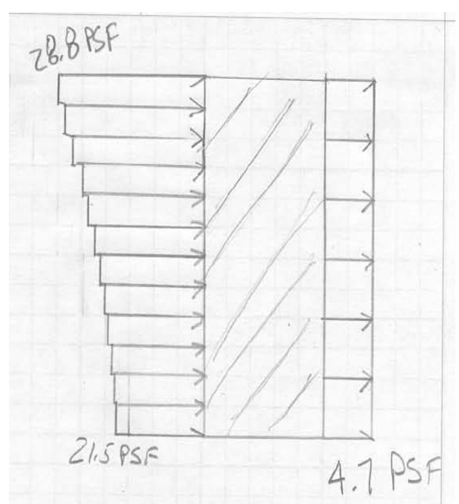
SEISMIC

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	810	58	40,922	0.132	146		8,460
5	2603	48	109,535	0.343	379	146	18,193
4	2603	36	82,959	0.260	287	525	10,343
3	2603	24	56,074	0.175	193	812	4,641
2	2603	12	28,705	0.090	99	1,006	1,193
1						1,105	
	Σ = 11222		Σ = 318195	Σ = 1.000	Σ = 1105		Σ = 42830

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

Tables 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.



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 range in size from 2'6" diameter with 6 #6 vertical reinforcing bars to 7' diameter with 16 #11 vertical reinforcing bars. The largest of the caissons are located under the columns along the middle column line of the building. All caissons are to bear at least 2'-0" into decompressed bedrock and have a minimum shaft length of 5'-0". Grade beams run along the exterior of the building except at the auditorium and along the centerline of the building. The grade beams on the north and west sides of the building are 44"X 32" beams with 8 #6 bars along the top and bottom of the beam. The beams on the south side are 24"X 28" with 6 #6 top and bottom reinforcing bars. Those along the east side of the building and along the centerline are 36"X 42" with 6 #9 top and bottom reinforcing bars. Instead of a grade beam along the exterior of the auditorium, a continuous footing with stem wall is utilized. The footings on the north and west sides are 12" deep and 2'6" wide with 3 #5 longitudinal reinforcing. Those along the east side are 16" deep and 5'6" wide with 6 #6 top and bottom longitudinal reinforcing bars and #5 at 18" on center top and bottom transverse bars. The footing between the auditorium and the main building structure is 24" deep and 8' wide with 8 #7 top and bottom longitudinal and #6 at 18" on center top and bottom reinforcing bars.

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. There are only three different sizes of precast beams used in the framing throughout the building. The most common is an inverted t-beam which is a 16"X 27" beam with 9"X 10" flanges. These beams are located along all of the interior column lines on the upper floors except where there are openings in the floors. The beams located around the openings are similar to the t-beams but are l-shaped having only one flange. The other type of beam is a 24"X 26" rectangular beam which is used sparingly at a few areas which are cantilevered. All of the columns throughout the building are 24" square precast 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. The lower roof is broken into two sections since the mezzanine is through the middle of the building. The lower roof is made up of three types of open web joists which span between W-shaped steel beams. The northern roof has 20" open web joist which span between W21x44 steel beams and cantilevers six feet in one direction (refer to Appendix for profile of all joists). The southern roof has 24" joists which span between W24x68 and W24x76 beams, as well as 28" joists which span from the W24x76 to W24x55 beams and cantilever six feet. The joists are covered with 1-1/2 " deep, 36" wide, 20 gage painted steel deck. The upper roof has W30x116 beams spanning in the N-S direction. The E-W direction has four rows of steel I beams consisting of members ranging is size from W12x45 to W24x55. This upper roof has a 3-1/2" deep, 24" wide, 18 gage 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. Both wind loads and seismic loads are important in the design of this building.

My assumption for the floor to floor height being 12 feet was incorrect. The actual floor to floor height was found to be 14 feet. This was information that I did not have until just before this report was finished. I used the simplified method of determining wind loads. Since finding out the actual height of the building I realized that since the height is more than 6o feet, I would not be allowed to used the simplified method based on ASCE 7-02. I feel the wind loads that were found are a good estimate for this juncture in the project.

Structural Analysis

As a way to check that the loads are sensible, spots checks were performed for a typical plank, an interior beam, and an interior column. Since these members are all precast concrete and the exact information on them are not given on the structural drawings, many assumptions had to be made.

