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CALIFORNIA STATE UNIVERSITY, LOS ANGELES

CE 4630
Timber & Masonry Design Final Project

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Introduction

California State University, Los Angeles CE 4630 wants a new two-story building that has a wood floor, wood roof and reinforced masonry walls. The client has chosen our team to design this timber and masonry project. The project instructions were given to us with a design checklist and a pair of drawings of the buildings ground floor plan, Second floor plan, roof plan and the buildings elevations from each side. Based on the information given to our team; we designed various parts of the building that was mentioned on the checklist. Our teams design process was divided into different parts. We first determined all the loadings that the building will have on the high roof/ ceiling, 2nd floor, ground floor and CMU walls. After determining the loads, we designed the timber part of the project. The timber part consists of designing the space and size of ceiling joists, floor joists, floor beams, roof beams, stud walls, headers, columns and roof ledger. For the masonry part we designed the CMU parapets, CMU walls thickness, bearing walls, pilasters, headers and gravity loads. Lastly, we checked that everything meets the code requirements.

Summary of Design

1. Weights of Materials

Designing a two-story building is determined by the dead loads and the live loads it must support. The loading is determined from the weight of the materials that comprise the structure and the structures intended usage. The self-weight of the structures materials is the dead loads and the usage or occupancy loading, which is called the live load. The live loads for the high roof is 20 psf and the live loads for the floor is 50 psf. The live loads were prescribed in the building code to account for a person working on the roof during construction and routine maintenance, which is the same for the floor. The dead load for the high roof is a total of 20 psf. It consists mechanical/ electrical/ plumbing (M/E/P), two Air handling Units (AHU), light rolled-on roofing, allowance for one reroof, an inch of rigid insulation, sheathing and framing. For the second floor we determined that the dead loads will consist of M/E/P, carpet finish, one inch of cement topping, ¾" thick plywood subfloor, framing, the loading of the office ceiling and partitions. The total of the dead loads of the second floor is 60 psf. The ground floor consists of the same as the second floor, including the load of the second floor which is a total of 95 psf. For the masonry walls we considered the out of plane wind loads and the fully grouted CMU wall which gives us a total dead load of 114 psf. The structure consists of a garage ceiling that has a high roof and reflective fiberglass batt insulation that has a total dead load of 22 psf. The office ceiling has a high roof, one-inch fiberglass batt insulation and a 5/8" gypsum board which all gives a total dead load of 23 psf.

The following are the total Dead Loads and Live Loads:

High Roof Dead Loads	PSF
M/E/P	2
AHU (2)	5
Light Rolled-on Roofing	1
Allowance for 1 ReRoof	1
1 inch Rigid Insulation	1.5
Sheathing	1.5
Framing	3
Total:	20

Office 2nd Floor Dead Loads	PSF
M/E/P	2
Carpet Finish	2
1 inch of Cement Topping	12
3/4" thick plywood subfloor	2.3
Framing	3
Office Ceiling	22.84
Partitions	15
Total:	59.14

Office 1st Floor Dead Loads	PSF
M/E/P	2
Carpet Finish	2
1 inch of Cement Topping	12
3/4" thick plywood subfloor	2.3
Framing	3
Office 2nd Floor	59.14
Partitions	15
Total:	95.44

Garage Ceiling Dead Loads	PSF	
High Roof	20	
Reflective Fiberglass Batt Insulation	0.5	per inch
Total:	22	
Office Ceiling Dead Loads	**PSF**	
High Roof	20	
5/8" Gypsum Board	2.8	
1" Fiberglass Batt Insulation	0.04	per inch
Total:	22.84	

Reinforced Masonry Wall	PSF
Out-of-Plane Wind Loads	30
Fully Grouted CMU Wall	84
Total:	114

Live Loads	PSF
High Roof	20
Floor	50

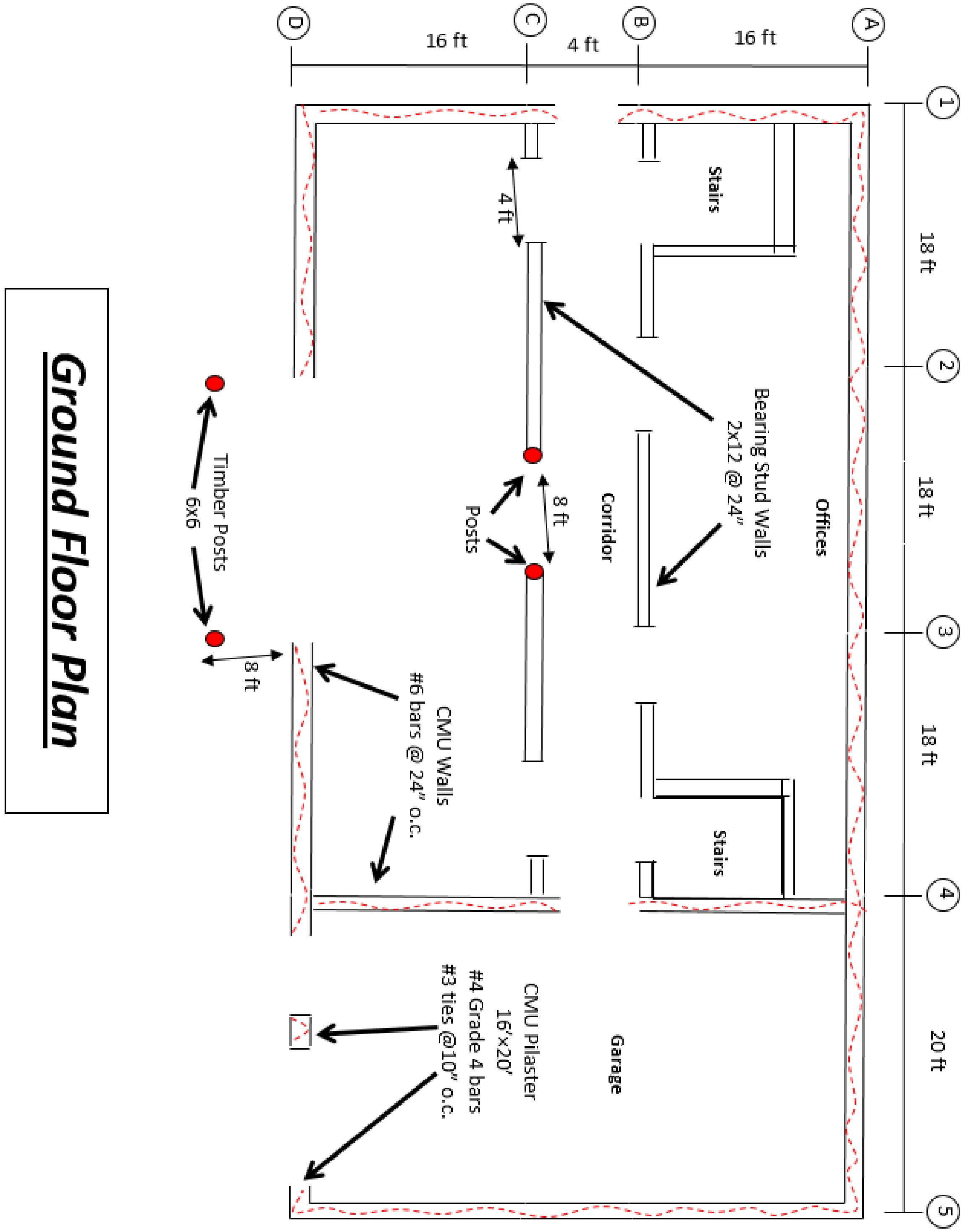
2. Timber Part

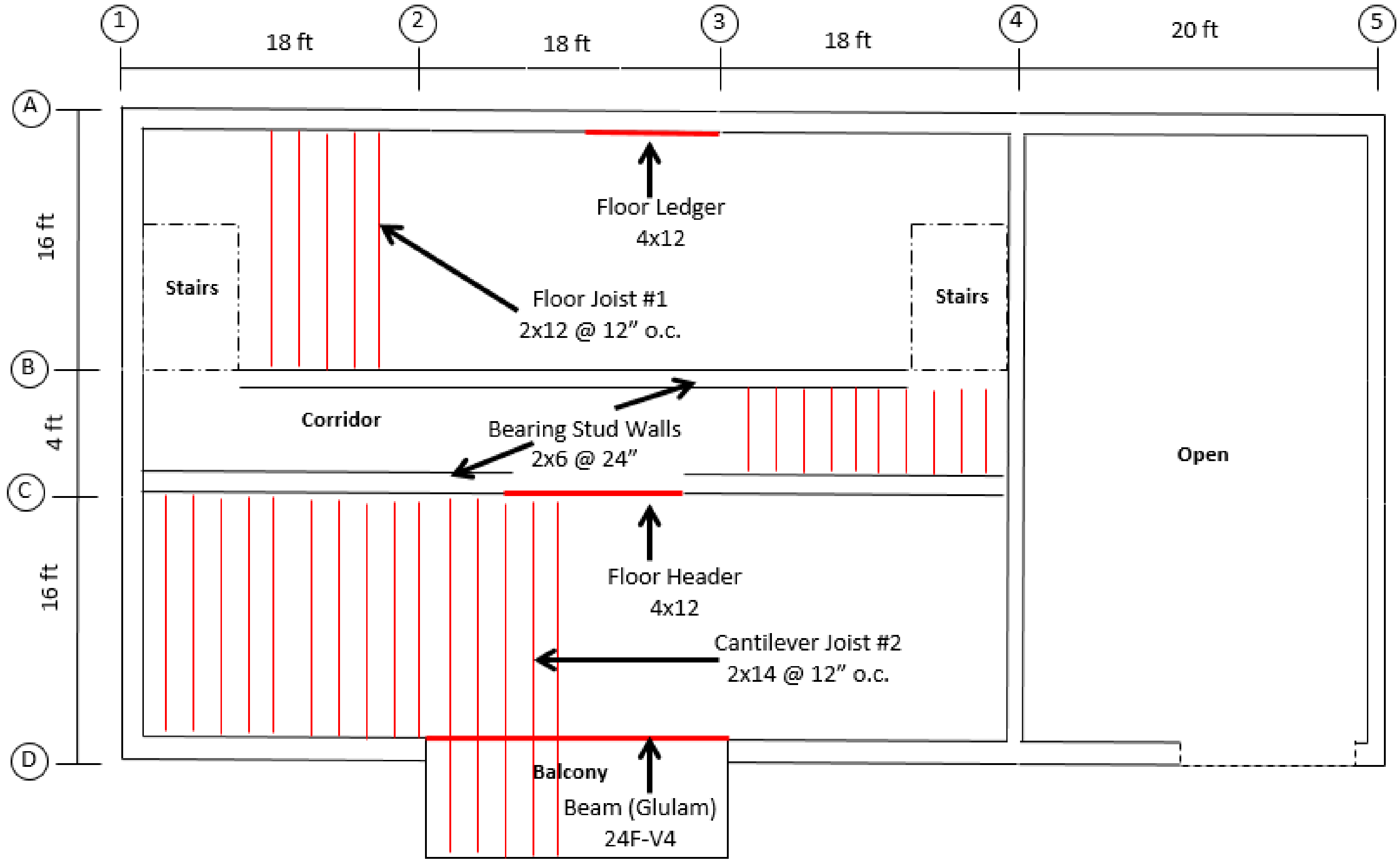
The timber part was broken into three main components; high roof, low roof, and floor. Starting with the high roof once the weight of the material was known, the wood framing that comprises the high roof could be designed. The layout of the roof was based of use and geometry of the building. Starting with the roof we calculated the roof joist at the celling to be 2x8 at a 16" O.C. span up to 16'-6". To come up with this design we first used ASD tp check that the flexure checked out followed by the shear stress, and the deflection once those we concluded we approved the use of a 2x8 joist. For the second set of roof joist using the same design method ASD we checked that all criteria were met we decided to use a 2x4 celling joist at a 12" O.C. and a span of 8'-9". A third set of celling joist was also design at 2x10 16" O.C. and span up to 21'-1". The same process was used to design the floor joist which came out to 2x12 with 12" O.C. and up to 18'-0" and 2x14 with 12" O.C. span up to 20'-1". After doing the calcs of the roof and floor beams we found the best design would be compromised of Glulam. The Roof beam and second floor beam was designed to be a DF 24F-V4. For the stud walls we design checked the bending stress on the stud and axial compressive stress on a stud after these checked out we came out with the design of a 2x6 at 24" at the 2nd floor and a 2x12 at 24". The biggest load in the building were at the air handling units where we had to have extra checks to ensure they would take the load. After checking them using bending stress and shear we found that a 4x12 was most appropriate. The header design carried over the same design checks in checking for bending, shear, and deflection. After these checked out we chose to use a 4x12 for the 2nd floor header and floor header. As for the timer columns we used live load reduction and a strength check to ensure the columns could handle the loads at a 20-foot height and 6x6 dimensions. The roof and floor ledgers were designed using a 4x12 at 5 ft O.C. where bending, shear, and deflection all met design standards. Overall the timber design of the building is more than safe to handle the given loads and more.

