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UNIVERSITY OF CENTRAL FLORIDA'S ACADEMIC VILLAGES
ORLANDO, FL

Executive Summary
Structural Technical Report 1
Structural Concepts / Structural Existing Conditions Report
October 5, 2005

The 4 story complex in University of Central Florida is a cast in place concrete structure with reinforced masonry shear walls and a shallow foundation system. The roof is a hip roof composed of a combination of trusses supporting metal decking.

The main codes used for the design of the Academic Villages are the 1999 Standard Building Code (SBC), American Institute of Steel Construction (AISC) Load and Resistance Factor Design (LRFD), Specifications for Structural Concrete (ACI 301), and Specifications for Masonry Structures (ACI 530.1). The materials are specified by the American Society for Testing and Materials (ASTM).

I calculated the design loads using ASCE 7-02 and found the design loads in the structural drawings to be less than what I calculated using ASCE 7-02. Due to the location, I expected the wind loads to be significantly greater than the seismic loads. However, this is not consistent with my calculations. I will be investigating this concept further.

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BUILDING DESCRIPTION:

The University of Central Florida's Academic Villages is a complex of ten separate dormitories built to accommodate 500 new freshman students. The buildings are various sizes ranging from 14,000 square feet to 22,000 square feet. The buildings are 4 stories tall and 44'-8" above the ground. Each floor has between eleven and fifteen 24 ft x 28 ft apartment units.

FOUNDATION:

The foundation used in the Academic Villages is a shallow foundation system consisting of continuous strip footings to support 8" masonry shear walls and stepped footings of various sizes centered under the interior concrete columns. The footings were designed to take the maximum soil bearing pressure of 2000 psi. The footings work together with a 4" concrete slab on grade. Both the footings and the slab have a 28 day strength of 3000 psi.

LATERAL SYSTEM:

The lateral system for the Academic Villages uses both exterior and interior shear walls in both N-S and E-W directions to resist seismic and wind forces. The exterior shear walls, which surround the entire building also serves as a load bearing wall. All shear walls are 8" masonry units with a strength of 3000 psi and Type S mortar.

TYPICAL FLOOR:

The Typical floor consists of 2" 22 gage Epicore metal decking galvanized with 4 ½" concrete slab reinforced with 6x6 - W2.9xW2.9 welded wire fabric. The slab has a 28 day strength of 3000 psi. Supporting the floors are 3'10" to 5'10" x 6" to 8 ½" concrete slab beams of various lengths.

BEARING WALLS:

In addition to the exterior 8” masonry bearing walls which surround the building spanning its height uninhibited, the interior bearing walls are 14 to 20 gage metal stud walls spaced 16” o.c.

COLUMNS:

Concrete Columns with a 28 day compressive strength of 4000 psi span only between the foundation and the first floor. The columns are reinforced with #6 bars and #3 ties at various spacings. In addition to the concrete columns, there are also light gage metal built-up columns incorporated within the metal stud walls. These columns are found on every floor.

ROOF:

The roof of the Academic Villages is a hip roof consisting of hip trusses, girder trusses and light gage metal trusses spaced 4’ o.c. All trusses are shop fabricated and have a minimum yield strength of 33 ksi. Metal roof decking is 11” - 2Ø Gauge Galvanized G-9Ø spanning a minimum of 3 spans. Several of the buildings have flat roofs. The roofs of these building consist of the same Epicore metal decking and concrete slab found in the floor systems.

BUILDING CODES AND REGULATIONS:

The building code used for the design of the Academic Villages was the 1999 Standard Building Code (SBC). Minimum design loads were calculated using ASCE 7-95. Other design codes and standards are listed below:

Structural Steel	American Institute of Steel Construction (AISC) Load and Resistance Factor Design (LRFD) American Society for Testing and Materials (ASTM)
Concrete and Reinforcing	Specifications for Structural Concrete (ACI 301)
Masonry	Specifications for Masonry Structures (ACI 530.1)

DEAD LOAD:

Assuming that normal weight concrete and masonry was used, I estimated the total dead load of one of the 13,800 square foot unit buildings in the Academic Villages to be around 2785 kips. This load takes into consideration the roof components, the slab on grade on each level and the masonry bearing walls. Concrete columns were not considered in this design since they only exist on the first floor and do not contribute to the overall dead load. Steel build-up columns and stud walls were also neglected since the weight contributed by these is very small in relation to the entire structure.