Hollow Core Plank Check:

For the hollow core plank check, it was assumed a 4'x 10" Dy-Core plank was used. In order for the plank to carry the loads set forth earlier in this report, eight .6" strands had to be used. Although this layout may not be common practice, the table in the *PCI Manual for the Design of Hollow Core Slabs* shows this section to have the capability t accommodate it.

- 10" Hollow core plank.
- Span = 36 ft.
- W_u = 1.8 klf.
- Assume simple span beam.
- $M_u = w_u*l^2/8$ = 280 ft-kips.
- 8 .6" strands.
- $øM_n$ = 317 ft-kips > M_u = 280 ft-kips <u>OK for bending</u>.
- See Appendix for full calculations.

Precast Beam Check:

An interior beam like that along column line 6 was checked for flexure. This inverted t-beam was assumed to have bottom prestressing strands as well as top reinforcing bars in order to achieve the high midspan moment that was found. A valid solution using the given size of the beam was not able to be obtained due to the calculated moment of 1640 foot-kips. My limited knowledge of prestressed concrete has caused me not to find reinforcing that can hold this load.

- 34"x 27" Inverted T-beam.
- Span = 36 ft.
- M_u = 1640 ft-kips.
- Try 24 Prestressing strands and 6 #9 top bars.
- Live Load reduced from 100 PSF to 54.5 PSF.
- $øM_n$ = 1552 ft-kips.
- See Appendix for full calculations.

Column Check:

The College of Business Administration has only 24" square precast concrete columns. A check was done on a first floor column in which the beam from the beam check was connected. Since the vertical load is acting at an eccentricity to the center of the column, a moment was calculated and checked simultaneously with the load acting from all the floors above by using an Interaction Diagram from Appendix A of *Design of Concrete Structures*. The eccentricity of the dead load will cancel itself out since it will be acting from both sides of the columns. This will cause the worst moment to occur when one side of the column experiences the live load and the other side does not.

- 24" Square Column.
- P_u = 1456 kips.
- M = 55.4 ft-kips.
- P_g = 0.025 (from Interaction Diagram).
- Area of Steel Required = 14.4 sq. in.
- Assume 12 #10 Reinforcing bars used.
- See Appendix for full calculations.

The checks of the gravity loads seem to show it could be possible that some assumptions that were made are not accurate. The discrepancies found may be due in large part to the fact that reinforcing for these members is not given on the structural drawings.

Shear Wall Check:

The south shear wall between column lines 4 and 5 was chosen to be checked. An assumption of the thickness of the wall was made to be 12". A method of using tributary areas to disperse the loads to the different lateral resisting members was used. It was found that this shear wall would need hold downs that could resist a force of 64 kips.

- Tributary Width = 28 ft.
- Force at roof = 40.5 kips.
- Force at 4^{th} Floor = 105.1 kips.
- Force at 3^{rd} Floor = 79.6 kips.
- M_{OT} = 8,124 ft-kips.
- M_R = 5,832 ft-kips.
- T = 64 kips.
- See Appendix for full calculations.