3. Masonry Part

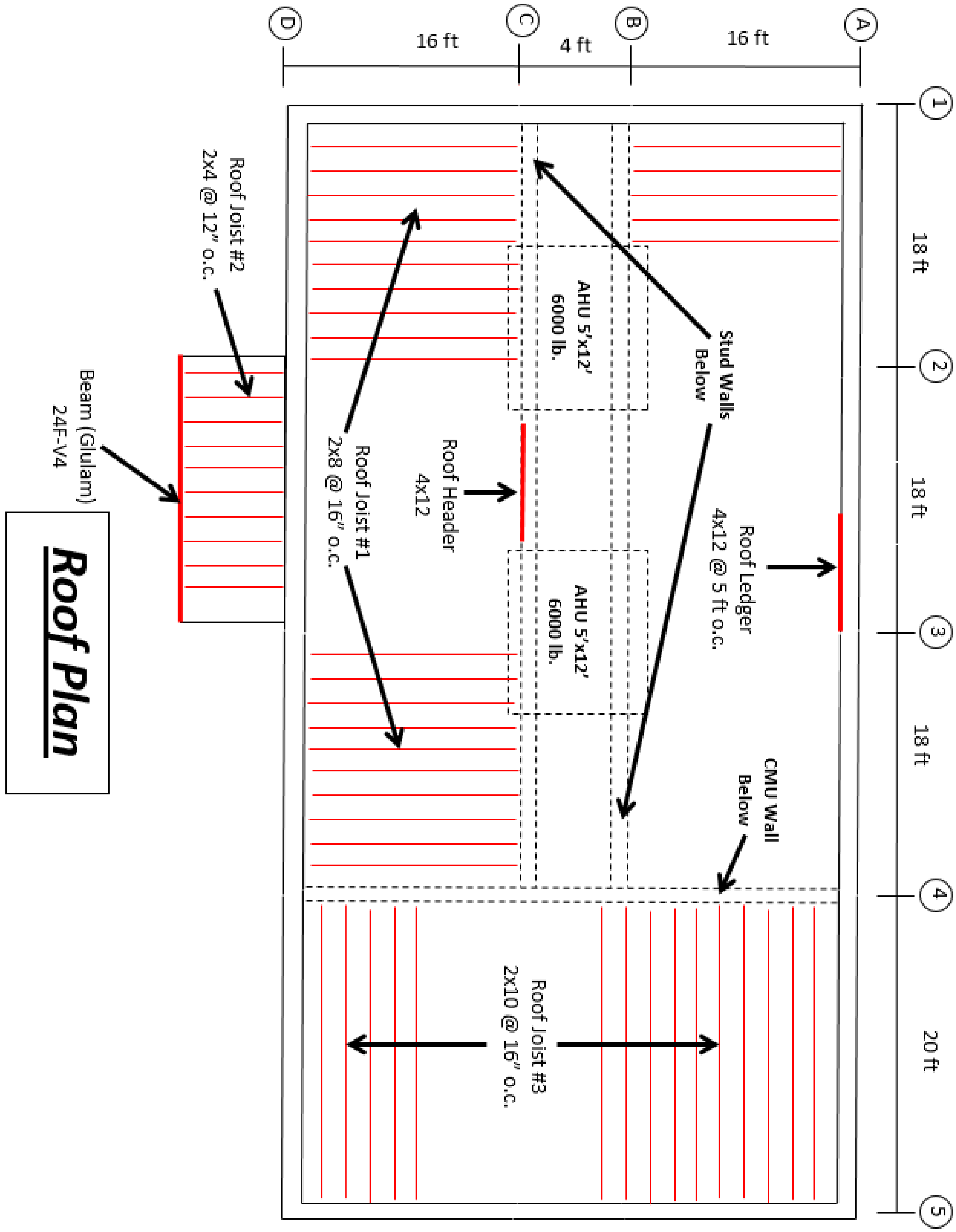
The masonry part of this design project describes calculations for CMU parapet, CMU wall thickness, CMU walls as bearing walls, CMU pilaster, CMU headers under various loads and the main objective is to create the structure which is capable to withstand with these loads without failure. Starting with (1) the Parapet wall, which is 43" high and 10" thick has #5 Grade bars @24" o.c. and it satisfies max. and min. reinforcement to withstand with 30psf wind load. (2) we took 10" thick CMU wall with layer of #6 bars @16" o.c horizontally which are sufficient to resist the shear demand of wall and adequate to force flexural capacity. (3) For bearing walls, we took #6 bars @24" o.c. and has 42.5" development length which gives enough flexural capacity and axial capacity. Piers along line D between windows has #4 bars @16" o.c. with 14.53" lap length. Which are suitable for withstand with 30psf out of plane wind load. (4) We provided 16'×20' pilaster along line D, it has 4 numbers of #4 bars with #3 bars ties @8" o.c., with lap length which gives adequate axial and flexural strength to carry out of plane wind load, dead load and self-weight. (5) Header along line D over large garage door, we provided #6 grade 2 bars @6" o.c. with stirrups of #4 grade 8 bars @20" o.c. Header over the balcony has little bit less self-weight compare to header over garage door, so we took #5 grade -2 bars @6" o.c. with stirrups #4 grade-10 bars @24" o.c.