REQUIRED LOADS:

Design Live Loads

Roof	20 psf
Corridors	80 psf
Mechanical Rooms	150 psf
Stairs, Public Areas, Lobby	100 psf
All Other Rooms	40 psf

Superimposed Dead Loads

M/E/P	10 psf
Partitions	20 psf

SNOW LOADS:

According to ASCE 7-02 Section 7.0 "Snow Loads", the ground snow load, p_g for Florida is equal to zero according to figure 7.1. Therefore, snow loads don't need to be taken into consideration for this building.

LATERAL LOADS:

Using the ASCE 7-02 to calculate the lateral loads acting on each floor due to wind and seismic, I found that the base shear on the structure due to wind was 72.4 kips. The base shear due to seismic loads was nearly twice that at 131.6 kips. The loads found in the structural drawings are slightly less than my wind calculations and significantly less than my seismic calculations. In conclusion to this, I'm not sure if these calculations are valid since wind forces almost always control over seismic forces in the Florida areas.

APPENDICES:

A. Dead Load Calculation

B. Wind Load Calculation

C. Seismic Load Calculation

D. Building Sketches

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Design Calculations

Project Appendix A	Prepared by	Date
Subject/Title Dead LOAD Calculations	Reviewed by	Date
	Calculation Number	Sheet of

Assume normal weight concrete (150 pcf)

Roof → 10 psf + 35 psf + 5 psf = 50 psf

M/E/P ↑ Trusses ↑ Steel Deck ↑

Floors → $(150 \text{ pcf})(4''/12) = 50 \text{ psf}$

Masonry bearing walls → $(65)(141' + 141' + 68' + 68' + 73' + 73')(8''/12)$

$(w = 65 \text{ pcf}) = 24{,}420 \text{ lbs}$

columns → concrete columns are located on ground level so they dont contribute, built-up columns are neglible.

$W = \Sigma DL = (50 + 50 + 50 + 50 \text{ psf})(13{,}800 \text{ sq ft})$

$+ 24{,}420 \text{ lbs} = \boxed{2785 \text{ K}}$

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Subject/Title: WIND LOAD Calculations	Reviewed by	Date
	Calculation Number	Sheet of

From Structural Drawings

$V = 100$ mph
$I = 1.15$

Assume.
- Exposure Category B

Height above ground	K_Z
0 - 15'	0.57
15'-20'	0.62
20'-25'	0.66
25'-30'	0.70
30'-40'	0.76

from Table 6.3 based on assumption of exposure category B

$q_Z = 0.00256 K_Z K_{Zt} K_d V^2 I$ (eq. 6-15)

$K_d = 0.85$ (Table 6.4)

$K_{Zt} = (1 + K_1 K_2 K_3)^2$ (eq 6-3)

$= 1.0$ (conservative)

$\Rightarrow q_Z = 0.00256(1.0)(.85)(100)^2(1.15)\ K_Z$

$= 25.024\ K_Z$

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Subject/Title: WIND LOAD CALCULATIONS	Reviewed by	Date
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WINDWARD

Height (ft)	K_Z	q_z (psf)
0-15'	0.57	14.26
15-20'	0.62	15.51
20-25'	0.66	16.52
25-30'	0.70	17.52
30-40'	0.76	19.02

LEEWARD

$q_h = 0.00256(1.0)(.85)(100)^2 \times (1.15)(0.76)$

$= 19.02$ psf

$p = q_z G C_p$ (neglecting internal pressure) (eq 6-17)

$G = 0.85$ or $0.925\left(\frac{(1+1.7 g_Q I_{\bar{z}} Q)}{1+1.7 g_v I_{\bar{z}}}\right)$ (eq. 6-4)

↑ Rigid

$C_p = 0.85$ windward

$= 0.25$ leeward (fig 6-6)

$\Rightarrow p = q_z(.85)(.85)$

$= 0.7225\, q_z$

$p = q_h(.85)(-.25)$

$= -0.2125\, q_h$

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Design Calculations

Project: Appendix B	Prepared by	Date
Subject/Title: WIND LOAD CALCULATIONS	Reviewed by	Date
	Calculation Number	Sheet of

