TECHNICAL REPORT I

Aubert Ndjolba | Structural Option

PENN COLLEGE OF TECHNOLOGY

Existing Conditions

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Date: 09/23/11

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EXECUTIVE SUMMARY

The purpose of the Technical Report I is to analyze the existing structure of the Dauphin Hall and gain a throughout understanding of its current conditions. This analysis is being done through illustrations and summaries of the foundation, floor system, framing system, lateral system, and roof system. Moreover, it includes design codes and materials used, and a check of gravity and lateral loads. These loads were then compared to the actual loads on the structural drawings.

ASCE 7-10 was used for the calculations of wind load, Snow load, and seismic load. In this report, these loads were only partials calculated to show an understanding of how the lateral system is being resisted by the current structure. A more developed and complete analysis will be provided in the revision of Tech. I. The snow load was found to be equal to 24.3 lb/ft^2 similar to the one provided in the structural drawings. A base seismic shear force of 3427 k was found to be acting on the building.

Using the latest codes, the column X-33 spot check found that it was overdesigned. Moreover, the slab, the deck, and the composite beam were all found to be adequate for flexural and deflection criteria.

Furthermore, included in this technical report are appendixes, which contain hand calculations and partial drawings necessary for the understanding of the building structure.

BUILDING INTRODUCTION

The Pennsylvania College of Technology is located in the 200 block of Rose Street in Williamsport, PA. It is the newest dormitory on campus constructed in August 2010 by Murray Associates Architects, P.C in collaboration with IMC as the general contractor; Woodburn & Associates, INC as the food service designer; Whitney, Bailey, Cox & Magnani, LLC as the civil engineering firm; and Gatter & Diehl, INC as the MEP firm. This new structure costs approximately $ 26,000,000 and used the old design-bid-build project delivery method.

Figure1: Map

This latest addition of the student housing provides 268 students with suites and single rooms. A 40-50 student seating commons enclosed with glass provides a perfect social space for student collaboration. Located within the dormitory are other amenities such as: a 460 seat dining room, two private dining rooms for faculties, a 40 station satellite fitness center, two large leisure rooms, a student grocery store, laundry facilities, student mail boxes, Resident Life Offices, campus police office, and a Hall Coordinator apartment.

Figure 2: South facade

On the left side are different facades provided for an understanding of the shape of the building. A set of floor plans are provided in appendix D.

Figure 3: South facade

STRUCTURAL OVERVIEW

The Dauphin Hall rests entirely on shallow foundation and stone piers. The exterior and interior walls are composed of masonry walls. The whole structure is made out of steel framing (joists, beams, and columns), which supports a 4" concrete slab reinforced with welded wire mesh on a composite deck.

FOUNDATIONS

Base on the analysis done by CMT Laboratories, Inc. for this site, they have determined that the site was filled with Brown Silty Clay, and Brown Silty Sand with Gravel. They have also determined that the cohesive alluvial soils beneath the fill materials have low shear strength and the ground water tale has shallow depth.

In light of these conditions, the conventional spread/column and continuous footing foundations will not provide adequate allowable bearing capacity to support the building. Deep foundations such as concrete filled tapered piles could support the structure but are not the most economical approach. Therefore, a practical solution is subsurface improvement with the use of shallow foundation.

All in all, the final decision comes down to using stone piers which were considered the most technically sound and economically feasible method. Those stone piers are typically eighteen (18) to thirty-six (36) inches in diameter depending on their loading and settlement criteria.

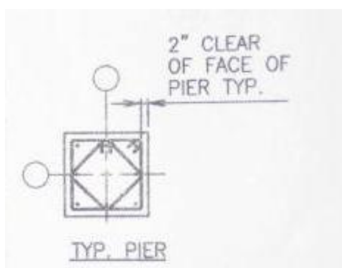


Figure 4: Typical Pier

FLOOR SYSTEMS

Due to the simplicity of the foot prints of the Dauphin hall, a typical floor consists of 4" concrete slab reinforced with 6"X6" –W2.9XW2.9 welded wire mesh. The concrete slab rests on 1 ½" - 20 gage composite deck (Vulcraft). The joists supporting the floor system are spaced equally in column bays with a maximum spacing of 2'-0" O.C in areas of floor framing.

A typical bay for the three floors above is 25' X 30'.

The figure below provides a typical bay size.