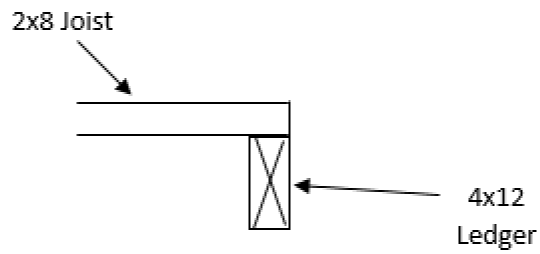
Project Plans

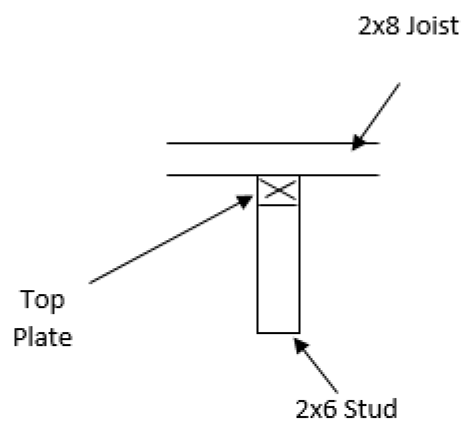


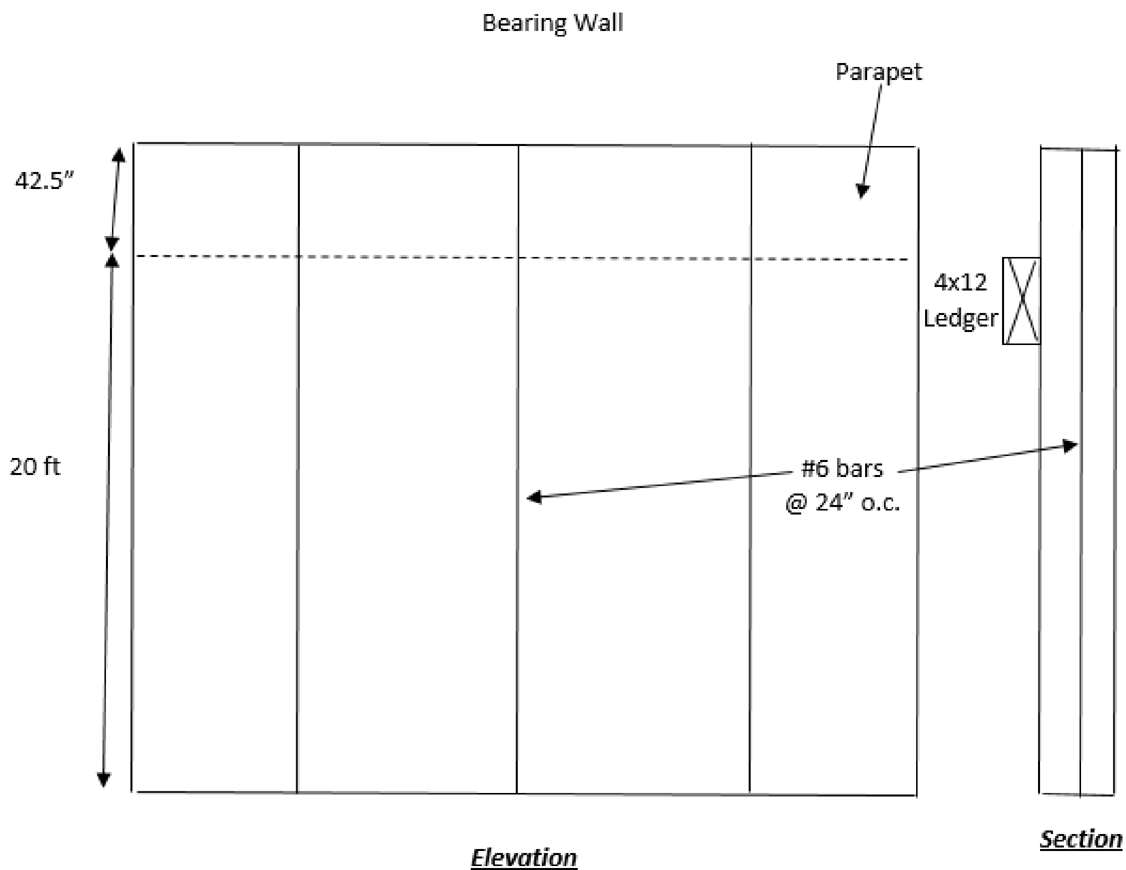


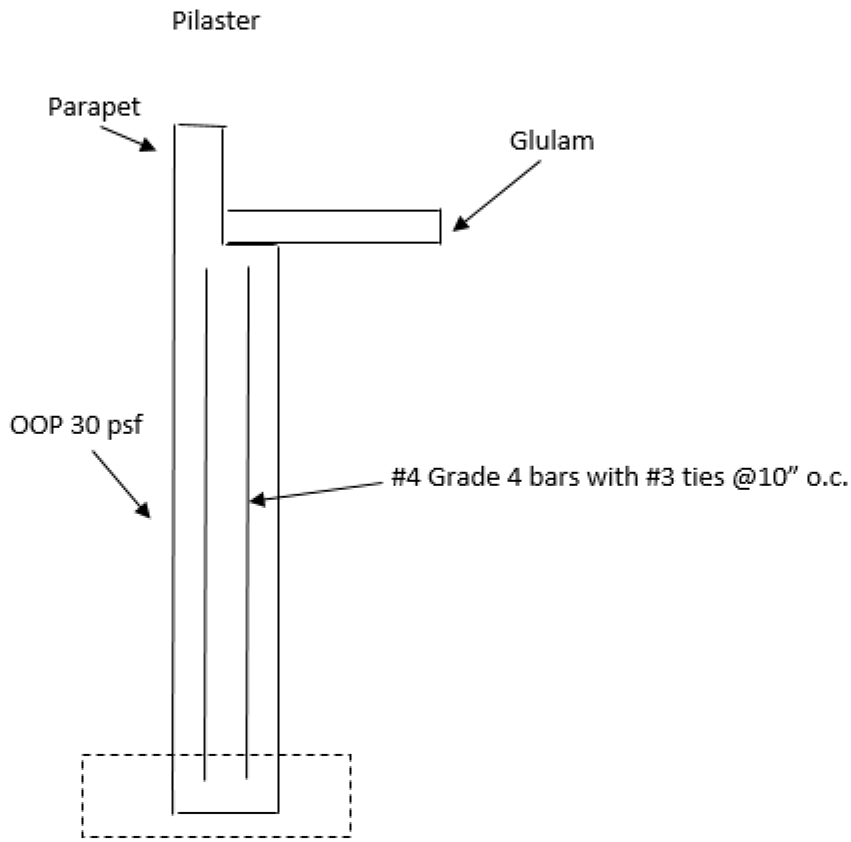
Second Floor Plan


A
B
C
D
1
2
3
4
5
16 ft
4 ft
16 ft
18 ft
18 ft
18 ft
20 ft
Stud Walls Below
AHU 5'x12' 6000 lb.
AHU 5'x12' 6000 lb.
Roof Ledger 4x12 @ 5 ft o.c.
Roof Header 4x12
Roof Joist #1 2x8 @ 16" o.c.
Roof Joist #2 2x4 @ 12" o.c.
Beam (Glulam) 24F-V4
CMU Wall Below
Roof Joist #3 2x10 @ 16" o.c.
Roof Plan


2x8 Joist
4x12
Ledger


2x8 Joist
Top
Plate
2x6 Stud


Bearing Wall
Parapet
42.5"
4x12
Ledger
20 ft
#6 bars
@ 24" o.c.
Elevation
Section


Pilaster
Parapet
Glulam
OOP 30 psf
#4 Grade 4 bars with #3 ties @10” o.c.

Section

Appendix

The Following are all the calculations of the project:

Roof Joist #1:

Span = 16 ft. We use DF # 1 2x8 Ceiling Joist @ 16" o.c. span up to 16'-6"

Section Properties: A = 10.88 in^2 S_x = 13.14 in^3 I_x = 47.63 in^4

1. ASD Flexure Check:

$$w = DL + LL\left(\frac{o.c.}{12\frac{in}{ft}}\right) = 20 + 20\left(\frac{16}{12\frac{in}{ft}}\right) = 46.7\,plf$$

$$\mathrm{M} = \frac{wL^2}{8} = \frac{(46.7\,plf)(16\,ft)^2(12\frac{in}{ft})}{8} = 17920\,lb - ft$$

$$\mathrm{fb} = \frac{M}{Sx} = \frac{17920\,lb - in}{13.14\,in^3} = 1363.77\,psi$$

$$\mathrm{Fb'} = \mathrm{fb} * \mathrm{CD} * \mathrm{CL} * \mathrm{CF} * \mathrm{Cfu} * \mathrm{Cr} = (1363.77\ \mathrm{psi})(1.25)(1)(1.2)(1.15)(1.15) = 2705.39\ \mathrm{psi}$$

$$\mathrm{fb} \le Fb' \text{ O.K.}$$

2. ASD Shear Stress:

$$\mathrm{V} = \frac{1}{2}wL = \left(\frac{1}{2}\right)(46.7\,plf)(16\,ft) = 374\,lb$$

$$\mathrm{fv} = (1.5)\left(\frac{V}{A}\right) = (1.5)\left(\frac{374\,lb}{10.88\,in^2}\right) = 51.51\,psi$$

$$\mathrm{Fv'} = \mathrm{fv} * \mathrm{CD} = (180)(1.25) = 225\ \mathrm{psi}$$

$$\mathrm{fv} \le Fv' \text{ O.K.}$$

We Can Use 2x8

3. Deflection:

$$\Delta T = \frac{5wL^4}{384EI} = \frac{(5)(46.7\,plf)(16\,ft)[(16ft)\left(12\frac{in}{ft}\right)^3]}{(384)(1{,}700{,}000)(47.63\,in^4)} = 0.0033\,inch$$

$$\Delta Tallow = \frac{L}{180} = \frac{(16\,ft)(12\frac{in}{ft})}{180} = 1.07\,inch$$

$\Delta_{Tallow} \geq \Delta T$ O.K.

$$\Delta L = (\Delta T)\left(\frac{wL}{wT}\right) = (0.0033\,inch)\left(\frac{20\,psf}{40\,psf}\right) = 0.0017\,inch$$

$$\Delta Lallow = \frac{L}{240} = \frac{(16\,ft)(12\frac{in}{ft})}{240} = 0.8\,inch$$

$\Delta_{Lallow} \geq \Delta L$ O.K.

Roof Joist #2:

Span = 8 ft. We use DF # 1 2x4 Ceiling Joist @ 12” o.c. span up to 8’-9”

Section Properties: A = 5.25 in^2 S_x = 3.063 in^3 I_x = 5.359 in^4

1. ASD Flexure Check:

$$w = DL + LL\left(\frac{o.c.}{12\frac{in}{ft}}\right) = 20 + 20\left(\frac{12}{12\frac{in}{ft}}\right) = 40\,plf$$

$$\mathrm{M} = \frac{wL^2}{8} = \frac{(40\,plf)(8\,ft)^2(12\frac{in}{ft})}{8} = 3840\,lb - in$$

$$\mathrm{fb} = \frac{M}{Sx} = \frac{3840\,lb - in}{3.063\,in^3} = 1253.67\,psi$$

$$\mathrm{Fb'} = \mathrm{fb} * \mathrm{CD} * \mathrm{CL} * \mathrm{CF} * \mathrm{Cfu} * \mathrm{Cr} = (1253.67\ \mathrm{psi})(1.25)(1)(1.5)(1.15)(1.1) = 2973.56\ \mathrm{psi}$$

$\mathrm{fb} \leq Fb'$ O.K.

2. ASD Shear Stress:

$$\mathrm{V} = \frac{1}{2}wL = \left(\frac{1}{2}\right)(40\,plf)(8\,ft) = 160\,lb$$

$$\mathrm{fv} = (1.5)\left(\frac{V}{A}\right) = (1.5)\left(\frac{160\,lb}{5.25\,in^2}\right) = 45.71\,psi$$

$$\mathrm{Fv'} = \mathrm{fv} * \mathrm{CD} = (180)(1.25) = 225\ \mathrm{psi}$$

$\mathrm{fv} \leq Fv'$ O.K.

We Can Use 2x4

3. Deflection:

$$\Delta T = \frac{5wL^4}{384EI} = \frac{(5)(40\ plf)(8\ ft)[(8ft)\left(12\frac{in}{ft}\right)^3]}{(384)(1{,}700{,}000)(5.359\ in^4)} = 0.4\ inch$$

$$\Delta Tallow = \frac{L}{180} = \frac{(8\ ft)(12\frac{in}{ft})}{180} = 0.53\ inch$$

$\Delta_{Tallow} \geq \Delta T$ O.K.

$$\Delta L = (\Delta T)\left(\frac{wL}{wT}\right) = (0.4\ inch)\left(\frac{20\ psf}{40\ psf}\right) = 0.2\ inch$$

$$\Delta Lallow = \frac{L}{240} = \frac{(8\ ft)(12\frac{in}{ft})}{240} = 0.4\ inch$$

$\Delta_{Lallow} \geq \Delta L$ O.K.

Roof Joist #3:

Span = 20 ft. We use DF # 1 2x10 Ceiling Joist @ 16" o.c. span up to 21'-1"

Section Properties: A = 13.88 in^2 S_x = 21.39 in^3 I_x = 98.93 in^4

1. ASD Flexure Check:

$$w = DL + LL\left(\frac{o.c.}{12\frac{in}{ft}}\right) = 20 + 20\left(\frac{16}{12\frac{in}{ft}}\right) = 53.33\ plf$$

$$\mathrm{M} = \frac{wL^2}{8} = \frac{(53.33\ plf)(20\ ft)^2(12\frac{in}{ft})}{8} = 31998\ lb - in$$

$$\mathrm{fb} = \frac{M}{Sx} = \frac{31998\ lb - in}{21.39\ in^3} = 1495.93\ psi$$

$$\mathrm{Fb'} = \mathrm{fb} * \mathrm{CD} * \mathrm{CL} * \mathrm{CF} * \mathrm{Cfu} * \mathrm{Cr} = (1495.93\ \mathrm{psi})(1.25)(1)(1.1)(1.15)(1.2) = 2838.53\ \mathrm{psi}$$

$\mathrm{fb} \leq Fb'$ O.K.

2. ASD Shear Stress:

$$\mathrm{V} = \frac{1}{2}wL = \left(\frac{1}{2}\right)(53.33\ plf)(20\ ft) = 533.3\ lb$$

$$\mathrm{fv} = (1.5)\left(\frac{V}{A}\right) = (1.5)\left(\frac{533.3\ lb}{13.88\ in^2}\right) = 57.63\ psi$$

$$\mathrm{Fv'} = \mathrm{fv} * \mathrm{CD} = (180)(1.25) = 225\ \mathrm{psi}$$

$\mathrm{fv} \leq Fv'$ O.K.

We Can Use 2x10

3. Deflection:

$$\Delta T = \frac{5wL^4}{384EI} = \frac{(5)(53.33\,plf)(20\,ft)[(20ft)\left(12\frac{in}{ft}\right)^3]}{(384)(1{,}700{,}000)(98.93\,in^4)} = 1.14\,inch$$

$$\Delta Tallow = \frac{L}{180} = \frac{(20\,ft)(12\frac{in}{ft})}{180} = 1.33\,inch$$

$\Delta_{Tallow} \geq \Delta T$ O.K.

$$\Delta L = (\Delta T)\left(\frac{wL}{wT}\right) = (1.14\,inch)\left(\frac{20\,psf}{53.33\,psf}\right) = 0.427\,inch$$

$$\Delta Lallow = \frac{L}{240} = \frac{(20\,ft)(12\frac{in}{ft})}{240} = 1\,inch$$

$\Delta_{Lallow} \geq \Delta L$ O.K.