Design wind Pressure

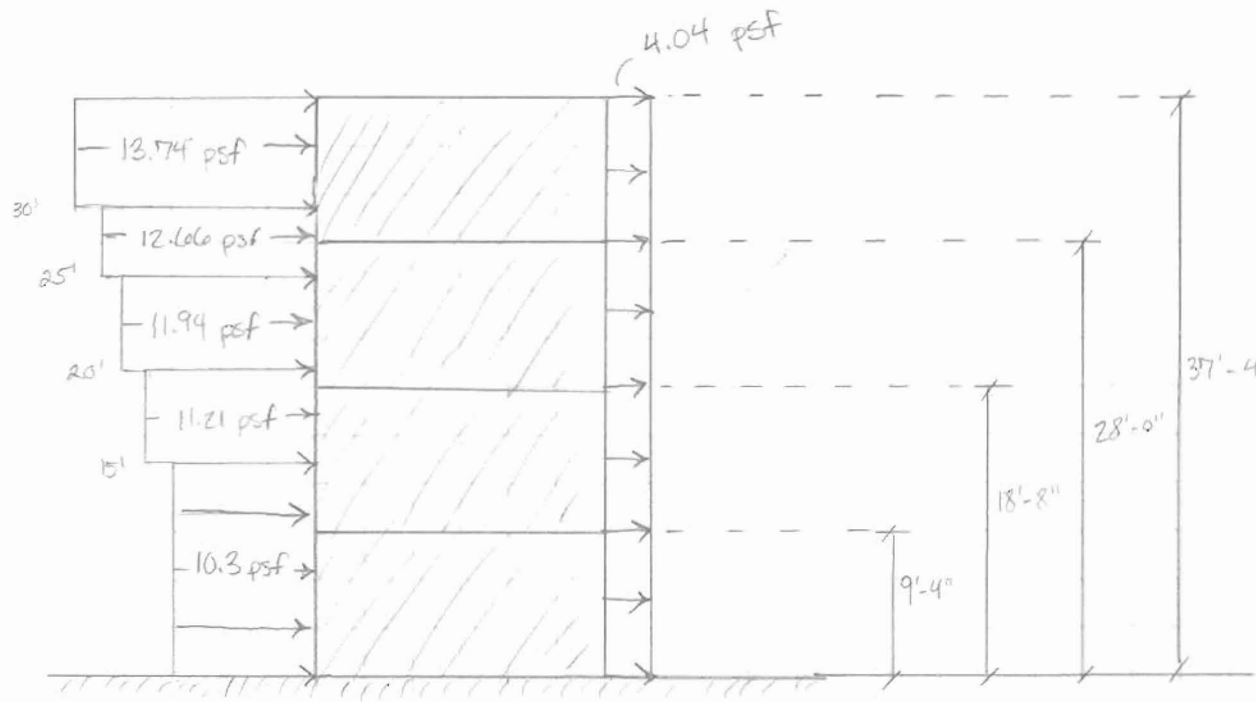
WINDWARD

Height (ft)	q_z (psf)	P (psf)
0-15	14.26	10.30
15-20	15.51	11.21
20-25	16.52	11.94
25-30	17.52	12.66
30-40	19.02	13.74

Leeward

$P = (19.02)(.85)(-.25)$

$= -4.04$ psf



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Design Calculations

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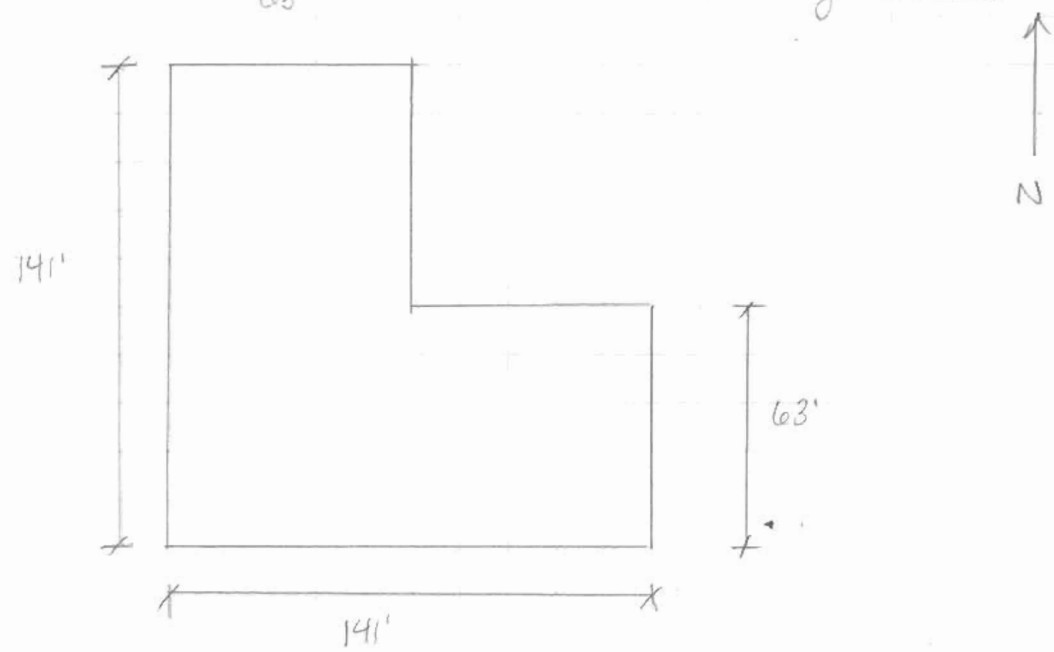
Building footprint