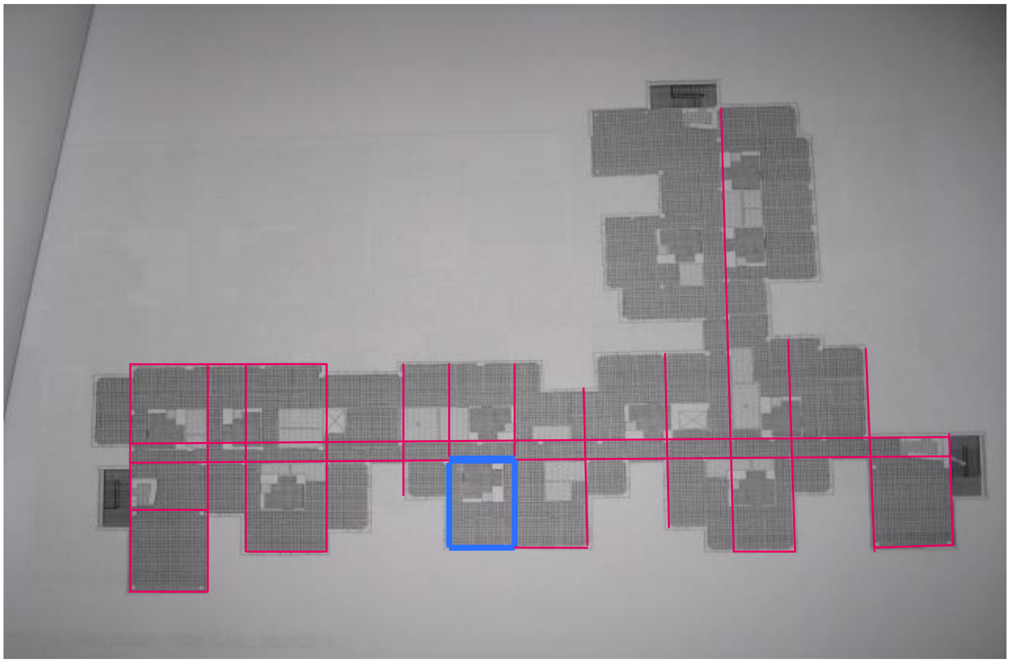


Figure 5: Typical Floor Bay Size (Blue Square)

FRAMING SYSTEM

Almost all the structural columns supporting the floors are either a wide flange W10 or W8. They are all encased by 5/8" Gypsum board or 6" painted CMU. In locations near the stair cases, HSS columns were used. Concrete Masonry Units (CMU) is the typical interior partitions.

LATERAL SYSTEM

To resist the lateral system in the dauphin Hall, the structural engineers used wind moment frames with moment connections throughout the building. This configuration provides no obstruction and therefore allows a great use of the open floor plan.

ROOF SYSTEMS

There is only one roof system on the Dauphin Hall dormitory due to the similarity of the outline of the building. The whole roof is composed of 1 1/2" – 20 gage type B roof decks, which rests on light gage trusses at 2'-0" O.C. The joists supporting the roof system are spaced at a maximum distance of 4'-0" O.C. between the column bays.

DESIGN CODES

All equipments and components of the Dauphin Hall shall comply with all applicable latest editions of articles and sections of the following codes in compliances with all Federal, State, County, and Local ordinances and regulations:

- 2006 International Building Code (IBC)
- National Electrical Code (NEC),
- Uniform Plumbing Code (UPC),
- National Sanitation Foundation (NSF)
- Specifications for structural concrete for buildings (ACI 301)
- Building Code Requirements for Reinforced Concrete (ACI 318-08)
- Recommended Practice for Hot Weather Concreting (ACI 305R)
- Recommended Practice for Cold Weather Concreting (ACI 306R)
- Recommended Practice for Concrete Formwork (ACI 347)
- American Society of Civil Engineers (ASCE 7- 10)

MATERIALS USED

The following table provides a list of materials used in the design of this building. Those values were found in the structural drawing and the specifications.

Concrete		
Usage	**Weight**	**Strength (psi)**
Footings	Normal	4000
Foundation alls	Normal	4000
Slab-on-Grade	Normal	4000
Suspended Slabs	Normal	4000
Toppings	Normal	5000
Piers	Normal	4000

Table 1: Concrete materials

Steel		
Type	**Standard**	**Grade**
W-Shaped Structural Steel	ASTM A 572/A 572M	50
Channels, Angles-Shapes	ASTM A 36/A 36M	36
Plate and Bar	ASTM A 36/A 36M	36
Cold-Formed Hollow	ASTM A 500	B
Steel Pipe	ASTM A 53/A 53M	B
Bolts, Nuts, and Washers	ASTM A325/ASTM F 1852	N/A
Steel Deck	ASTM A 653	A
Reinforcing Bars	ASTM A 615/A 615M	60
Deformed Bars	ASTM 767	A
Welded Wire Fabric	ASTM A 615	65

Table 2: Steel materials

Masonry		
Type	**Standard**	**Strength (psi)**
Concrete Block	ASTM C 90/ ASTM C 145	1900
Split Face CMU	ASTM C 90lightweight	1900
Bond Beam	N/A	3000
Precast Stone	N/A	5000-7000
Concrete Brick	ASTM C 1634/ASTM C 55	N/A
Mortar	ASTM C 979	N/A
Grout	ASTM C 404	N/A

Table 3: Masonry materials

Miscellaneous	
Type	**Strength (psi)**
Concrete Fill	3000
Non-Shrink Nonmetallic Grout	ASTM C 1107

Table 4: Miscellaneous materials

GRAVITY LOADS

Included in this report is a calculation of dead, live, and snow loads. There were compared to the actual calculations in the structural drawings. Several members were checked to verify adequacy.

DEAD AND LIVE LOADS

Superimposed Dead Loads	
Description	**Loads**
Roof	
Roofing	3 PSF
Framing	5 PSF
Insulation	3 PSF
Ceiling	2 PSF
Elec./Lights	3 PSF
Mechanical	5 PSF
Sprinklers	3 PSF
Miscellaneous	1 PSF
Total	**25 PSF**
Floor	
4” Slab and Deck	44 PSF
Framing	5 PSF
Mechanical	5 PSF
Elec./Lights	3 PSF
Ceiling	2 PSF
Sprinklers	3 PSF
Miscellaneous	3 PSF
Total	**65 PSF**
Snow	**35 PSF**

Table 5: Design Dead Loads

Description	Quantity (ft2)
Ground floor	14,473
2nd Floor	10,320
3rd Floor	10,320
4th Floor	10,320
Roof	10,320

Table 6: Area of Typical Floor

Design Live Loads		
Description	Design Loads	Thesis Loads
Roof	35 PSF	35 PSF
First Floor	100 PSF	100 PSF
Stairs	100 PSF	100 PSF
Dorm Rooms	40 PSF	40 PSF
Corridors	100 PSF	100 PSF
Storage	125 PSF	125 PSF
Mechanical room	150 PSF	125 PSF
Common Areas	100 PSF	100 PSF

Table 7: Design Live Load

SNOW LOAD

The flat roof snow load was calculated to be 24.3 psf using ASCE 7-10 design criteria. It is found to be the same as the one on the structural drawing. In addition, there has been found to be a snow drift on the fourth floor due to the roof structure configuration. The snow drift calculations are in appendix A. The figure below shows a facade of the building.