Floor Joist #1:

Span = 16 ft. We use DF # 1 2x12 Floor Joist @ 12" o.c. span up to 18'-0"

Section Properties: A = 16.88 in^2 S_x = 31.64 in^3 I_x = 178 in^4

1. ASD Flexure Check:

$$w = DL + LL\left(\frac{o.c.}{12\frac{in}{ft}}\right) = 50 + 60\left(\frac{12}{12\frac{in}{ft}}\right) = 110\,plf$$

$$\mathrm{M} = \frac{wL^2}{8} = \frac{(110\,plf)(16\,ft)^2(12\frac{in}{ft})}{8} = 42240\,lb - in$$

$$\mathrm{fb} = \frac{M}{Sx} = \frac{42240\,lb - in}{31.64\,in^3} = 1335\,psi$$

$$\mathrm{Fb'} = \mathrm{fb} * \mathrm{CD} * \mathrm{CL} * \mathrm{CF} * \mathrm{Cfu} * \mathrm{Cr} = (1335\ \mathrm{psi})(1)(1)(1)(1.15)(1.2) = 1842.3\ \mathrm{psi}$$

$\mathrm{fb} \leq Fb'$ O.K.

2. ASD Shear Stress:

$$\mathrm{V} = \frac{1}{2}wL = \left(\frac{1}{2}\right)(110\,plf)(16\,ft) = 880\,lb$$

$$\mathrm{fv} = (1.5)\left(\frac{V}{A}\right) = (1.5)\left(\frac{880\,lb}{16.88\,in^2}\right) = 78.19\,psi$$

$$\mathrm{Fv'} = \mathrm{fv} * \mathrm{CD} = (180)(1) = 180\ \mathrm{psi}$$

$\mathrm{fv} \leq Fv'$ O.K.

We Can Use 2x12

3. Deflection:

$$\Delta\text{T} = \frac{5wL^4}{384EI} = \frac{(5)(110\ plf)(16\ ft)[(16ft)\left(12\frac{in}{ft}\right)^3]}{(384)(1{,}700{,}000)(178\ in^4)} = 0.536\ inch$$

$$\Delta\text{Tallow} = \frac{L}{180} = \frac{(16\ ft)(12\frac{in}{ft})}{180} = 1.067\ inch$$

$\Delta_{Tallow} \geq \Delta T$ O.K.

$$\Delta L = (\Delta T)\left(\frac{wL}{wT}\right) = (0.536\ inch)\left(\frac{50\ psf}{110\ psf}\right) = 0.244\ inch$$

$$\Delta\text{Lallow} = \frac{L}{240} = \frac{(16\ ft)(12\frac{in}{ft})}{240} = 0.8\ inch$$

$\Delta_{Lallow} \geq \Delta L$ O.K.

Cantilever Joist #2:

Span = 20 ft. We use DF # 1 2x14 Floor Joist @ 12" o.c. span up to 20'-1"

Section Properties: A = 19.88 in^2 S_x = 43.89 in^3 I_x = 290.8 in^4

1. ASD Flexure Check:

$$w = DL + LL\left(\frac{o.c.}{12\frac{in}{ft}}\right) = 50 + 60\left(\frac{12}{12\frac{in}{ft}}\right) = 110\,plf$$

$$\mathrm{M} = \frac{wL^2}{8} = \frac{(110\,plf)(20\,ft)^2(12\frac{in}{ft})}{8} = 66000\,lb - in$$

$$\mathrm{fb} = \frac{M}{Sx} = \frac{66000\,lb - in}{43.89\,in^3} = 1503.76\,psi$$

$$\mathrm{Fb'} = \mathrm{fb} * \mathrm{CD} * \mathrm{CL} * \mathrm{CF} * \mathrm{Cfu} * \mathrm{Cr} = (1503.76\text{ psi})(1)(0.9)(1)(1.15)(1.2) = 1867.67\text{ psi}$$

$\mathrm{fb} \le Fb'$ O.K.

2. ASD Shear Stress:

$$\mathrm{V} = \frac{1}{2}wL = \left(\frac{1}{2}\right)(110\,plf)(20\,ft) = 1100\,lb$$

$$\mathrm{fv} = (1.5)\left(\frac{V}{A}\right) = (1.5)\left(\frac{1100\,lb}{19.88\,in^2}\right) = 82.99\,psi$$

$$\mathrm{Fv'} = \mathrm{fv} * \mathrm{CD} = (180)(1) = 180\text{ psi}$$

$\mathrm{fv} \le Fv'$ O.K.

We Can Use 2x14

3. Deflection:

$$\Delta T = \frac{5wL^4}{384EI} = \frac{(5)(110\,plf)(20\,ft)[(20ft)\left(12\frac{in}{ft}\right)^3]}{(384)(1{,}700{,}000)(290.8\,in^4)} = 0.80\,inch$$

$$\Delta \text{Tallow} = \frac{L}{180} = \frac{(20\,ft)(12\frac{in}{ft})}{180} = 1.33\,inch$$

$\Delta_{Tallow} \geq \Delta T$ O.K.

$$\Delta L = (\Delta T)\left(\frac{wL}{wT}\right) = (0.8\,inch)\left(\frac{50\,psf}{110\,psf}\right) = 0.36\,inch$$

$$\Delta \text{Lallow} = \frac{L}{240} = \frac{(20\,ft)(12\frac{in}{ft})}{240} = 1\,inch$$

$\Delta_{Lallow} \geq \Delta L$ O.K.

Roof Beam (Glulam):

Span = 18 ft. DF grade 24F-V4 L_r = 20 psf D_L = 20 psf

$$Tributary\ width\ =\ 4\,ft \qquad w\ =\ (20\,psf\ +\ 20\,psf)(4\,ft)\ =\ 160\,lb/ft$$

$$\mathrm{M} = \frac{1}{8}wL^2 = \left(160\frac{lb}{ft}\right)\left(\frac{(18\,ft)^2}{8}\right) = 8000\,lb - ft$$

$$\mathrm{F'b} = (Fb)(CD)(Cv) = (2400\,psi)(1.25)(0.8) = 2400psi$$

$$\mathrm{F'b} \geq \mathrm{fb} = \frac{M}{Sx} \quad \longrightarrow\ Sx\ \geq \frac{M}{F'b} = \frac{(8000\,lb - ft)\left(12\frac{in}{ft}\right)}{2400\,psi} = 40\,in^3$$

Try 3- 1/8 “x 9”

A = 28.13 in^2 I_x = 189.8 in^4 S_x = 42.19 in^3

$$\mathrm{Cv} = \left(\frac{1292}{(L)(A)}\right)^{0.1} = \left(\frac{1292}{(18\,ft)(28.13\,in^2)}\right)^{0.1} = 1.098 > 0.8$$

Flexure is OK

$$\Delta L = \left(\frac{5wL^4}{(384)(E')(Ix)}\right) = \left(\frac{(5)(20\,psf)(4ft)(18\,ft)^4\left(12\frac{in}{ft}\right)^3}{(384)(1.8x10^6\,psi)(189.8\,in^4)}\right) = 0.55\,inch$$

$$\Delta Lallow\ = \left(\frac{L}{180}\right) = \left(\frac{(18\,ft)(12\frac{in}{ft})}{180}\right) = 1.2\,inch$$

ΔL < ΔLallow O.K.

$$fv\ = \left(\frac{1.5V}{A}\right) = \left(\frac{(1.5)(\frac{1}{2})(160)(18)}{28.13}\right) = 76.79\,psi$$

$$F'v\ =\ (Fv)(CD) = (265\,psi)(1.25) = 331\,psi$$

F’v > fv O.K.

Second Floor Beam (Glulam):

Span = 18 ft. DF grade 24F-V4 L_L = 50 psf D_L = 60 psf

$Tributary\ width\ =\ 10\ ft$ $\qquad w\ =\ (50\ psf\ +\ 60\ psf)(10\ ft)\ =\ 1100\ lb/ft$

$$\mathrm{M} = \frac{1}{8}wL^2 = \left(1100\frac{lb}{ft}\right)\left(\frac{(18\ ft)^2}{8}\right) = 44550\ lb - ft$$

$$\mathrm{F'b} = (Fb)(CD)(Cv) = (2400\ psi)(1)(0.8) = 1920psi$$

$$\mathrm{F'b} \geq \mathrm{fb} = \frac{M}{Sx} \quad \dashrightarrow\ Sx\ \geq \frac{M}{F'b} = \frac{(44550\ lb - ft)\left(12\frac{in}{ft}\right)}{1920\ psi} = 278.44\ in^3$$

Try 5- 1/8 “x 19 – 1/2”

A = 99.94 in^2 $\quad I_x$ = 3167 in^4 $\quad S_x$ = 324.8 in^3

$$\mathrm{Cv} = \left(\frac{1292}{(L)(A)}\right)^{0.1} = \left(\frac{1292}{(18\ ft)(99.94\ in^2)}\right)^{0.1} = 0.97 > 0.8$$

Flexure is OK

$$\Delta L = \left(\frac{5wL^4}{(384)(E')(Ix)}\right) = \left(\frac{(5)(50\ psf)(10ft)(18\ ft)^4\left(12\frac{in}{ft}\right)^3}{(384)(1.8x10^6\ psi)(3167\ in^4)}\right) = 0.21\ inch$$

$$\Delta Lallow = \left(\frac{L}{180}\right) = \left(\frac{(18\ ft)(12\frac{in}{ft})}{180}\right) = 1.2\ inch$$

ΔL < ΔLallow O.K.

$$fv = \left(\frac{1.5V}{A}\right) = \left(\frac{(1.5)(\frac{1}{2})(1100)(18)}{99.94}\right) = 148.59\ psi$$

$$F'v =\ (Fv)(CD) = (265\ psi)(1.25) = 331\ psi$$

F’v > fv O.K.

2^{nd} Floor Interior Bearing Stud Walls (Supporting Roof and Ceiling):

2x6 @ 24" DF # 1 L_e = 10 ft D_L = 20 psf L_L = 50 psf L_r = 20 psf

d = 5.5 A = 8.25 in^2 S_x = 7.563 in^3 I_x = 20.80 in^4

Bending Stress on a stud:

$$M = \frac{1}{8}wL^2 = \left(\frac{1}{8}\right)(D+L)\left(\frac{24"}{12}\right)(10\,ft)^2 = 1000\,lb\,ft$$

$$\mathrm{fb} = \frac{M}{Sx} = \frac{(1000)(12\frac{in}{ft})}{7.563\,in^3} = 1586.67\,psi$$

$$\mathrm{Fb' = fb * CD * CF * Cr = (1000)(1)(1.3)(1.35) = 1755\ psi}$$

Axial Compressive Stress on a stud:

$$\mathrm{fc} = \frac{\mathrm{P}}{\mathrm{A}} = \frac{(40)\left(\frac{24}{12}\right)}{8.25\,in^2} = 9.69\,psi$$

$$\frac{\mathrm{Le}}{\mathrm{d}} = \frac{(10\,ft)(12\frac{in}{ft})}{5.5} = 22 < 50\ O.K.$$

$$\mathrm{Fc *= Fc * CD * CF = (1500)(1)(1.1) = 1650\ psi}$$

$$\mathrm{Fce} = \left(\frac{(0.822)(E'min)}{(\frac{Le}{d})^2}\right) = \frac{(0.822)(620{,}000\,psi)}{(22)^2} = 1052.98$$

$$\frac{Fce}{FC*} = \left(\frac{1052.98}{1650}\right) = 0.64 \ \ --\rightarrow Cp = 0.524$$

$$F'c = (FC*)(Cp) = (1650)(0.524) = 864.6\,psi$$

$$\left(\frac{fc}{F'c}\right)^2 + \frac{fb}{F'b[1-\left(\frac{fc}{Fce}\right)]} = \left(\frac{9.69}{864.6}\right)^2 + \frac{1586.67}{1755[1-\left(\frac{9.69}{1052.98}\right)]} = 0.91$$

0.91 < 1 O.K.