$$F_4 = \left[(13.74)(4.625') + 4.04(4.625')\right](141')$$
$$= 11.59^K$$

$$F_3 = \left[13.74(2.625') + 12.66(5.0') + 11.94(1.625') + 4.04(9.25')\right](141')$$
$$= 22.02^K$$

$$F_2 = \left[11.94(3.29) + 11.21(5.0') + 10.3(.958) + 4.04(9.25')\right](141')$$
$$= 20.1^K$$

$$F_1 = \left[10.3(9.25') + 4.04(9.25')\right](141')$$
$$= 18.7^K$$

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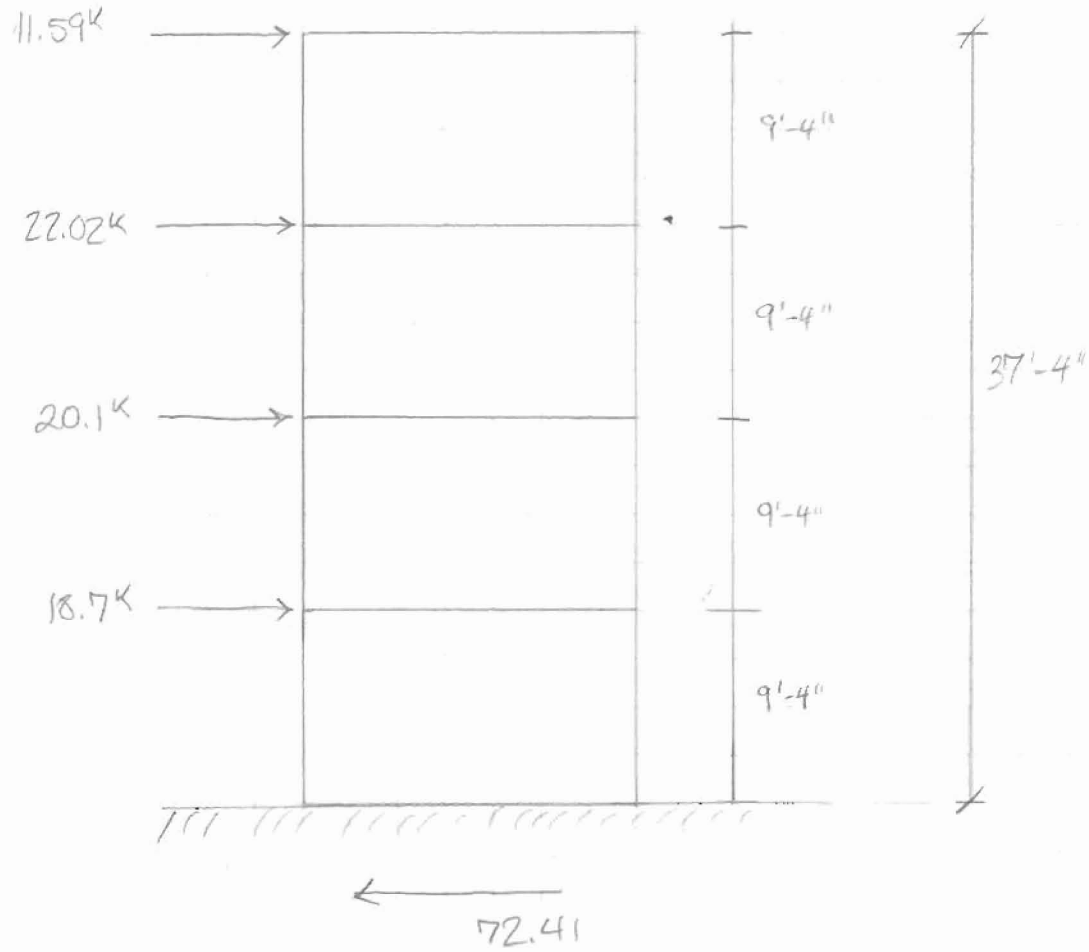
East/West and North/South have same result due to symmetry