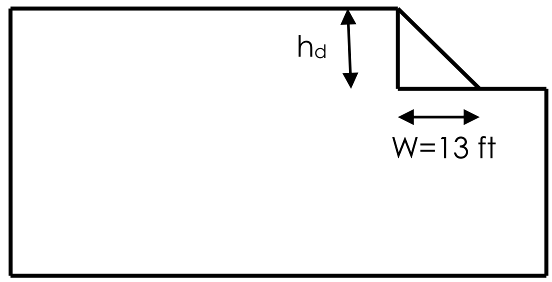


Figure 6: Snow drift

COLUMN X-33 GRAVITY CHECK

A gravity spot check was performed on an interior column X-33 around the commons area on the second floor. However, due to the storage room near the column, a storage load was assumed to be a better load value. A W10X77 was found to be an adequate column to support the loads above. The design column (W10X88) has about 13% more capacity than the thesis column. This discrepancy may be due to the fact that the thesis column reduced the unbalance moment applied to the column. An extended calculation on the spot check can be found in appendix A. The figure below shows the column location relative to the overall position of the building.

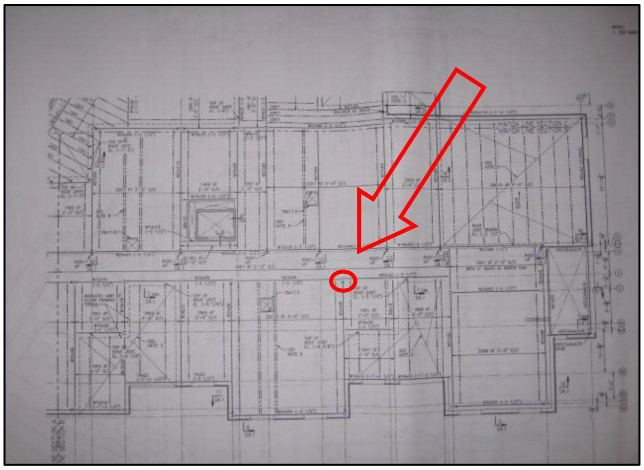


Figure 7: Column Checked

COMPOSITE DECK SPOT CHECK

Based on the composite deck catalog (Vulcraft), a 1.5VLR20 is more than adequate for a 4" concrete slab with 6x6 – W2.1XW2.1. Both the unshored length (construction span) and loading factor (clear span) conditions were met. The slab was overdesign. Appendix A provides a set of calculations for the spot check.

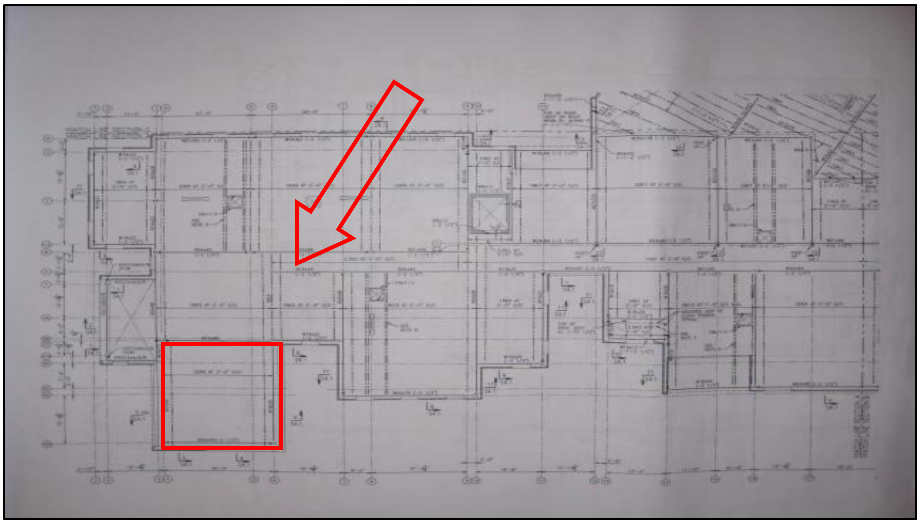


Figure 8: Typical Deck Location

JOIST SPOT CHECK

Using Vulcraft manual, the K-series joists were designed with a live load of 100 psf and a dead load of 65 psf. It was determined that for these loads, an 18K6 that weights 8.6 lbs. is an economical design. However, a 22K6 used in the actual building is more than satisfactory for live and total load conditions. Refer to figure 8 for the location on the deck used for this calculation. An analysis of the design is located in appendix A.

COMPOSITE BEAM SPOT CHECK

The design beam, a W24X68, in the column line (cc), between (4) and (6), has an ultimate moment and strength well under the flexural moment and strength (44 ft-k << 904 ft-k; 6.97k << 295k). The live load and total load deflection checks were satisfactory. This discrepancy may be due to the fact that the design was deflection control (See Appendix A and figure 8).

LATERAL LOADS

In this report, two main lateral loads were partially analyses just to provide a better understanding of how the lateral resisting system (moment frame) works. A complete and elaborate analysis will be provided in the revision of this report at a later time or in Tech. II or III.

WIND LOADS

To purely provide an example of the transfer of lateral loads to the ground, an analysis of the N-S wind pressures applied to the building has found the pressures at the roof level to be moderate.

Using applicable sections of ASCE 7-10, the following design values were determined.