Ground Floor Interior Bearing Stud Walls (Supporting Roof, Ceiling and 2nd floor):

2x12 @ 24"   DF # 1 L_e = 10 ft D_L = 60 psf L_L = 50 psf L_r = 20 psf

d = 11.25   A = 16.88 in^2 S_x = 31.64 in^3 I_x = 178 in^4

Bending Stress on a stud:

$$\mathrm{M} = \frac{1}{8}wL^2 = \left(\frac{1}{8}\right)(D+L)\left(\frac{24"}{12}\right)(10\ ft)^2 = 2750\ lb\ ft$$

$$\mathrm{fb} = \frac{M}{Sx} = \frac{(2750)(12\frac{in}{ft})}{31.64\ in^3} = 1042.98\ psi$$

$$\mathrm{Fb}' = \mathrm{fb} * \mathrm{CD} * \mathrm{CF} * \mathrm{Cr} = (1000)(1)(1)(1.15) = 1150\ \mathrm{psi}$$

Axial Compressive Stress on a stud:

$$\mathrm{fc} = \frac{\mathrm{P}}{\mathrm{A}} = \frac{(110)\left(\frac{24}{12}\right)}{16.88\ in^2} = 13\ psi$$

$$\frac{\mathrm{Le}}{\mathrm{d}} = \frac{(10\ ft)(12\frac{in}{ft})}{11.25} = 11 < 50\ O.K.$$

$$\mathrm{Fc} *= \mathrm{Fc} * \mathrm{CD} * \mathrm{CF} = (1500)(1)(1) = 1500\ \mathrm{psi}$$

$$\mathrm{Fce} = \left(\frac{(0.822)(E'min)}{(\frac{Le}{d})^2}\right) = \frac{(0.822)(620{,}000\ psi)}{(11)^2} = 4211.9$$

$$\frac{Fce}{FC*} = \left(\frac{4211.9}{1500}\right) = 2.8 \ \dashrightarrow Cp = 0.91$$

$$F'c = (FC*)(Cp) = (1500)(0.91) = 1365\ psi$$

$$\left(\frac{fc}{F'c}\right)^2 + \frac{fb}{F'b[1-\left(\frac{fc}{Fce}\right)]} = \left(\frac{13}{1365}\right)^2 + \frac{1042.98}{1150[1-\left(\frac{13}{4211.9}\right)]} = 0.91$$

0.91 < 1   O.K.

Check the walls under the Air Handling Unit (AHU):

<u>Using 4x12:</u>

Load from the roof and units = 320 plf

$$\mathrm{M} = \frac{1}{8}wL^2 = \left(\frac{1}{8}\right)(D+L)(5\,ft)^2 = 1000\,lb\,ft$$

$$\mathrm{fb} = \frac{M}{Sx} = \frac{(1000)(12\frac{in}{ft})}{73.83\,in^3} = 163.53\,psi$$

$$\mathrm{Fb'} = \mathrm{fb} * \mathrm{CD} * \mathrm{CF} = (1000)(1.25)(1.1) = 1375\text{ psi} - -> (1375)(2) = 2750$$

$$\mathrm{V} = \frac{1}{2}wL = \left(\frac{1}{2}\right)(320\,plf)(12\,ft) = 1920\,lb$$

$$\mathrm{fv} = (1.5)\left(\frac{V}{A}\right) = (1.5)\left(\frac{lb}{39.38\,in^2}\right) = 73.13\,psi \;<\; (2)(F'v) = 450\,psi$$

$$\mathrm{Fv'} = \mathrm{fv} * \mathrm{CD} = (180)(1.25) = 225\text{ psi}$$

Roof Header Design:

4x12 DF # 1 L = 8 ft D_L = 20 psf = 160 plf L_L = 50 psf L_r = 20 psf = 160 plf

A = 39.38 in^2 S_x = 73.83 in^3 I = 415.3 in^4

Bending Stress on a stud:

$$\mathrm{M} = \frac{1}{8}wL^2 = \left(\frac{1}{8}\right)(D+L)(8\,ft)^2 = 2560\,lb\,ft$$

$$\mathrm{fb} = \frac{M}{Sx} = \frac{(2560)(12\frac{in}{ft})}{73.83\,in^3} = 416\,psi$$

$$\mathrm{Fb'} = \mathrm{fb} * \mathrm{CD} * \mathrm{CF} = (1000)(1.25)(1.1) = 1375\ \mathrm{psi}$$

$$\mathrm{V} = \frac{1}{2}wL = \left(\frac{1}{2}\right)(320\,plf)(8\,ft) = 1280\,lb$$

$$\mathrm{fv} = (1.5)\left(\frac{V}{A}\right) = (1.5)\left(\frac{1280\,lb}{39.38\,in^2}\right) = 48.76\,psi$$

$$\mathrm{Fv'} = \mathrm{fv} * \mathrm{CD} = (180)(1.25) = 225\ \mathrm{psi}$$

$\mathrm{fv} \leq Fv'$ O.K.

$$\Delta\mathrm{T} = \frac{5wL^4}{384EI} = \frac{(5)(320\,plf)(8\,ft)[(8ft)\left(12\frac{in}{ft}\right)^3]}{(384)(1{,}700{,}000)(415.3\,in^4)} = 0.00065\,inch \quad < \quad \frac{L}{240} = 0.033$$

2^{nd} Floor Header Design:

4x12 DF # 1 L = 8 ft D_L = 60 psf = 480 plf L_L = 50 psf = 400 plf

A = 39.38 in^2 S_x = 73.83 in^3 I = 415.3 in^4

Bending Stress on a stud:

$$\text{M} = \frac{1}{8}wL^2 = \left(\frac{1}{8}\right)(D + L)(8\,ft)^2 = 7040\,lb\,ft$$

$$\text{fb} = \frac{M}{Sx} = \frac{(7040)(12\frac{in}{ft})}{73.83\,in^3} = 1144\,psi$$

$$\text{Fb}' = \text{fb} * \text{CD} * \text{CF} = (1000)(1.25)(1.1) = 1375\text{ psi}$$

$$\text{V} = \frac{1}{2}wL = \left(\frac{1}{2}\right)(880\,plf)(8\,ft) = 3520\,lb$$

$$\text{fv} = (1.5)\left(\frac{V}{A}\right) = (1.5)\left(\frac{3520\,lb}{39.38\,in^2}\right) = 134\,psi$$

$$\text{Fv}' = \text{fv} * \text{CD} = (180)(1.25) = 225\text{ psi}$$

$\text{fv} \leq Fv'$ O.K.

$$\Delta\text{T} = \frac{5wL^4}{384EI} = \frac{(5)(880\,plf)(8\,ft)[(8ft)\left(12\frac{in}{ft}\right)^3]}{(384)(1{,}700{,}000)(415.3\,in^4)} = 0.0018\,inch \quad < \quad \frac{L}{240} = 0.033$$

Timber Columns:

Height = 20 ft DL = 20 psf Lr = 20 psf DF # 1 grade

$$\text{Column Tributary Area} = \left(\frac{1}{2}\right)(8\,ft)\left(\frac{1}{2}\right)(18\,ft) = 36\,ft^2$$

Live Load Reduction:

R_1 = 1 due to $A_T \leq 200$ ft^2

R_2 = 1 due to $F \leq 4$

$$\text{Lr} = \text{Lo} * \text{R1} * \text{R2} = (20\,psf)(1)(1) = 20psf$$

$$E'min = 580{,}000$$

$$Fc = 1{,}000 \text{ psi}$$

$$E'min = Emin {-\!\!\longrightarrow} \text{Kf} = 1.76 \text{ and } \phi = 0.85$$

$$F'c = Fc {-\!\!\longrightarrow} \text{Kf} = 2.40 \text{ and } \phi = 0.90$$

$$\lambda = 0.8$$

$$\text{Pu} = (1.2\text{ D} + 1.6\,Lr)(AT) = [(1.2)(20) + (1.6)(20)](36\,ft^2) = 2016\,k$$

$$\text{F}'\text{c} = \text{Fc} * \text{KF} * \phi * \lambda = (1000)(2.40)(0.90)(0.8) = 1728\,psi$$

$$\text{E}'\text{min} = \text{Emin} * \text{KF} * \phi = (580000)(1.76)(0.85) = 867680\,psi$$

Strength Check (LRFD)

Try 6x6 A = 30.25 in^2 d = 5.5"

$$\frac{le}{d} = \left(\frac{(20ft)(12\frac{in}{ft})}{5.5\,in}\right) = 43.63 \;\leq 50 \quad O.K. \;\; Slenderness\,ratio$$

$$\text{Fce} = \left(\frac{(0.822)(867680)}{(43.63)^2}\right) = 374.68\,psi$$

$$\frac{Fce}{FC*} = \left(\frac{374.68}{1728}\right) = 0.2168 = 0.22 {-\!\!\longrightarrow} Cp = 0.208 = 0.21$$

$$\text{Pn}' = (\text{FC}*)(\text{Cp})(\text{A}) = (1728)(0.21)(30.25) = 10977.12\text{ k}$$

$P_u \leq P'_n$ O.K.

We Can Use 6x6

Roof Ledger:

Using a 4x12 @ 5 ft o.c. Lr= 20 psf DL=20 psf

Flexure Check:

$$\text{fb} = \frac{M}{Sx} = \frac{\left(\frac{1}{8}\right)(40\ psf)(5\ ft)(18)^2(12\frac{in}{ft})}{73.83\ in^3} = \ 1316.5\ \ psi$$

$$\text{Fb}' = \text{fb} * \text{CD} * \text{CF} * \text{Cr} = (1000)(1.25)(1.1)(1) = 1375\ \text{psi}$$

Shear Check:

$$\text{fv} = (1.5)\left(\frac{V}{A}\right) = (1.5)\left(\frac{\left(\frac{1}{2}\right)(40\ psf)(18\ ft)(5\ ft)}{39.38\ in^2}\right) = 68.6\ psi$$

$$\text{Fv}' = \text{fv} * \text{CD} = (180)(1.25) = 225\ psi > fv\ \ O.K.$$

Deflection Check:

$$\Delta Lallow = \frac{\text{L}}{180} = \frac{(18)(12\frac{in}{ft})}{180} = 1.2" \qquad \Delta TLallow = \frac{\text{L}}{120} = \frac{(18)(12\frac{in}{ft})}{120} = 1.8"$$

$$\Delta L = \frac{5\text{WL}^4}{384\text{EI}} = \frac{(5)(20\ psf)(5ft)(18\ ft)^4\left(12\frac{in}{ft}\right)^3}{(384)(1700000)(415.3\ in^4)} = 0.33" \ < \ \Delta Lallow$$

$$\Delta TL = \frac{5\text{WL}^4}{384\text{EI}} = \frac{(5)(40\ psf)(5ft)(18\ ft)^4\left(12\frac{in}{ft}\right)^3}{(384)(1700000)(415.3\ in^4)} = 0.67" \ < \ \Delta TLallow$$

Floor Ledger:

Using a 4x12 @ 1.5 ft o.c. LL= 50 psf   DL=60 psf

Flexure Check:

$$\text{fb} = \frac{M}{Sx} = \frac{\left(\frac{1}{8}\right)(110\,psf)(1.5\,ft)(18\,ft)^2(12\frac{in}{ft})}{73.83\,in^3} = 1086\ psi$$

$$\text{Fb}' = \text{fb} * \text{CD} * \text{CF} * \text{Cr} = (1000)(1.15)(1)(1) = 1150\text{ psi}$$