Appendix

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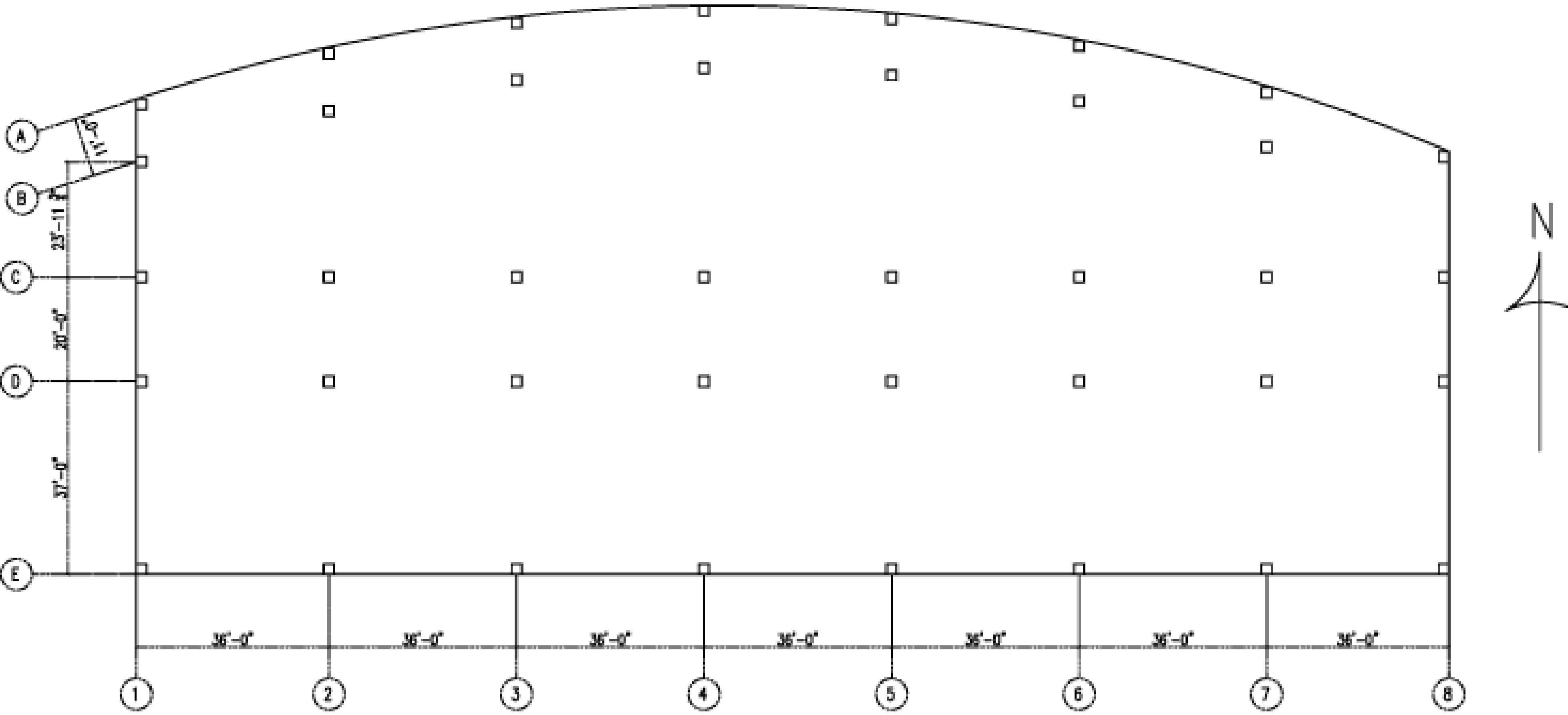