General Wind Design Criteria		
Design wind Speed (V)	90 MPH	ASCE 7-10
Directionality Factor (k_d)	0.85	ASCE 7-10
Important factor (I_w)	1.15	ASCE 7-10
Exposure Category	C	ASCE 7-10
Topographic Factor (K_{zt})	1	ASCE 7-10
Internal Pressure (Gcpi)	0.18	ASCE 7-10

Table 8: Wind design values

Figure 9: Projected area for wind calculations

External Pressure coeff. (Cp)		
Description	N-S Wind	E-W Wind
L/B	1.84	0.54
Winward Wall	0.8	0.8
Leeward Wall	-0.3	-0.5
Side Walls	-0.7	-0.7
h/L	0.179	0.33
Roof < 0.25	-0.2	-0.3
Roof < 0.25	0.3	0.2

Table 9: External pressure coefficient

Velocity Pressure Coeff. And Velocity Pressure			
Level	Elevation	Kz	Qz
Gound	0	0.85	17.229024
2nd	16	0.9	18.242496
3rd	29.3	0.98	19.8640512
4th	42.6	1.04	21.0802176
Attic Space	56.6	1.1	22.296384
Roof	70.6	1.17	23.7152448

Table 10: Velocity pressure coefficient and velocity pressure

SEISMIC LOADS

A partial seismic ground motion was calculated per ASCE 7-10. It was found to be 3427 k. The distribution of the ground force in different floor levels will be provided in the revision of Tech. I. The table below provides all the design values calculated.

Seismic values	
S_s	0.18g
S_1	0.06g
S_{ms}	0.288
S_{m1}	0.144
S_{DS}	0.192
S_{D1}	0.096
I_e	1.25
R	3.0
C_T	0.028
x	0.8
T	0.84 sec
C_s	0.8
k	2
w	4284 k
v	3427 k

Table 11: Seismic Values

CONCLUSION

In conclusion, the analysis and examinations done in this report provided an overall understanding of the building system as a whole. It has demonstrated that the building was design to code and all the different criteria were met. However, we can state that some of the members were overdesigned. It may be due to a conservative approach, or that older versions of the codes were used, or the loads were unfactored, or simply as a designer's choice to apply a safety factor.

Calculations of the wind and seismic loads were only done partially as a tool to understand the transfer of the loads to the ground. A full analysis of these systems will be provided in upcoming reports.

APPENDIXES

APPENDIX A: GRAVITY CHECKS

Aubert Ndjolba	AE Senior thesis	Colum Spot check	1

Estimated Column size

Axial loads on interior columns:

SDL =

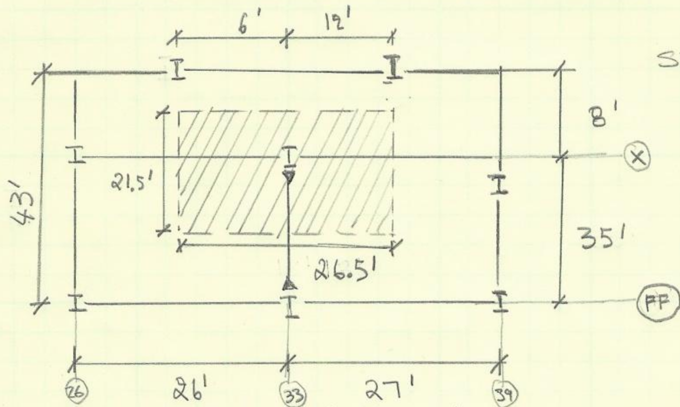


Figure 11 : Typical floor plan

- Tributary area = (26.5 ft)(21.5 ft) = 570 sq ft

#1 Load below level 3: Roof + 2 floors

$LL_{red} = 0.25 + \frac{15}{\sqrt{4(2)(570)}} = 0.47 \Rightarrow$ use 0.5

$P_L = 0.5\,(125)(570)(2) = 71.3\,K$

$P_S = 35(570) = 20\,K$

$P_D = 65(2)(570) + 25(570) = 88.3$

$P_u = 1.2(88.3) + 1.6(71.3) + 0.5(20) = \boxed{230^k}$

#2 Load below level 4: Roof + 1 floor

$LL_{red} = 0.25 + \frac{15}{\sqrt{4(570)}} = 0.56$

$P_L = 0.56(125)(570) = 39.9^k \simeq 40^k$

$P_S = 35(570) = 19.95\,K = 20^k$

$P_D = 25(570) + 65(570) = 51.3\,K$

$P_u = 1.2(51.3) + 1.6(40) + 0.5(20) = \boxed{136K}$

#3 Load below ground level: Roof + 3 floors.

$$LL_{red} = 0.25 + \frac{15}{\sqrt{4(3)(570)}} = 0.43$$

use $0.5 = LL_{red}$

$P_L = 0.5(125)(570)(3) = 107\ K$

$P_S = 35(570) = 20\ K$

$P_D = 25(570) + 65(570)(3) = 125\ K$

$P_u = 1.2(125) + 1.6(107) + 0.5(20) = \boxed{331\ K}$

Note: Load combination #2 is controlling.

$P_u = 136\ K$

ESTIMATE UNBALANCED MOMENT FOR INTERIOR COLUMN

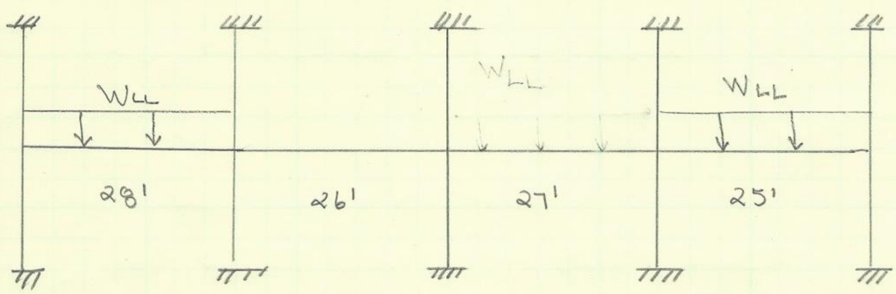


Figure 12: Largest Unbalanced moment for interior columns.