Shear Check:

$$\text{fv} = (1.5)\left(\frac{V}{A}\right) = (1.5)\left(\frac{\left(\frac{1}{2}\right)(110\,psf)(18\,ft)(1.5\,ft)}{39.38\,in^2}\right) = 56.6\,psi$$

$$\text{Fv}' = \text{fv} * \text{CD} = (180)(1) = 180\,psi > fv\ \ O.K.$$

Deflection Check:

$$\Delta Lallow = \frac{\text{L}}{180} = \frac{(18)(12\frac{in}{ft})}{180} = 1.2" \qquad \Delta TLallow = \frac{\text{L}}{120} = \frac{(18)(12\frac{in}{ft})}{120} = 1.8"$$

$$\Delta L = \frac{5\text{WL}^4}{384\text{EI}} = \frac{(5)(50\,psf)(1.5ft)(18\,ft)^4\left(12\frac{in}{ft}\right)^3}{(384)(1700000)(415.3\,in^4)} = 0.25" < \Delta Lallow$$

$$\Delta TL = \frac{5\text{WL}^4}{384\text{EI}} = \frac{(5)(110\,psf)(1.5ft)(18\,ft)^4\left(12\frac{in}{ft}\right)^3}{(384)(1700000)(415.3\,in^4)} = 0.55" < \Delta TLallow$$

Masonry CMU Parapets:

CMU Wall Thickness – 10” (9.625’ actual)   Wind load = 30 psf

Parapet Height: 42” +1” = 3.58 ft   Fm’: 2000 psi   Fy: 60 ksi

#5 grade bars @ 24” o.c.

$$\mathrm{A} = 0.31\ \mathrm{in}^2 \qquad \mathrm{b} = 24" \qquad \mathrm{d} = \left(\frac{h}{2}\right) = 4.8125"$$

$$\mathrm{a} = \left(\frac{As\, fb}{0.8\, f'm\ \ b}\right) = \left(\frac{(0.31\ in^2)(60\ ksi)}{(0.8)(2\ ksi)(24")}\right) = 0.48"$$

$$\phi \mathrm{Mn} = \phi\ As\ fy\left(d - \frac{a}{2}\right) = (0.9)(0.31in^2)(60\ ksi)(4.8125" - \frac{0.48"}{2}) = 76.5\ k - in$$

$$Mu = (30\ psf)(3.58\ ft)(24")(\frac{3.58\ ft}{2}) = 4.61\ k - in < \ \phi Mn\ \ O.K.$$

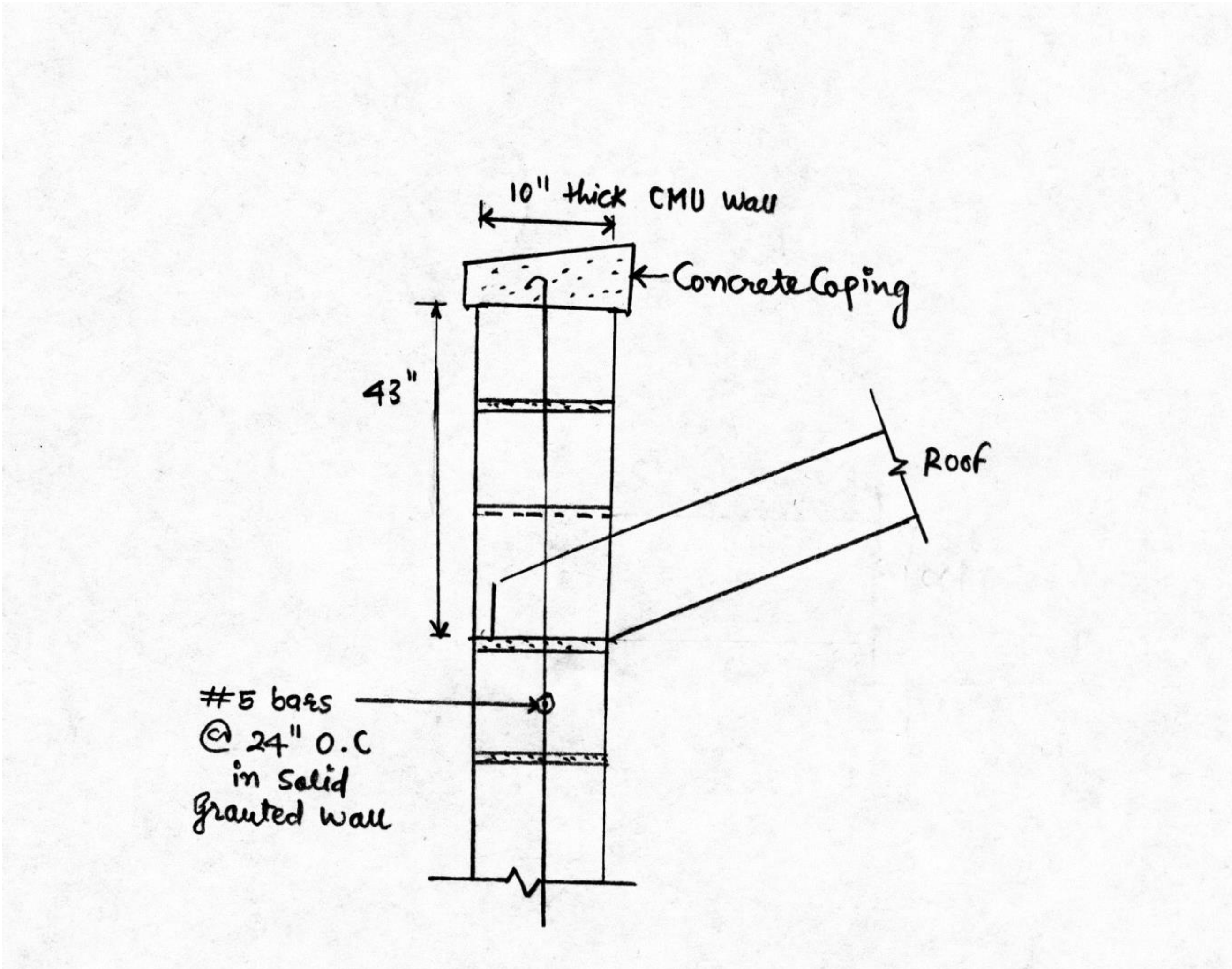
Max Reinforcement:

$$\frac{a}{d} = \frac{0.48"}{4.8125"} = 0.049 < 0.356\ O.K.$$

Minimum Reinforcement:

$$\phi 1.3Mcr = 1.17\ \ Fr\ \ b\ \ \frac{h^2}{6} = (1.17)(163\ psi)(24")(\frac{(9.625")^{\wedge}2}{6}) = 70.6\ k - in < \ \phi Mn\ \ \ O.K.$$

Provide #5 grade bars @ 24” o.c.



CMU Parapet wall

Masonry CMU Wall thickness:

F'm = 2000 psi fy = 60 ksi Thickness of wall = 10" Dead Load = 95.44 psf Live Load = 20 psf

Total Load:

$$Pu = 1.2D + 1.6L = (1.2)(95.44\,psf)(18ft)(20ft) + (1.6)(20\,psf)(18\,ft)(20\,ft) = 52\,kip$$

$$\text{a} = \left(\frac{Pu + fy\ \ As}{0.8\,f'm\ \ b}\right) = \left(\frac{(52\,kip + 60\,ksi)(0.88\,in^2)}{(0.8)(2\,ksi)(9.626")}\right) = 6.80"$$

$$\phi\text{Mn} = \phi\ \left[As\,fy\left(d - \frac{a}{2}\right) + P\left(\frac{h}{2} - \frac{a}{2}\right)\right]$$

$$= (0.9)[(60\,ksi)(0.88\,in^2)(216" - \frac{6.80"}{2}) + 52\,kip(120" - \frac{6.80"}{2})] = 1295\,k - ft$$

$$\text{w} = (95.44\text{ psf})(18\text{ ft}) = 1717.92\,lb/ft$$

$$\text{Mu} = \frac{1}{8}wL^2 = \left(\frac{1}{8}\right)\left(1717.92\frac{lb}{ft}\right)(20\,ft)^2 = 85.86\,k - ft$$

$$Vu = \frac{wl}{2} = \frac{\left(1717.92\frac{lb}{ft}\right)(20\,ft)}{2} = 17\,kips$$

$$Mn = \frac{1295}{0.9} = 1438\,k - ft$$

$$Vu = 1.25V\left(\frac{\phi Mn}{Mu}\right) = (1.25)(17)\left(\frac{1438}{85.86}\right) = 355.8\,kips$$

The shear span ratio of wall:

$$\frac{Mu}{Vd} = \frac{343.58}{17(20)} = 0.99$$

$$Vnm = \left[4 - 1.75\left(\frac{mu}{Vu\,dv}\right)\right]Anv\ \sqrt{\text{f'm}} + 0.25\,Pu$$

$$= \ [4 - 1.75(0.99)](353\,in^2)\ \sqrt{2000\text{ psi}} + 0.25\,(52\,kip) = 48.79\,kips$$

Try #6 bars @16" o.c.

$$Vns = \left[\left(\frac{Av\ \ fy\ \ dv}{2\ \ s}\right)\right] = \left(\frac{(0.88\,in^2)(60\,ksi)(20\,ft)\left(12\frac{in}{ft}\right)}{(2)\ (16")}\right) = 396\,kips$$

$$\phi(Vnm + Vns) = (0.8)(48.79 + 396) = 356\ \geq 355.8 \qquad O.K.$$

#6 bars @ 16" o.c. are sufficient to resist the shear demands of the wall

Check Reinforcement:

$\#6 @ 24" \; o.c. horizontally \longrightarrow \; e = \left(\frac{0.44in^2}{16"(9.625")}\right) = 0.0028 > 0.0007 \; O.K.$

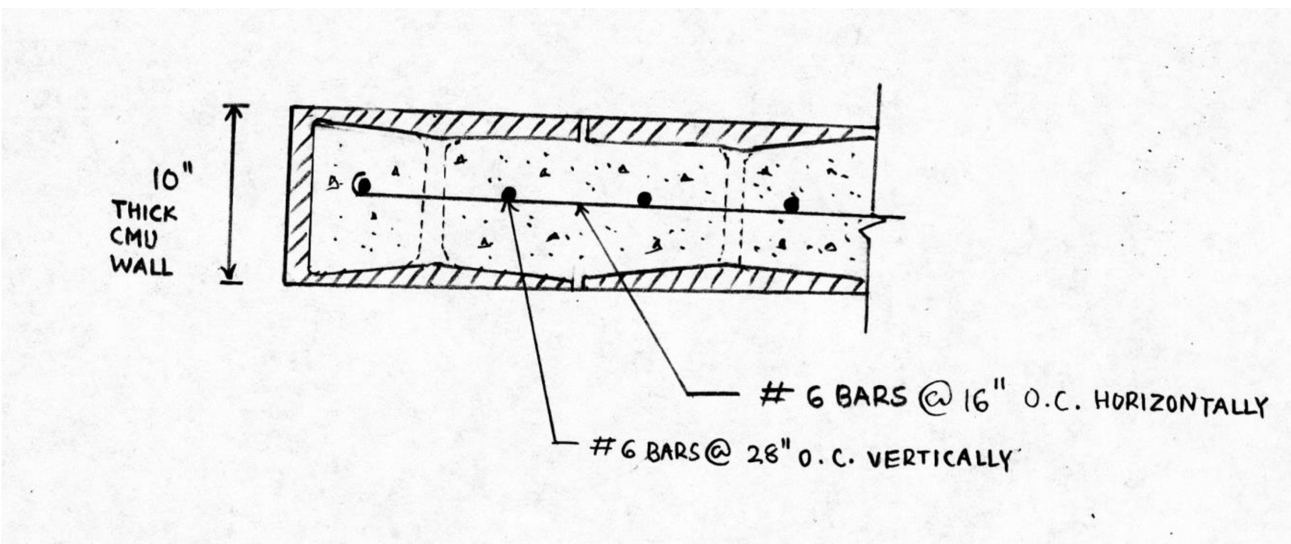
$\#6 @ 28" \; o.c. \; \longrightarrow \; e = \left(\frac{0.44in^2}{28"(9.625")}\right) = 0.0016 > 0.0007 \; O.K.$