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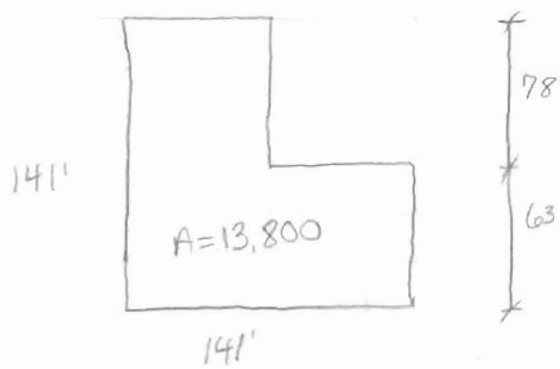
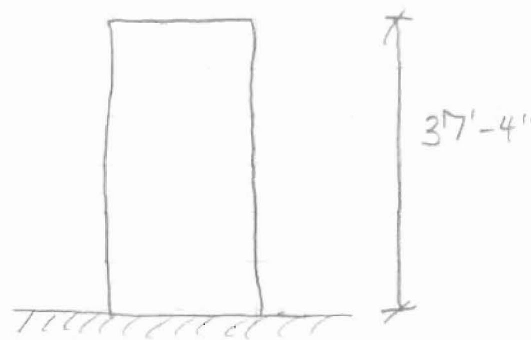
Design Calculations

Project: Appendix C	Prepared by	Date
Subject/Title: Seismic Calculations	Reviewed by	Date
	Calculation Number	Sheet of





Roof Dead Load = 35 psf (trusses) + 10 psf (M/E/P) + 5 psf (Roof deck) = 50 psf

Floor Dead Load = (150 pcf)(4"/12) + 10 psf = 60 psf

Masonry WALL Dead Load = (63 pcf)(8"/12) = 42 psf

Assumptions

- Seismic Use Group II since theres a large number of people there. (I=1.25) (Table 9.1.3)
- Site Class D since allowable soil pressure is 2000 psi (Table 9.4.1.2)

From fig 9.4.1.1 a)

$S_s = 7.5\%$

From fig 9.4.1.1 b)

$S_1 = 3.0\%$

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Project: Appendix C	Prepared by	Date
Subject/Title: Seismic Calculations	Reviewed by	Date
	Calculation Number	Sheet of

From Table 9.1.2.4a

$F_a = 1.6$

From Table 9.1.2.4b

$F_v = 2.4$

$S_{MS} = F_a S_S = (1.6)(.075) = 0.12$ (eq 9.4.1.2.4-1)

$S_{M1} = F_v S_1 = (2.4)(.03) = 0.072$ (eq 9.4.1.2.4-2)

$S_{DS} = 2/3\, S_{MS} = 2/3(.12) = 0.08$ (eq 9.4.1.2.5-1)

$S_{D1} = 2/3\, S_{M1} = 2/3(.072) = 0.048$ (eq 9.4.1.2.5-2)

From Table 9.4.1.2

Seismic Design Category A

Seismic Base Shear

$V = C_s W$ (eq 9.5.5.2-1)

$W_{Roof} = (13,800\ ft^2)(50\ psf) = 690^K$

$W_{Floor} = (13,800\ ft^2)(60\ psf) = 828^K$

$W_{Wall} = (42\ psf)(4 \times 141')(37.25') = 882^K$

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Seismic Response Coefficient (eq 9.5.5.2.1-1)

$$C_S = \frac{SDS}{R/I} \leq \frac{SD_1}{(R/I)T}$$

$$\geq 0.044\ SDS\ I$$

- $R = 3$ ordinary reinforced masonry shear walls (Table 9.5.2.2)
- $I = 1.25$
- $T = C_t h_n^x$ (eq 9.5.5.3.2-1)