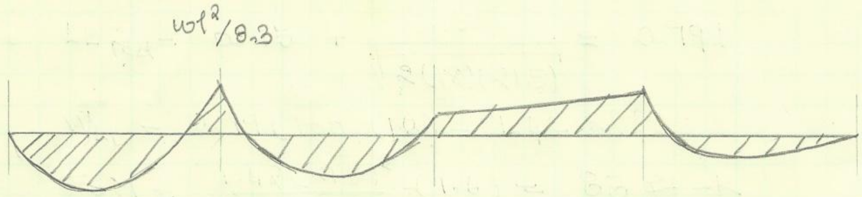


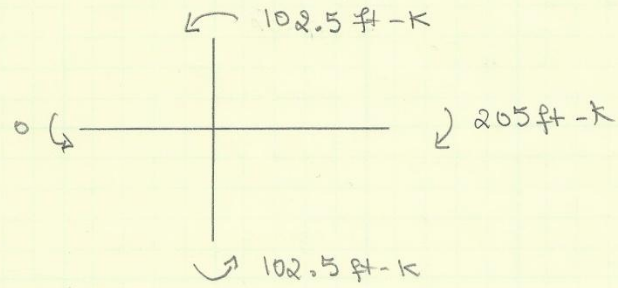
Figure 13: Moment diagram.

Aubert Ndjolba | AE Senior Thesis | Column Spot check | 3

$$LL_{red} = 0.25 + \frac{15}{\sqrt{2(21.5)(26.5)}} = 0.69$$

$$W_{LL} = 0.69\,(125)(26.5) = 2.30\ klf$$

$$FEM = \frac{2.3\,(21.5)^2}{8.3}\,(1.6) = 205\ ft\text{-}k$$



Estimate Column size

$P_{eq} = P_u + 24\,M_u/d$ Assume $d = 10$ (W10X)

4: $P_u = 136^k$, $M_u = 103\ ft\text{-}k$ → $P_{eq} = 136 + \frac{24(103)}{10} = 383^k$

Try W10X54

$\phi P_n = 520^k$ @ 14' > 383^k ✓

3: $P_u = 230^k$, $M_u = 103\ ft\text{-}k$ → $P_{eq} = 230 + \frac{24(103)}{10} = 477^k$

Try W10X54

$\phi P_n = 543k$ @ 13' > 477^k ✓

2: $P_u = 331\,k$, $M_u = 103\ ft\text{-}k$ → $P_{eq} = 331 + \frac{24(103)}{10} = 578^k$

Try W10X77

$\phi P_n = 684K$ @ 16' > 578K ✓

Designed Column W10X88 $\phi P_n = 789K$ (≈ 13.3% greater)

Aubert Ndjolba | AE Senior thesis | Deck spot check | 1

DECKING SPOT CHECK

Fourth (4^{th}) Floor typical bay (Commons Area):

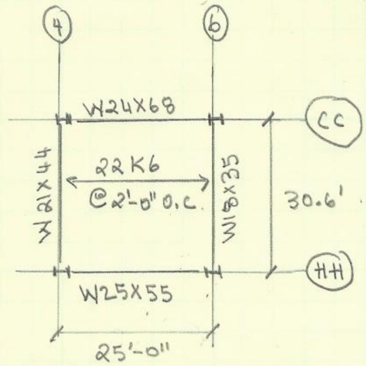


Figure 14: Typical Bay

Composite deck

4" Concrete Slab

1½" – 20 gauge steel deck

Light wt Concrete

Total t = 5.5

f'_c = 4000 psi

3 span Condition.

Loads:

LL = 100 psf

DL = 65 psf (include slab/Deck)

Total = 165 + 30 = 195 psf

(30 = partitions)

Vulcraft Decking Catalog

1.5VLR20: Check unshored length

8'-8 > 2' (joist span) ok for unshored length.

Check superimposed LL

5'-0" clear span

400 psf >> 195 psf ∴ ok for Loading

Reinforcing: 6X6-W2.1XW2.1 welded wire fabric

This slab is overdesigned based on the loading factor above

$\boxed{\text{Deck 1.5VLR20 is good}}$ ✓

Aubert Ndjolba | AE Senior Thesis | Joist spot check | 2

Floor joists: 22K6

Refer to figure above:

Loads:

LL = 100 psf   Partition walls = 30 psf
DL = 65 psf

Assumption: joist spacing 2'-0", Span = 25'

LL = 2(100) = 200 plf

DL = 2(95) = 190 plf + self wt

From Vulcraft steel joists catalog, the most economical joist is:

18K6 (wt = 8.6 plf) with 2 Rows bridging

TL = 435 plf > 390 plf
@ 25'
LL = 305 plf > 200 plf

Comparaison

For 22K6 (wt = 9.2 lbs/ft)   TL = 537 plf > 390 plf @ 25'
LL = 464 plf >> 200 plf @ 25'

Based on this analysis, the joists were overdesigned for LL.

Aubert Ndjolba | AE Senior thesis | Beam spot check | 1

Fourth Floor Framing Beam check

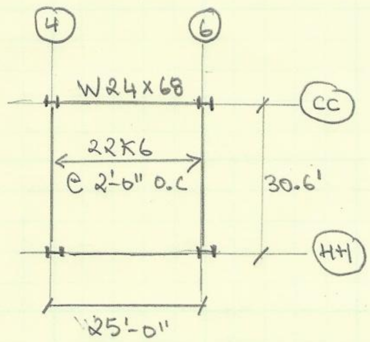


Figure 15: Typical bay

Composite beam CC

W24x68 $\quad A_g = 20.1$

$I_x = 1830$

Loads: $\quad F_y = 50$ ksi

LL = 100 psf

DL = 65 psf (include SDL).