$S \; total = s \; horizontal + s \; vertical = \; 0.0028 + 0.0016 = 0.0044 > 0.002 \; O.K.$

Hence Provide:

#6 bars @ 16” o.c. horizontally

#6 bars @ 28” o.c. vertically



CMU 10” thick Wall

Masonry CMU Bearing Walls:

#6 bars @ 24" o.c. A = 0.22 in^2 Height = 20 ft d = 4.81"

$$Puw = Self\ Wieght\ of\ wall = (1.2)(84\ psf)\left(\frac{20'}{2} + \frac{53"}{12"}\right) = 1453.2\ plf \quad at\ base = 2461.2\ plf$$

$$Puf = Roof\ and\ floor\ load\ at\ top\ of\ wall = (1.2)(12\ psf) + (1.6)(20\ psf)(\frac{20\ ft}{2}) = 464\ plf$$

$$Pu,\frac{h}{2} = 1453.2plf + 464\ plf = 1917.2\ plf\ @\ mid\ height = 1.9\ klf \quad at\ base = 2.92\ klf$$

$$\mathrm{Mu} = \frac{1}{8}wL^2 = \left(\frac{1}{8}\right)(30\ psf)\left(20\ ft + \frac{43"}{12"}\right)^2 = 2085\ lb - \frac{ft}{ft}wall$$

$$\mathrm{a} = \left(\frac{Pu + fy\ \ As}{0.8\ f'm\ \ b}\right) = \left(\frac{(1.9\ klf + 60\ ksi)(0.22\ in^2)}{(0.8)(2\ ksi)(12")}\right) = 0.78"$$

$$\phi\mathrm{Mn} = \phi\ \left[As\ fy\left(d - \frac{a}{2}\right) + Pu\left(\frac{h}{2} - \frac{a}{2}\right)\right] = (0.9)[(60\ ksi)(0.22\ in^2) + (\frac{1.6\ klf}{0.9})(4.81" - \frac{0.78"}{2})]$$
$$= 4962.5\ lb - ft/ft\ wall$$

$$\mathrm{M} = \frac{\mathrm{bt}^2}{6}\left(fr + \frac{Pu}{bt}\right) = \frac{(12")(9.625")}{6}\left(163\ psi + \frac{1900\ lb}{(12")(9.625")}\right) = 3454\ lb - ft$$

$$In = \frac{bt^3}{12} = \frac{12"(9.625)^3}{12} = 891.6\ in^4 \qquad Em = (900)(2000\ psi) = \ 1800\ ksi$$

$$\delta u = \frac{5\ \ Mu\ \ H^2}{48\ \ \ Em\ \ \ In} = \frac{(5)(2085\ lb - ft)(20)^2\left(12\frac{in}{ft}\right)^3}{(48)(1800000)(891.6\ in^4)} = 0.10"$$

$$Mu,\delta = 0.10" = \ 2085\ lb - ft\ \ + (1900\ lb\ \ - \ \frac{0.10"}{12\ in/ft}) = 2100\ lb - ft$$

δ and Mu converge 8.71% O.K. Mu < ϕMn

$$\phi\mathrm{Pn} = \phi\ 0.8\ \ [0.8\ \ f'm\ \ An]\left[1 - \left(\frac{hr}{140}\right)^2\right] = (0.72)[0.8(2\ ksi)(9.625")(12")][1 - \left(\frac{86.28}{140}\right)^2] = 82\ klf$$

ϕPn > Pu base O.K.

$$\frac{\delta}{h} = \frac{0.10"}{20\ ft(12\frac{in}{ft})} = 0.0004 < 0.007H \qquad O.K.$$

Development Length:

$$ld = \frac{0.13\ \ db^2\ \ fy\ \ \gamma}{k\ \ \sqrt{f'm}} = \frac{0.13(\ 0.750)^2\ (\ 60\ ksi)\ (1.3)}{3"\ \ \sqrt{2000}} \ = \ 42.5"\ \le\ 72db\ =\ 54"$$

Provide minimum lap length 42.5"

Pier:

$$Load\ from\ roof\ = (1.2)(12\ psf) + (1.6)(20\ psf)\left(\frac{20ft}{2}\right) = 464\ plf$$

$$Self\ Weight\ of\ Wall = \left(\frac{84\ psf}{2}\right)(10' + \frac{43"}{12"}) = 570.5\ plf$$

$$oop\ wind\ load = (30\ psf)(12\ ft) = 360\ plf$$

To check:

Pu < 0.05 As f’m

b ≥ 6t

b ≥ (6)(10) = 60”

$$Mu\ max = \frac{1}{8}(360\ plf)\left(10' + \frac{43"}{12"}\right)^2 (12) = 99.6k - in\ plf$$

$$Pu, \frac{h}{2} = 570.5\ plf + 464\ plf = 12.4\frac{k}{ft}wall$$

$$0.05\ f'm\ \ An = 0.05(2\ ksi)(9.625")(60") = \ 57.75\ k/ft\ wall$$

Hence provide 10” thick wall

$$\phi \text{Mn} = \phi\ As\ fy\left(d - \frac{a}{2}\right) = (0.8)(60\ ksi)(9)(0.20\ in^2)\left(9.625" - \frac{7"}{2}\right) = 113\ k - in$$

$$\text{a} = \left(\frac{As\ fb}{0.8\ f'm\ \ b}\right) = \left(\frac{(9)(0.20in^2)(60\ ksi)}{0.8(2\ ksi)(9.625")}\right) = 7"$$

$$Pu\ \text{max} = (464\ plf) + (570.5)(2) = 19.26\ kip\frac{ft}{wall} < 0.3\ An\ f'm = 23.1\ k\frac{ft}{wall} \quad O.K.$$

Development Length:

$$ld = \frac{0.13\ \ db^2\ \ fy\ \ \gamma}{k\ \ \sqrt{\text{f'm}}} = \frac{0.13(\ 0.5)^2\ (\ 60\ ksi)\ (1)}{3"\ \ \sqrt{2000}} = 14.53 \leq\ 72\ \text{db} = 36 \qquad O.K.$$

Provide 14.53” lap length

Masonry CMU Pilasters:

Provide 16’×20’ pilaster - #4 Grade 4 bars with #3 ties @10” o.c.

Load from Glulam $= \frac{1}{2}$ (1.2*12 psf + 1.6*20psf)18 ft*16ft = 6.68^{k}

Weight of Parapet wall = 1.2*84psf * $\dfrac{(43'' * 18'')}{144\ (\frac{in2}{ft2})}$ = 0.54^{k}

Self-Weight of wall = 1.2*84psf $(\frac{18"}{12"} * 2)$ = 302 lb/ft of wall

Wind load = 30psf *10ft = 300lb/ft of wall

$M_u(x) = P_u * e\,(1-\frac{x}{h}) + \frac{w*x}{2}(h-x) = 6.68^{k}\,(9''/12'') * (1-\frac{x}{20ft}) + \frac{0.54k}{2}(20ft - x)$

$\frac{dMu(x)}{dx} = 0$, When x= 9.53 ft

$P_{u,h=9.53ft} = 6.68^{k} + (302\ lb/ft * 9.53\ ft) + 0.54^{k} = 10.22^{k}$

$P_{u,base} = 13.26^{K}$

<u>Flexure Check:</u>

$$a = \left(\frac{As\,fb}{0.8\,f'm\ b}\right) = \left(\frac{(0.40in^2)(60\ ksi)}{0.8(2\ ksi)(15.625")}\right) = 0.76"$$

$$\phi Mn = \phi\ As\ fy\left(d - \frac{a}{2}\right) = (0.9)(60\ ksi)(0.40\ in^2)\left(7.8125" - \frac{0.76"}{2}\right) = 161.6\ k - in$$

<u>Axial Check:</u>

$$\left(\frac{h}{r}\right) = \frac{20\ ft*12\ in/ft}{0.289*15625"} = 53.14 < 99, \text{O.K.}$$

$$\phi Pn = \phi\ 0.8\ \ [0.8\ \ f'm\ \ (An - Ast) + fy * Ast]\left[1 - \left(\frac{hr}{140}\right)^2\right]$$

$$= (0.72)[0.8(2\ ksi)(306.64\ in2 - 0.8in2) + 60ksi * 0.8in2]\left[1 - \left(\frac{53.14}{140}\right)^2\right] = 332K$$

$\phi Pn > Pu, base\ O.K.$

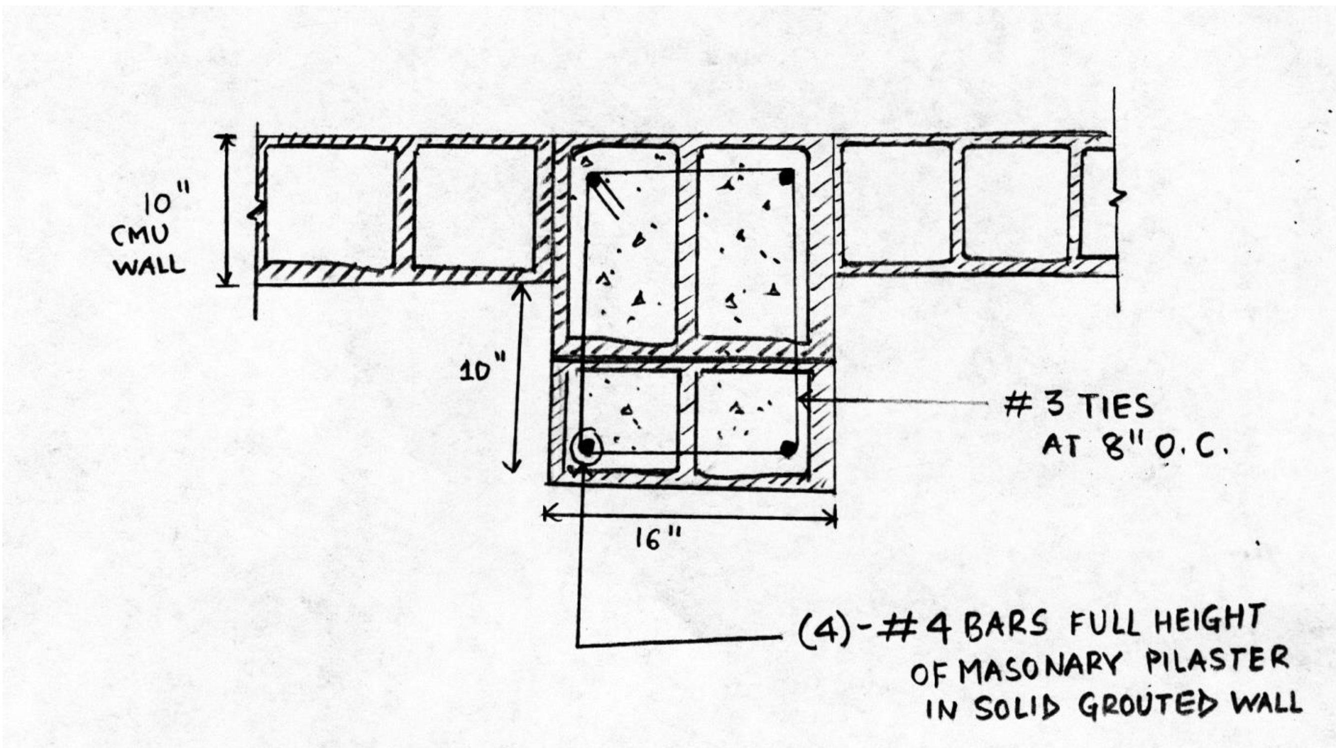
<u>Lateral Ties:</u>

Provide #3 ties @8” o.c.