$C_t = (0.02)(0.055)$
$x = .75$
$h_n = 37.25'$

$\Rightarrow T = (.02)(.055)(37.25)^{.75} = 0.017$ sec

$$\Rightarrow C_S = \frac{0.08}{3/1.25} = 0.0333 \leq \frac{0.048}{(3/1.25)(.017)} = 1.17 \quad \underline{ok}$$

$$\geq .044(.08)(1.25) = .0044 \quad \underline{ok}$$

$$\Rightarrow C_S = 0.0333$$

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Total weight

wall DL, perimeter, height

$$W_{roof} = 690^K + (42)(4 \times 141)(4.625') = 800^K$$

$$W_{floor} = 828 + (42)(4 \times 141)(9.25') = 1047^K$$

$$W_{TOTAL} = 800 + 3(1047) = 3941^K$$

$$V = C_s W = 0.0333(3941^K) = 131.6^K$$

$$C_{v_x} = \frac{w_x h_x^K}{\sum w_i h_i} \qquad K = 1.0 \text{ since } T < 0.5 \text{ sec}$$

$$\sum w_i h_i = 800(37.25) + 1047(28' + 18.75' + 9.5')$$
$$= 88{,}693.75 \text{ lb}$$

$$C_{v_4} = 800(37.25)/88693.75 = 0.34 \Rightarrow F_4 = 0.34(131.6) = 44.7^K$$

$$C_{v_3} = 1047(28')/88693.75 = 0.33 \Rightarrow F_3 = 0.33(131.6) = 43.4^K$$

$$C_{v_2} = 1047(18.75')/88693.75 = 0.22 \Rightarrow F_2 = 0.22(131.6) = 29.0^K$$

$$C_{v_1} = 1047(9.5)/88693.75 = 0.11 \Rightarrow F_1 = 0.11(131.6) = 14.5^K$$

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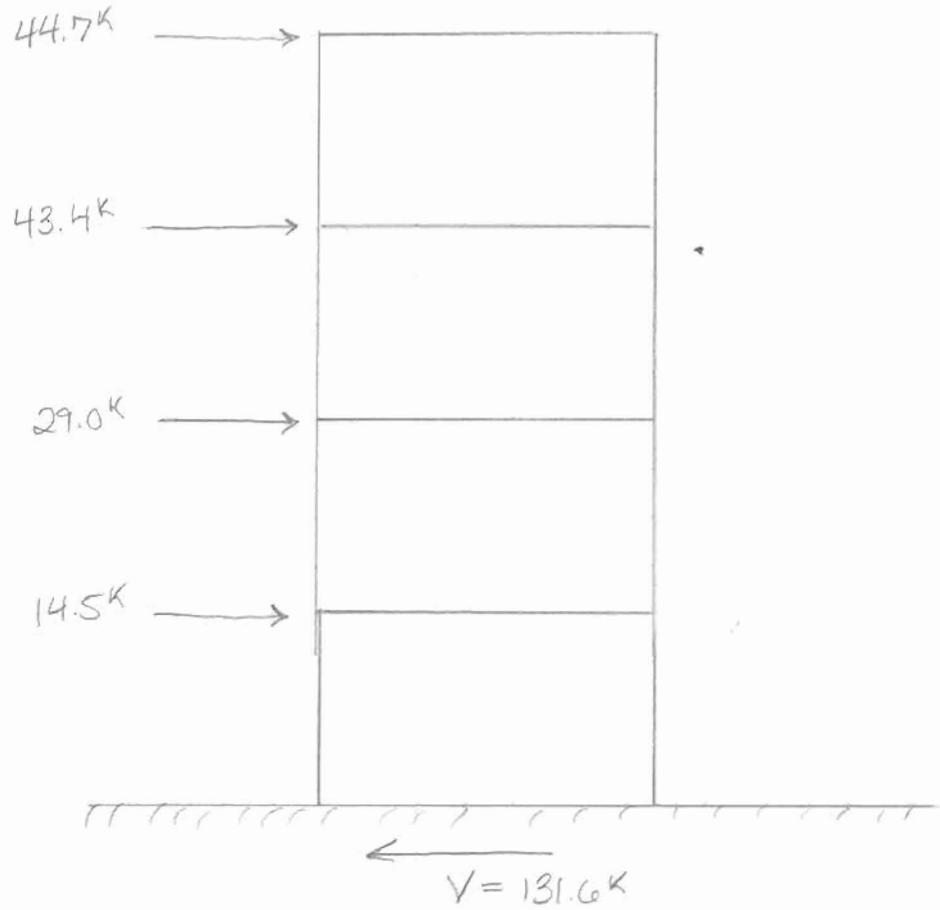