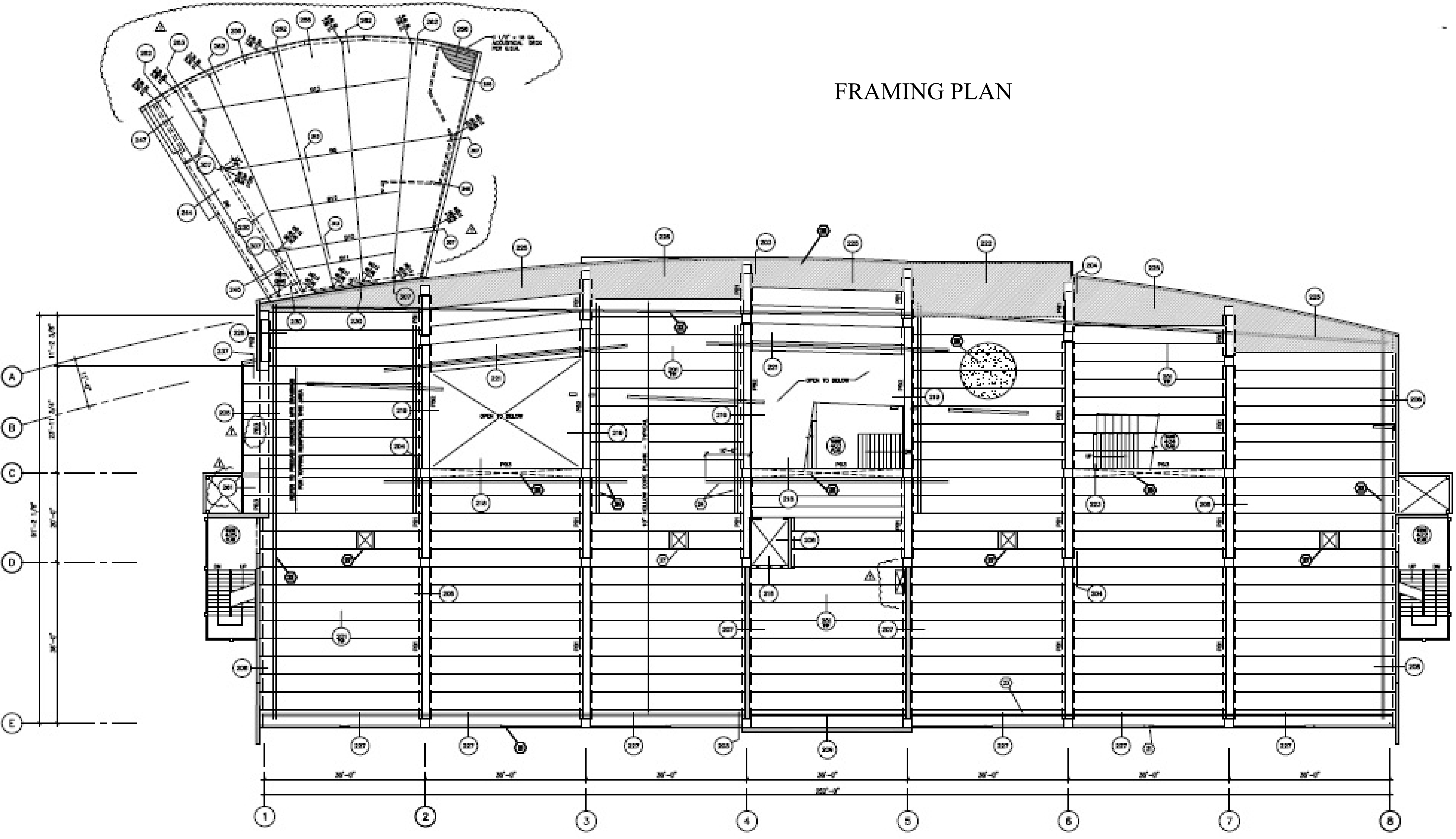
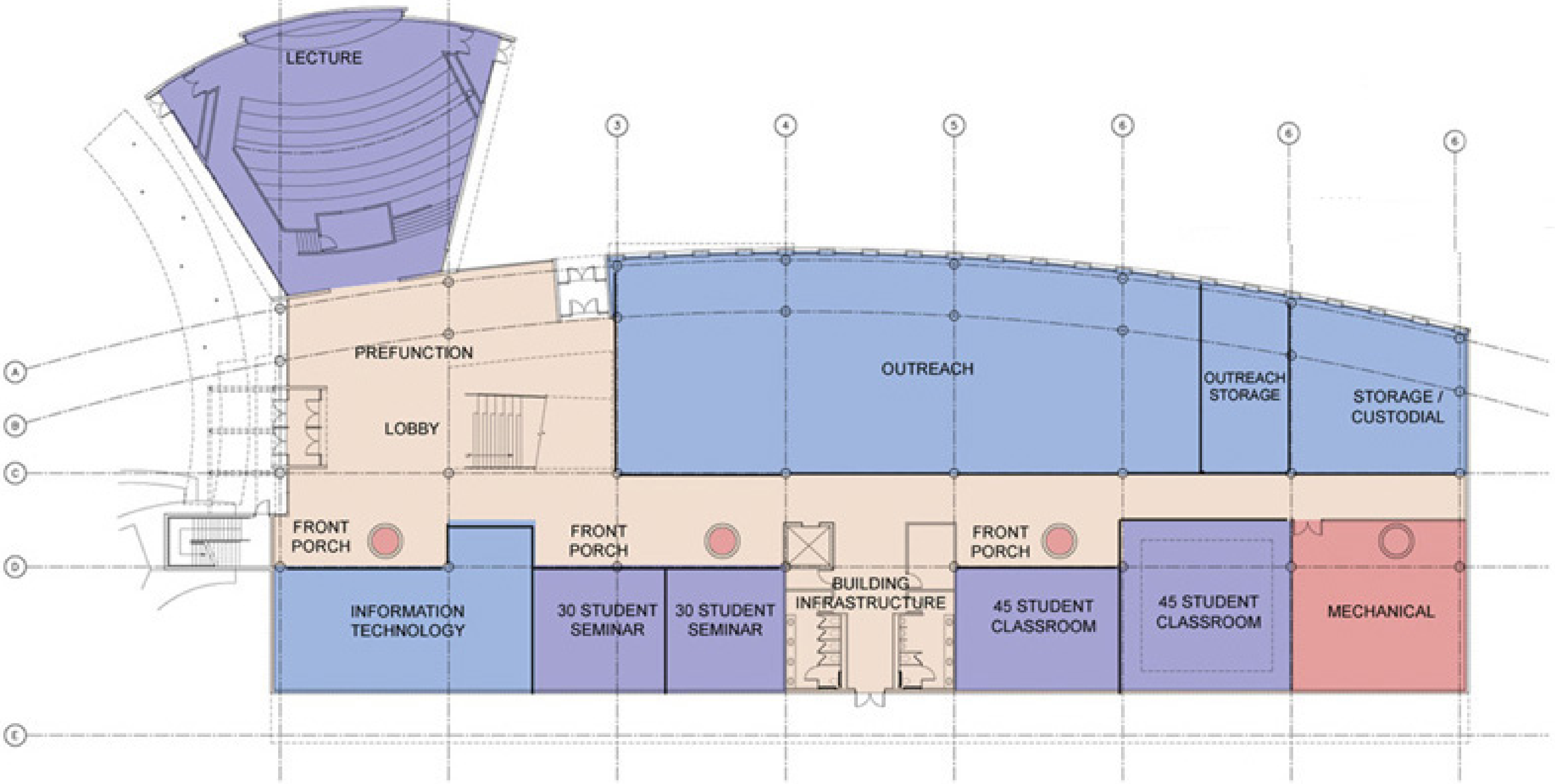