Tributary width = 2'-0"

Span = 25'-0"

$W_u = 1.2D + 1.6L$

$D = (65)(2) + 68 = 198$ plf

$L = 100(2) = 200$ plf

Assume Pin support.

$w_u = 1.2(198) + 1.6(200)$

$w_u = 0.558$ k/ft

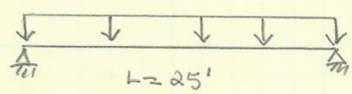


Figure 16: Typical beam

$$V_u = \frac{(0.558)(25)}{2} = 6.97\ \text{k}$$

$$M_u = \frac{wl^2}{8} = \frac{(0.558)(25)^2}{8}$$

$M_u = 43.59$ ft-k.

Full composite action

W24x68 with 4" LW. concrete topping on 1 ½" deck

(5.5" total slab thickness) $f'_c = 4000$ psi, $F_y = 50$ ksi

Span 25 ft, spacing = 2 ft

Deck perpendicular to beam

3/4" ϕ studs (Assumption)

1 stud/rib (Assumption) → $Q_n = 17.2$ kip

Aubert Ndjolba	AE Senior thesis	Beam Spot Check	2

$V'q = A_s F_y = (20.1)(50) = 1005^k \leq A_s F_y = 0.85 f'_c \, b_{eff} \, a$
$\leq \Sigma Q_n$

$\Rightarrow a = \dfrac{A_s F_y}{0.85 f'_c \, b_{eff}}$

b_{eff}: min $\left| \begin{array}{l} \dfrac{\text{span}}{8} = \dfrac{25(12)}{8} = 37.5 \text{ in} \\ \text{spacing} = 2(12) = 24 \text{ in} \leftarrow \text{controls.} \end{array} \right.$

$a = \dfrac{1005}{0.85(4)(24)} = 12.32'' > 3''$ ∴ PNA in web. N.G.

W24x68 , PNA = 7 $\Rightarrow \Sigma Q_n = 251k$ (Table 3-19) STEEL MANUAL

$\Rightarrow a = \dfrac{251}{0.85(4)(24)} = 3.0 = 3''$ use $a = 3.0$

$Y_2 = t_{slab} - \dfrac{a}{2} = 5.5 - \dfrac{3}{2} = 4$

$\phi M_n = 904$ ft-k $\gg M_u = 43.59$ ft-k ∴ ok

$Q_n = \dfrac{251}{17.2} = 14.5 \triangleq 15$ ∴ 30 studs required
< 2 studs per rib ∴ OK

(Table 3-2) $\phi V_n = 295^k \gg V_u = 6.97^k$ ∴ ok

Check Δ_{LL}:

$\Delta_{LL} = \dfrac{l}{360} = \dfrac{(25)(12)}{360} = 0.83$ in maximum deflection

$w_{LL} = (100)(2)/1000 = 0.2$ ksi

(Table 3-20) $I_{LB} = 2840 \text{ in}^4$

$\Delta_{LL} = \dfrac{5 w_{LL} l^4}{384 EI} = \dfrac{5(0.20)(25)^4(1728)}{384(29{,}000)(2840)} = 0.021$

$\Delta_{LL} = 0.021$ in $< \Delta_{LL\,max} = 0.83$ in ∴ OK

Aubert Ndjolba	AE Senior thesis	Beam spot check	3

check beam deflection under concrete wt

$$\Delta_{max} = \frac{l}{240} = \frac{(25)(12)}{240} = 1.25 \text{ in}$$

$$I_{req} = \frac{5 w l^4}{384 E \Delta_{max}}$$

$$w = [(65)(2) + 68]/100 = 0.198 \text{ plf}$$

$$I_{req} = \frac{(5)(0.198)(25)^4(1728)}{384(29,000)(1.25)} = 48.0 \text{ in}^4$$

$$I_{req} = 48.0 \text{ in}^4 << I_{Beam} = 1830 \text{ in}^4 \therefore \text{ok}$$

$$\boxed{\text{W 24 x 68 WORKS}}$$

APPENDIX B: WIND LOADS

Aubert Ndjolba	AE Senior Thesis	WIND LOAD	1

DESIGN CRITERIA : ASCE 7-10

Table 1.5.1 → Risk Category III

Fig 26.5-1B → Basic wind speed : V = 90 mph

Table 26.6-1 → Directionality factor : $K_d = 0.85$

Specification → Exposure Category C

Section 26.8 → Topographic factor : $K_{zt} = 1.0$

Section 26.9 → Gust effect factor: G = 0.85 (Rigid building)

Building height = 70 ft > 60 ft

Design Wind Pressures for MWFRS

$$P = q\,(GC_p) - q_i\,(GC_{pi}) \qquad (psf)/\left(\frac{N}{M^2}\right)$$

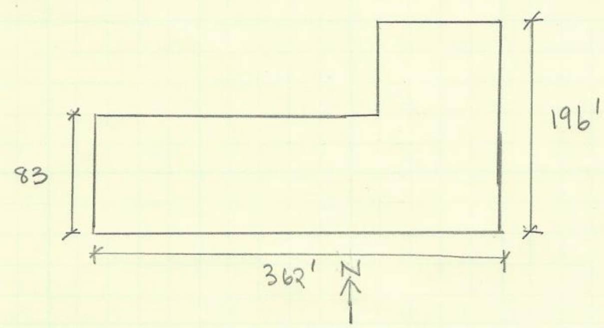


Figure 17: Projected shape   I = 1.15 (Table 6-1)