Development Length:

$$ld = \frac{0.13\ db^2\ fy\ \gamma}{k\ \sqrt{\text{f'm}}} = \frac{0.13(\,0.5)^2\ (\,60\ ksi)\ (1)}{2"\ \ \sqrt{2000}} = 21.80 \leq\ 72\ \text{db} = 36 \qquad O.K.$$

Provide 21.80" lap length



16'×20' CMU Pilaster

Masonry CMU Headers:

Header along line D over large garage door

Dead load = 22.84 psf

Live Load = 20 psf

Simple span Lintel L = 12’8” = 12.66 ft

$Wu = (22.84\, psf)(12.66\, ft) + (20\, psf)(12.66\, ft) = 542.35\, plf$

Lintel Steel (2) #6   As = 0.88 in^2 fy = 60 ksi

$$\mathrm{a} = \left(\frac{As\, fb}{0.8\, f'm\;\; b}\right) = \left(\frac{(0.88\, in^2)(60\, ksi)}{0.8(2\, ksi)(9.625")}\right) = 3.42"$$

$$\phi\mathrm{Mn} = \phi\, As\, fy\left(d - \frac{a}{2}\right) = (0.9)(60\, ksi)(0.88\, in^2)\left(14" - \frac{3.42"}{2}\right) = 584\, k - in$$

$$\mathrm{Mu} = \frac{1}{8}wL^2 = \left(\frac{1}{8}\right)\left(542.35\frac{lb}{ft}\right)(12.66\, ft)^2\left(12\frac{in}{ft}\right) = 130.38\, k - in < \phi Mn \quad O.K.$$

Max Reinforcement:

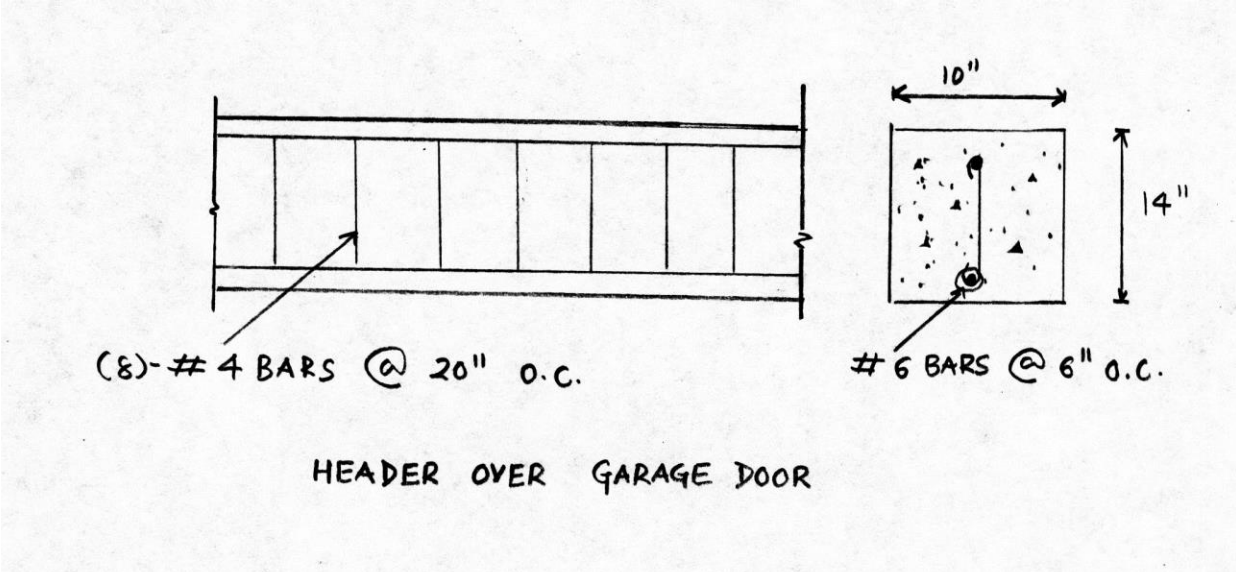
$$\frac{a}{d} = \frac{3.42"}{14"} = 0.24 < 0.356 \quad O.K.$$

Minimum Reinforcement:

$$\phi 1.3Mcr = \phi\;\; 1.3\;\; fr\;\; b\;\frac{h^2}{6} = (0.9)(1.3)(267\, psi)(9.625\, in)\left(\frac{(14)^2}{6}\right) = 98.22 \;\leq\; \phi Mn \quad O.K.$$

Provide #6 grade 2 bars 6” o.c.

Stirrups #4 grade 8 bars @ 20” o.c.



Header over the large balcony opening lines 2 and 3:

Effective length = 18' + 2(1'-4") = 20.66 ft

Dead Load = 20 psf   Live Load = 20 psf   Wu = 413.2 plf

Lintel Steel 2 #5 bars   As = 0.62 in^2 fy = 60 ksi

$$\mathrm{a} = \left(\frac{As\,fb}{0.8\,f'm\ b}\right) = \left(\frac{(0.62\ in^2)(60\ ksi)}{0.8(2\ ksi)(9.625")}\right) = 2.41"$$

$$\phi\mathrm{Mn} = \phi\ As\ fy\left(d - \frac{a}{2}\right) = (0.9)(60\ ksi)(0.62\ in^2)\left(14" - \frac{2.41"}{2}\right) = 428.3\ k - in$$

$$\mathrm{Mu} = \frac{1}{8}wL^2 = \left(\frac{1}{8}\right)\left(413.2\frac{lb}{ft}\right)(20.66\ ft)^2\left(12\frac{in}{ft}\right) = 264.55\ k - in < \phi Mn\ \ O.K.$$

Max Reinforcement:

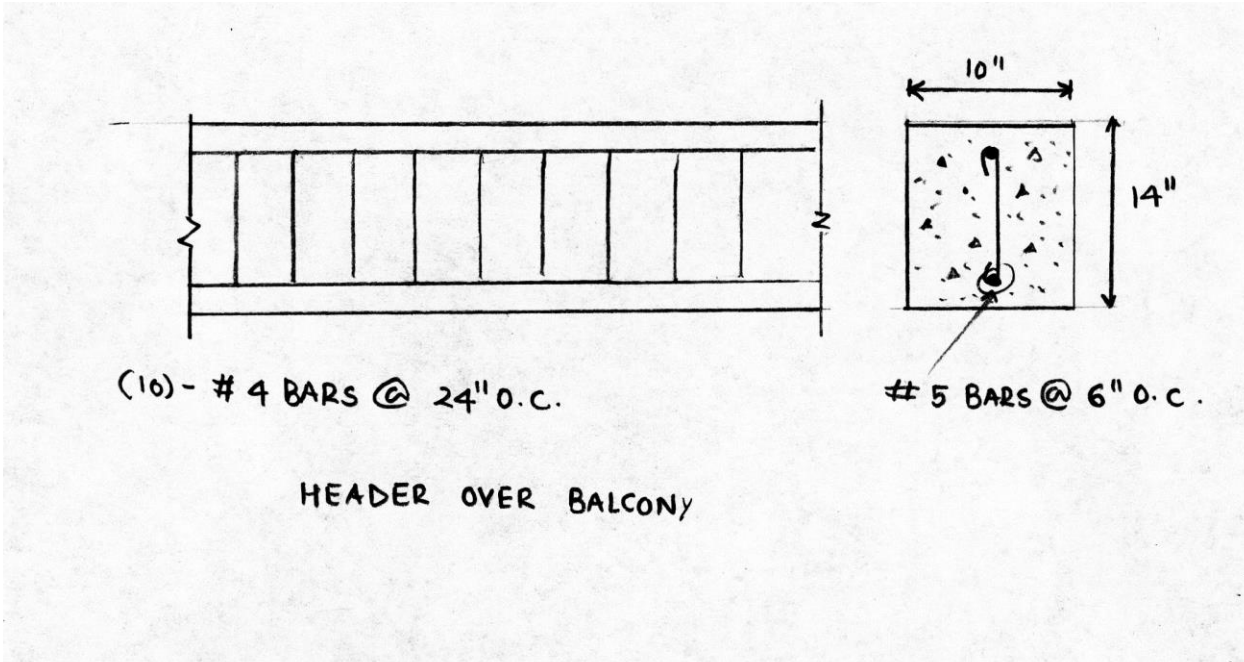
$$\frac{a}{d} = \frac{2.41"}{14"} = 0.172 < 0.356\quad O.K.$$

Minimum Reinforcement:

$$\phi 1.3Mcr = \phi\ 1.3\ fr\ b\ \frac{h^2}{6} = (0.9)(1.3)(267\ psi)(9.625\ in)\left(\frac{(14)^2}{6}\right) = 98.22\ \leq\ \phi Mn\ \ O.K.$$

Provide #5 grade 2 bars 6" o.c.

Stirrups #4 grade 10 bars @ 24" o.c.



Hanger Design Using Simpson Strong-Ties:

For a 2x10 Ceiling Joist:

$$V = \frac{(40\, psf)(\frac{16}{12})(20ft)}{2} = 533\ lbs$$

Use LUS 28 (1360 lbs)

For a 2x8 Ceiling Joist:

$$V = \frac{(40\, psf)(\frac{16}{12})(16ft)}{2} = 427\ lbs$$

Use LUS 28 (1360 lbs)

For a 2x12 Floor Joist:

$$V = \frac{(110\, psf)(\frac{12}{12})(16ft)}{2} = 880\ lbs$$

Use LUS 28 (1360 lbs)

For a 2x14 Ceiling Joist:

$$V = \frac{(110\, psf)(\frac{12}{12})(20ft)}{2} = 1100\ lbs$$

Use LUS 28 (1360 lbs)

For a 4x12:

$$V = \frac{(40\, psf)(5ft)(18ft)}{2} = 1800\ lbs$$

Use LUS 410 (2265 lbs)

Anchor Bolts for Ledger:

Anchor of diameter = 0.75   embedment = 6 inch f'm = 1500 psi A = 0.44 Apt = 113 fy = 36 ksi

We check for:

Masonry Shear Crushing:

$$\phi Bvnc = \phi(1050)\sqrt[4]{f'm * Ab} = (0.5)(1050)\sqrt[4]{(1500\, psi)(0.44)} = 2600\; lbs < --\; Controls$$

Anchor Shear Pry out:

$$\phi Bvpry = \phi(8)(Apt)\sqrt{\mathrm{f'm}} = (0.5)(8)(113)\sqrt{1500} = 17506\; lbs$$

Steel Shear Yielding:

$$\mathrm{\phi Bvns = \phi(0.6)(As)(fy) = (0.9)(0.6)(0.44)(36000) = 8554\; lbs}$$

Anchors can be place @ 48 inch o.c.

References

- *National Design Specification for Wood Construction – Wood Design Package, 2015, AWC, http://www.awc.org/codes-standards/publications/nds-2015*
- *2012 Design of Reinforced Masonry Structures, 7th ed., Brandow et al., CMACN, http://whymasonry.org/2012dorms/*
- *Design of Wood Structures ASD/LRFD, Breyer et al., 2015, 7th ed., McGraw-Hill Co.*
- *CE 4630: Timber & Masonry Design - Spring 2018 Notes From Josh Gebelein*