COLUMN LAYOUT OF COLLEGE OF BUSINESS ADMINISTRATION BUILDING

FRAMING PLAN




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

HOLLOW-CORE PLANK CHECK

10" HOLLOW-CORE PLANK – SPAN = 36 ft.

* ASSUME – FROM PCI MANUAL FOR DESIGN OF HOLLOW CORE SLABS – 2ND EDITION

– Dy-Core
– 4' x 10" w/2" TOPPING

wt = 81 PSF

LOADS

DL = 138 PSF
SDL = 23.5 PSF
WT = 181 PSF
LL = 100 PSF

$\omega_u = 4(1.2(138 + 23.5 + 81) + 1.6(100))$

$\omega_u = 1.8$ klf

SPAN = 36'
CleAR SPAN = 35'-3"

$$M_u = \frac{1.8(35.25)^2}{8}$$

$M_u = 280^{\text{1K}}$

TRY 8 - .6" STRANDS $A_{ps} = 1.73\text{in}^2$

$$f_{ps} = f_{pu}\left[1 - \frac{\gamma_p}{\beta_1}\left(\rho_p \frac{f_{pu}}{f_c'}\right)\right] \Rightarrow \beta_1 = 0.8 \quad d_p = 9\text{in}$$

$\gamma_p = 0.28$ – low lex STRANDS

$$\rho_p = \frac{A_{ps}}{bd_p} = \frac{1.73}{(48)(9)} = 0.004$$

$$f_{ps} = 270\left[1 - \frac{0.28}{0.8}\left(0.004\left(\frac{270}{5}\right)\right)\right]$$

$f_{ps} = 249.2$ ksi

$$\omega_p = \frac{\rho_p f_{ps}}{f_c'} = \frac{0.004(249.2)}{5} = 0.199 < .36\beta_1 = .288$$

OK

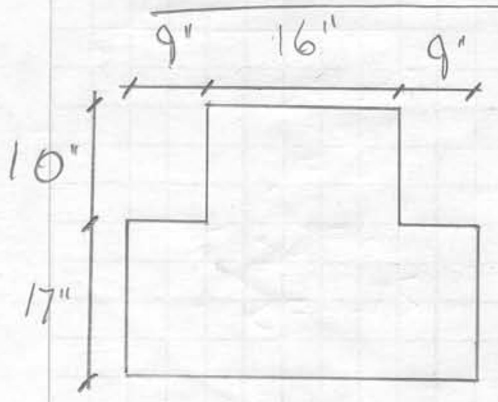
$$a = \frac{A_{ps} f_{ps}}{.85 f_c' b} = \frac{(1.73)(249.2)}{(0.85)(5)(48)} = 2.11\text{in}$$