$q = q_z$ (windward wall)

$q_z = 0.00256\, k_z\, k_{zt}\, k_d\, V^2 I$ $K_z = 1.17$ (table 27.3-1)

$= 0.00256\,(1.17)(1.0)(0.85)(90^2)(1.15)$

$q = q_z = 23.72$ psf (windward wall)

$q = q_h$ (leeward wall, side walls, and roof)

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$q = q_h = 23.72$ psf for all other (walls + roof)

$q_i = q_h = 23.72$ psf (enclosed building)

$(GC_{pi}) = \pm 0.18$ (Table 26.11-1)

$R_i = 1.0$

By Figure 27.4-1

windward wall pressure coeff: $C_p = 0.8$

The side wall pressure coeff: $C_p = -0.7$

Leeward wall pressure coeff.:

For $\frac{L}{B} = \frac{362}{84} = 4.3$, $C_p = -0.2$

For $\frac{L}{B} = \frac{362}{196} = 1.84$, $C_p = -0.3$

Roof C_p: $\frac{h}{L} = \frac{65}{362} = 0.179$ $\theta = 22.6°$

(5/12 slope)

$C_p = -0.2$

$C_p = 0.3$ use $C_p = 0.3$ for Roof.

MWFRS Pressures (For top floors only)

Windward wall:

$P = 23.7(0.85)(0.8) - 23.7(\pm 0.18) = 16.1 \pm 4.26$ psf

Leeward wall:

$P = 23.7(0.85)(-0.2) - 23.7(\pm 0.18) = -4.0 \pm 4.26$ psf

For wind normal to 362 ft

Leeward wall:

$P = 23.7(0.85)(-0.3) - 23.7(\pm 0.18) = -6.0 \pm 4.26$ psf

For win normal to 196 ft

Roof

$P = 23.7(0.85)(0.3) - 23.7(\pm 0.18) = 6.0 \pm 4.26$ psf

APPENDIX C: SEISMIC CALCULATIONS

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ASCE 7-10

(11.4) Seismic ground motion

DL = 65 psf LL = 100 psf Snow L = 30 roof DL = 35

W_{rf} = (35)(10,320) + 0.2(10,320)(30) = 423 K

W_{fL} = [(65)(10,320) + 0.2(100)(10,320)](3) = 2631 K

W_{ground} = [(65)(14,473) + 0.2(100)(14,473)] = 1230 K

Assumptions: Site class D: stiff soil

Per ASCE 7-10

Fig. 22-1 $S_S = 18\%\ g$ for PA

Fig 22-2 $S_1 = 6\%\ g$ for PA

$S_{MS} = F_a S_S$

$S_{M1} = F_v S_1$

$F_a = 1.6$ For PA with $S_S < 0.25$ & Site Class D.

$S_{MS} = 1.6(0.18) = 0.288$

$F_v = 2.4$ For PA with $S_1 < 0.1$ & Site class D

$S_{M1} = 2.4(0.06) = 0.144$

(11.4-3) $S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3}(0.288) = 0.192$

(11.4-4) $S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3}(0.144) = 0.096$

(11.5.1) $I_e = 1.25$ (Risk category III)

By Table 11.6-1 & Table 11.6-2

For PA for $S_{DS} = 0.192$ & occupancy Category (O.C) II

⇒ Seismic Design Category (SDC) = "B"

For $S_{D1} = 0.096$ & O.C II ⇒ SDC = "B"

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Equivalent Lateral Force Procedure: (12.8)

Seismic Base Shear

(Eq 12.8-1) $V = C_s W$

Table 12.2-1 : $R = 3$ for Steel ordinary moment frames

Table 12.8-2 : $C_T = 0.028$ (Steel moment-resisting frame)

$x = 0.8$

Fundamental Period: $T = C_T h_n^x$

$T = (0.028)(70)^{0.8} = 0.84$ sec

From Fig. 22-15, for PA, $T_L = 6$ sec.

$C_s = \frac{S_{DS}}{R/I} = \frac{0.192}{3/1.25} = 0.8$

$T = 0.84$ sec $< T_L = 6$sec

C_s should be $< \frac{S_{D1}}{(R/I)T} = \frac{0.192}{(3/1.25)(0.84)} = 0.095 < 0.8$ ✓

$C_s \doteq 0.8$ OK

Total DL ⇒ $W = W_{rf} + W_{f1} + W_{ground}$

$W = 4{,}284$ k

For PA $V = C_s W = (0.8)(4284) = \boxed{3427 \text{ K}}$

(12.8.3) Vertical Distribution of Seismic Forces

$F_x = C_{vx} V$ (Eq 12.8-11)

where $C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{4} w_i h_i^k}$ $k = 2$ for