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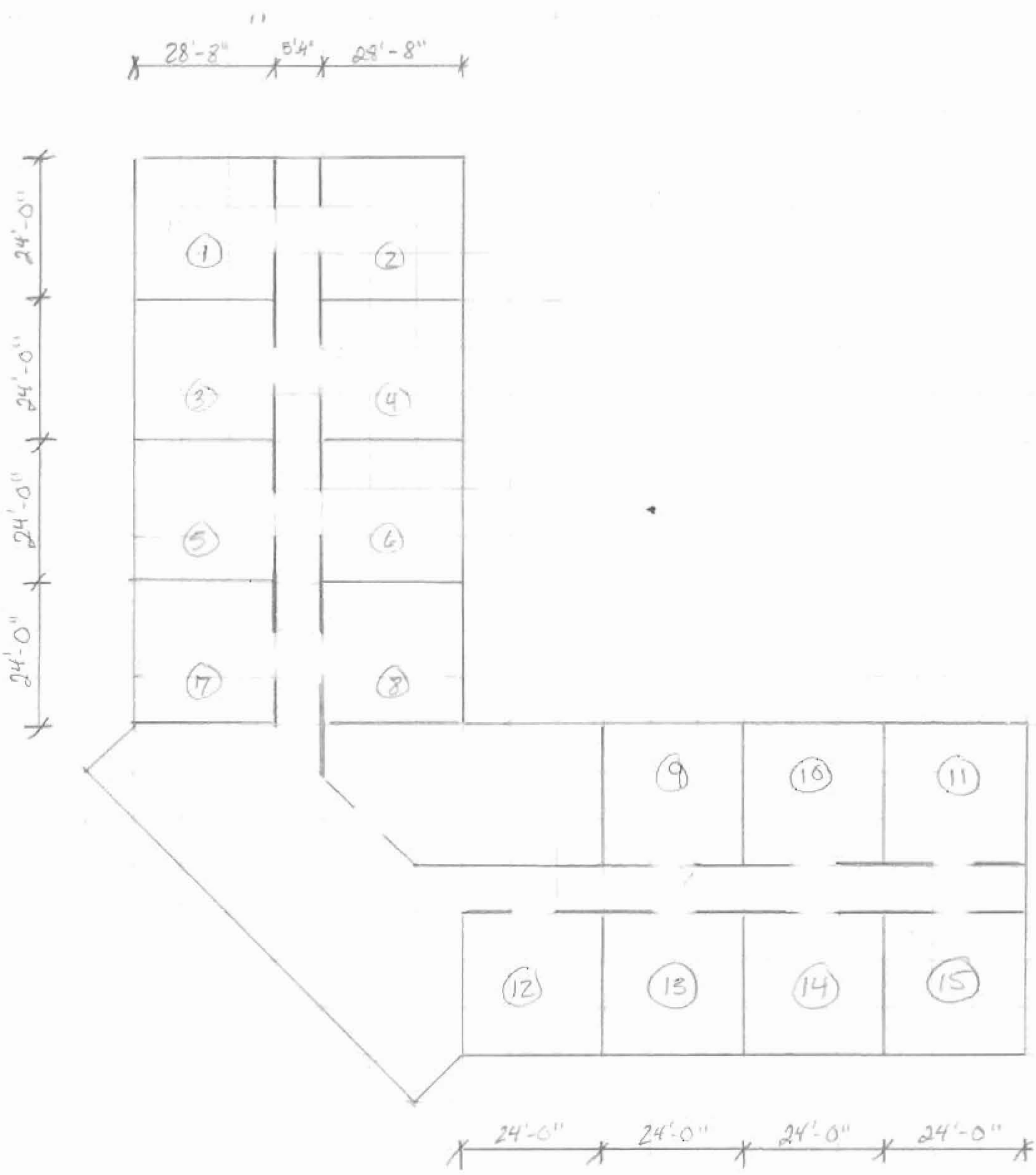
Project: Appendix D	Prepared by	Date
Subject/Title: Typical Floorplan / Layout	Reviewed by	Date
	Calculation Number	Sheet of