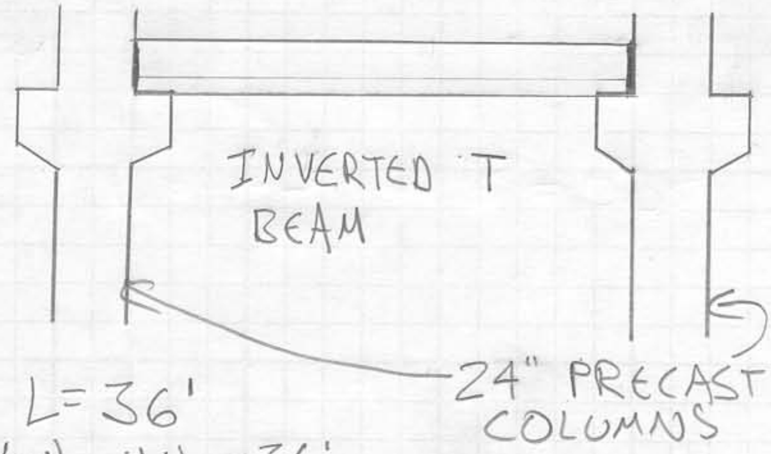
$$\phi M_n = \phi A_{ps} f_{ps}\left(d_p - \frac{a}{2}\right) = 0.9(1.73)(249.2)\left(9 - \frac{2.11}{2}\right)$$

$\phi M_n = 317^{\text{1K}} > M_u = 280^{\text{1K}}$

OK

BEAM CHECK





L = 36'
trib width = 36'

LOADS

DL = 138 PSF
SDL = 23.5 PSF

LL = 100 PSF → REDUCE

LL REDUCTION

$L_0 = L_0\left(.25 + \frac{15}{\sqrt{K_{LL}A_T}}\right)$

K_{LL} BEAM = 2

$A_T = (36\,ft)(36\,ft) = 1296\,ft^2$

$K_{LL} = 2592\,ft^2 > 400\,ft^2$
CAN BE REDUCED

$L = 100\left(.25 + \frac{15}{\sqrt{2592}}\right)$

$LL = 54.5\,PSF \doteq .54 > .5$ OK

* USE 1.2DL + 1.6LL

$\omega_u = 281\,PSF(36\,ft)$

$\omega_u = 10.12\,klf$

* ASSUME SIMPLE SPAN BEAM ∴

$M = \frac{\omega l^2}{8} = \frac{10.12\,klf(36)^2}{8}$

$M_u = 1640'^k$

ASSUMPTIONS:

1) ~~ASD~~ → 1.2 DL + 1.6 LL
2) SIMPLE SPAN BEAM
3) PRESTRESSED - BONDED TENDONS
4) ASSUME $f'_c = 5000\,psi$, $f_{pu} = 270\,ksi$, $f_{pe} = .55\,f_{pu} = 148.5\,psi$
5) 24 PRESTRESSING STRANDS ½" φ + 6-#9 TOP REINF.

24 STRAND $\rightarrow$ $A_{ps} = 3.672 \text{ in}^2$ $\qquad d_p = 27 - 3 = 24 \text{ in}$

$6 - \#11 \rightarrow A_s' = 9.36 \text{ in}^2$ $\qquad d_s = 3.5 \text{ in}$

$f_{ps} = f_{pu}\left[1 - \frac{\gamma_p}{\beta_1}\left(\rho_p \frac{f_{pu}}{f'_c}\right) + \frac{d_s}{d_p}(\omega ...)\right]$ $\qquad \gamma_p = 0.28$ – low lax tendons $\quad \beta_1 = 0.8$ SINCE $f'_c = 5 \text{ ksi}$

$$\rho_p = \frac{A_{ps}}{bd^2} = \frac{3.672}{(16)(24)^2} = 0.000398$$

$$f_{ps} = 270\left[1 - \frac{0.28}{0.8}\left(0.000398 \frac{270}{5}\right)\right] = 268 \text{ psi}$$

$$c = \frac{A_{ps} f_{ps} - A_s' f_y'}{.85 f'_c \beta_1 b} = \frac{(3.672)(268) - (9.36)(60)}{.85(5)(.8)(16)} = 7.77 \text{ in}$$

$$c/d = \frac{7.77}{24} = .323 < .375 \therefore \phi = 0.9$$

$a = 7.77(.8)$

$a = 6.218$

$$\phi M_n = \phi\left[A_{ps} f_{ps}\left(d_p - \frac{a}{2}\right) - A_s' f_y'\left(d_s' - \frac{a}{2}\right)\right]$$