(Eq 12.8-12) $0.5 < T \leq 2.5$

APPENDIX D: TYPICAL FLOOR PLANS

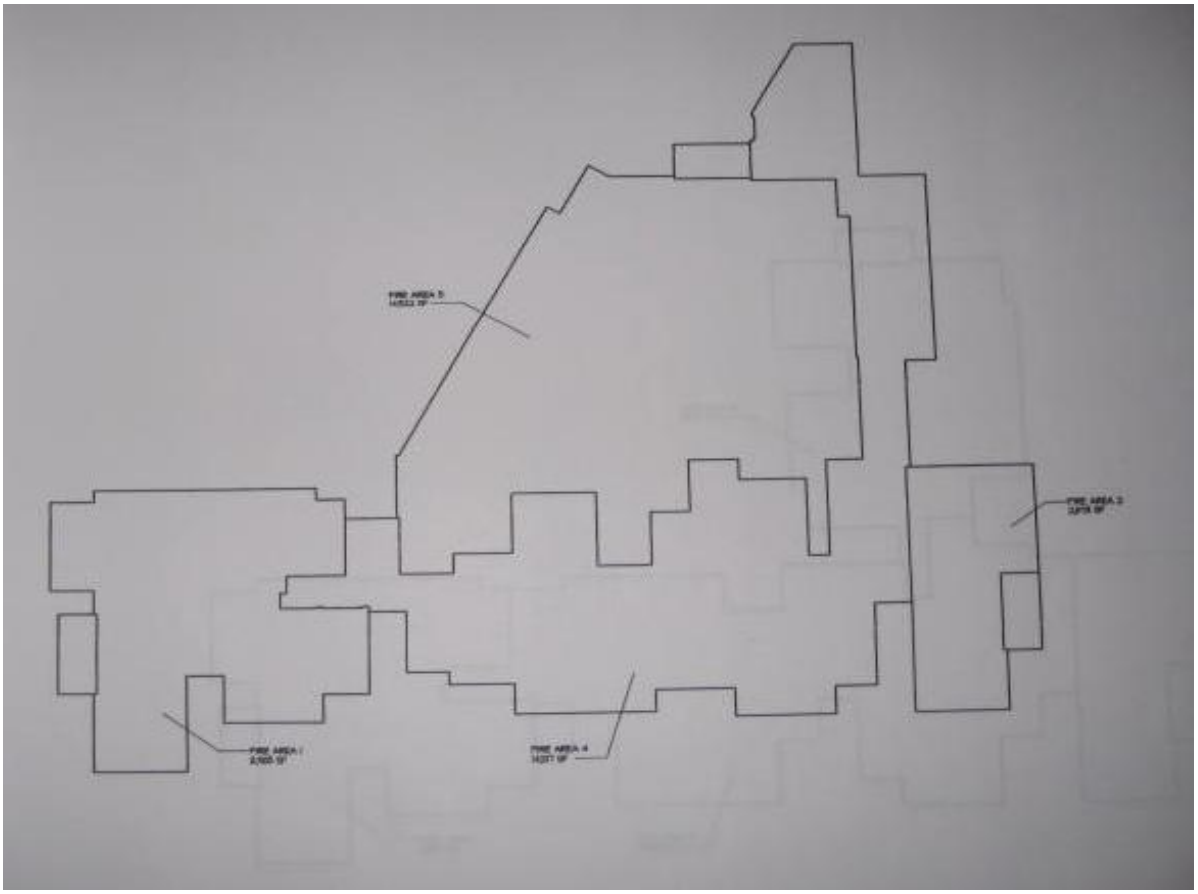

Figure 17: Ground floor

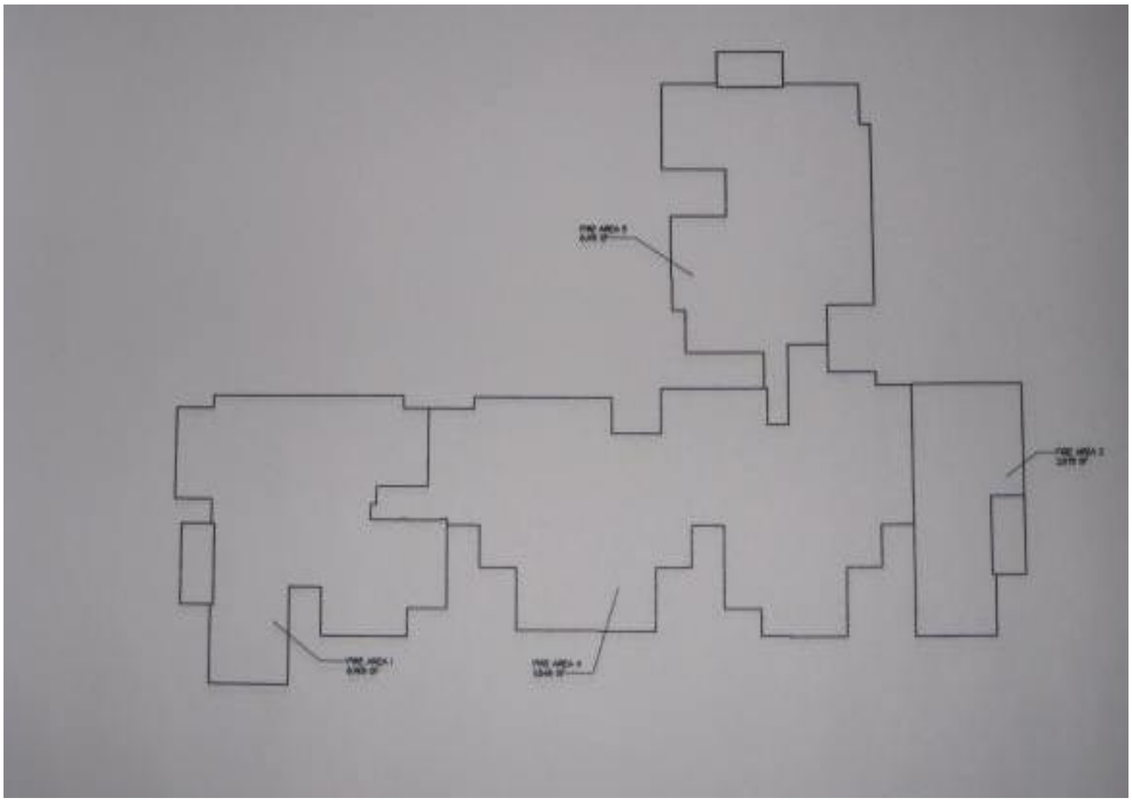

Figure 18: Upper Floors