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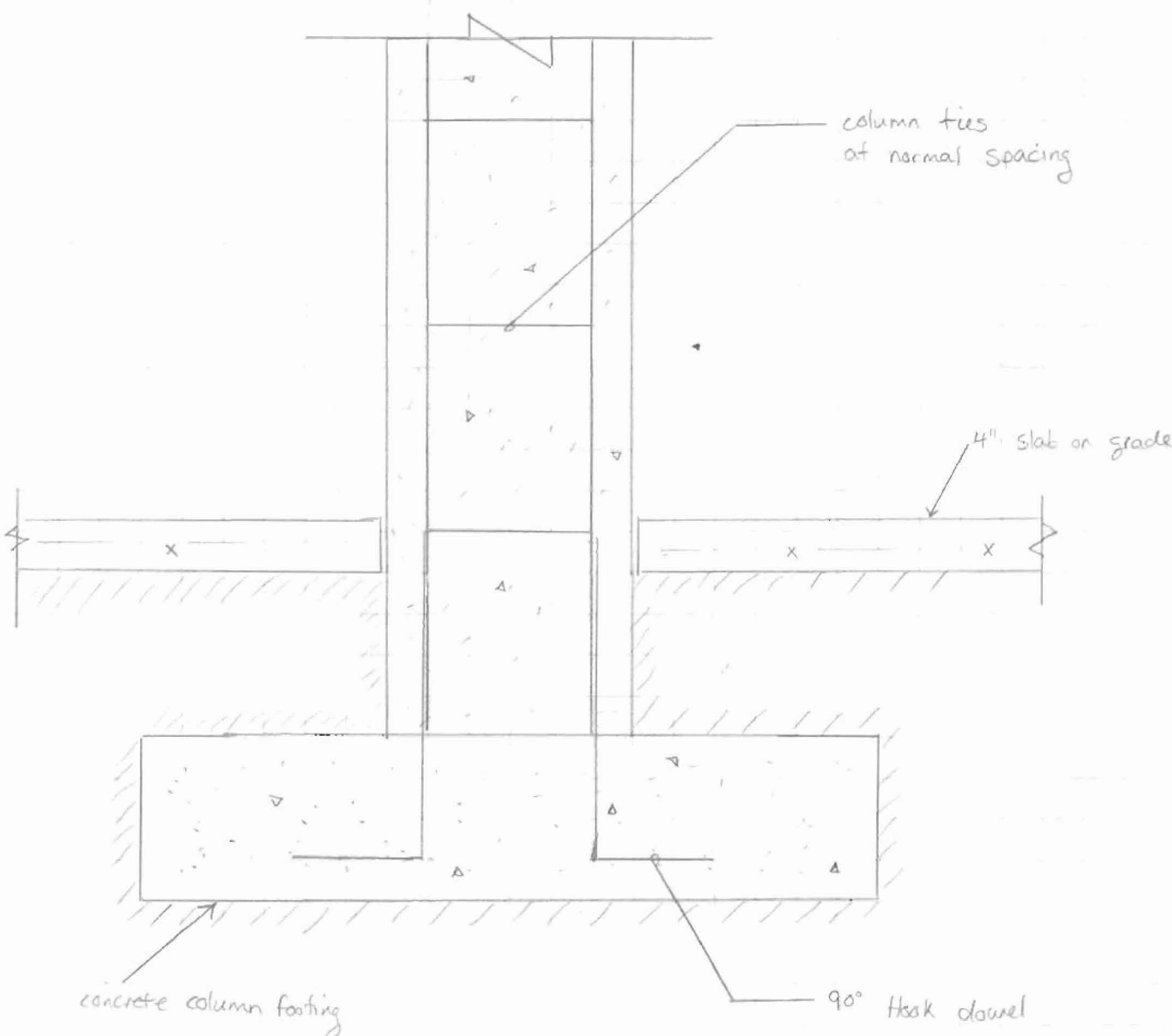
Design Calculations

Project Appendix D	Prepared by	Date
Subject/Title Concrete Column Footing Detail	Reviewed by	Date
	Calculation Number	Sheet of



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Project Appendix D	Prepared by	Date
Subject/Title Floor / Deck / Bearing WALL Detail	Reviewed by	Date
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