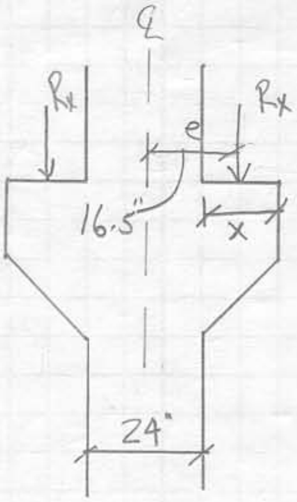
$$\phi M_n = 0.9\left[(3.672)(268)\left(24 - \frac{6.218}{2}\right) - 9.36(60)\left(3.5 - \frac{6.218}{2}\right)\right]$$

$$\phi M_n = 1552'^{k} < M_u$$

COLUMN CHECK

REACTION FROM BEAM $\Rightarrow \frac{w l}{2} = \frac{10.12\ ^{k}/_{ft}\,(36)\,ft}{2} =$

$R_x = 182$ kips



ASSUME $x = 9$ in

SO R_x ACTS @ 16.5 in FROM CENTER

LIVE LOAD REDUCTION

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

$= 100\left(.25 + \frac{15}{\sqrt{4 \cdot 28 \cdot 36(4)}}\right)$

$L = 100(.368)$ USE .4 - MAX REDUCTION

$LL = 40$ PSF

* DEAD LOAD CANCELS OUT FROM BEING APPLIED ON BOTH SIDES. WORST CASE IS WHEN LIVE LOAD IS APPLIED TO ONLY 1 SIDE.

$P = 40\text{PSF} \cdot 28\text{FT} \cdot 36\text{FT}$

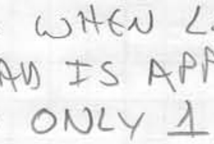
$P = 40.3$ k

$M = Pe = 40.3\text{k}\left(\frac{16.5\text{in}}{12\ ^{in}/_{FT}}\right) =$

* $M = 55.44^{1K}$

$P_U = R_x(2)(4)$

REACTION ON EACH SIDE — FOUR FLOORS



$P_U = 1456$ kips

FROM INTERACTION DIAGRAM $\rho_g = 0.025$

SO. STEEL REQUIRED $= 0.025(24)^2 = 14.4\ in$

USE 12 # 10 BARS

LATERAL SPOT CHECK

* SEISMIC CONTROLS DESIGN

USE TRIBUTARY WIDTH BETWEEN LATERAL RESISTING MEMBERS TO FIND FORCE IN SHEAR WALL.

TRIB = 28 FT

USE SHEARWALL ALONG COLUMN LINE E

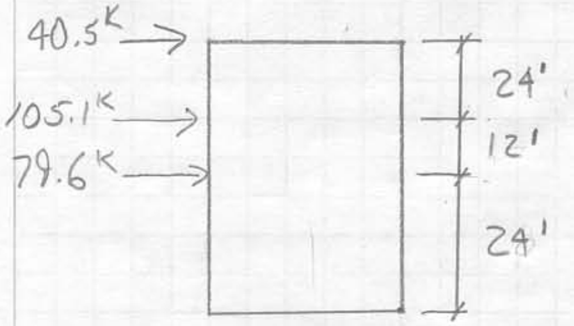
- LENGTH = 36 FT

ROOF FORCE = 40.5 kips

4^{TH} FLOOR FORCE = 105.1 kips

3^{RD} FLOOR FORCE = 79.6 kips

FLOOR DOES NOT ATTATCH AT 1^{ST} OR 2^{ND} FLOOR



SOUTH SHEAR WALL

$$M = 40.5^{k}(60') + 105.1^{k}(36') + 79.6^{k}(24')$$

$$M_{OT} = 8{,}124^{'k}$$

WT OF WALL:

ASSUME 12" THICK SHEAR WALL

$$WT = 150\,PCF\,(1\,FT)(60\,FT)(36\,FT)$$

$$WT = 324^{KIPS}$$

$$M_{RESISTING} = 324^{k}\left(\frac{36\,FT}{2}\right) = 5{,}832^{'k}$$

FORCE AT ENDS OF SHEARWALL

$$T = \frac{8124 - 5832}{36} =$$

$$T = 64^{KIPS}$$