Great Northern Services Housing

A Senior Project

presented to

Great Northern Services and the Faculty of the Architectural Engineering Department

California Polytechnic State University, San Luis Obispo

In Partial Fulfillment

of the Requirements for the Degree

Bachelor of Science in Architectural Engineering

by

Rory Sebastian de Sevilla

December 2017

INTRODUCTION TO PROJECT

Background

The purpose of this senior project is primarily to assist the community of Weed, CA after the Boles Fire in September 2014. The Boles fire destroyed 152 homes in the area of Weed, nearly one third of the entire city. Because of this fire, there is a need for new affordable housing in Weed. The non-profit organization, Great Northern Services (GNS), partnered with Cal Poly to design housing for a new subdivision.

Team

This senior project was unique in that it was interdisciplinary. Each team consisted of four senior level students: one Architectural Engineering student, one Construction Management student, and two Architecture students. The purpose of the interdisciplinary aspect of the teams was to implement Integrated Project Delivery (IPD) methodology.

IPD Approach

IPD is an emerging project delivery methodology focused on holistic design. IPD starts and ends with the entire design team working together as a unit to ensure the best possible product is delivered to a client. Most importantly, the IPD methodology involves every discipline early on in the design so as to avoid costly errors.

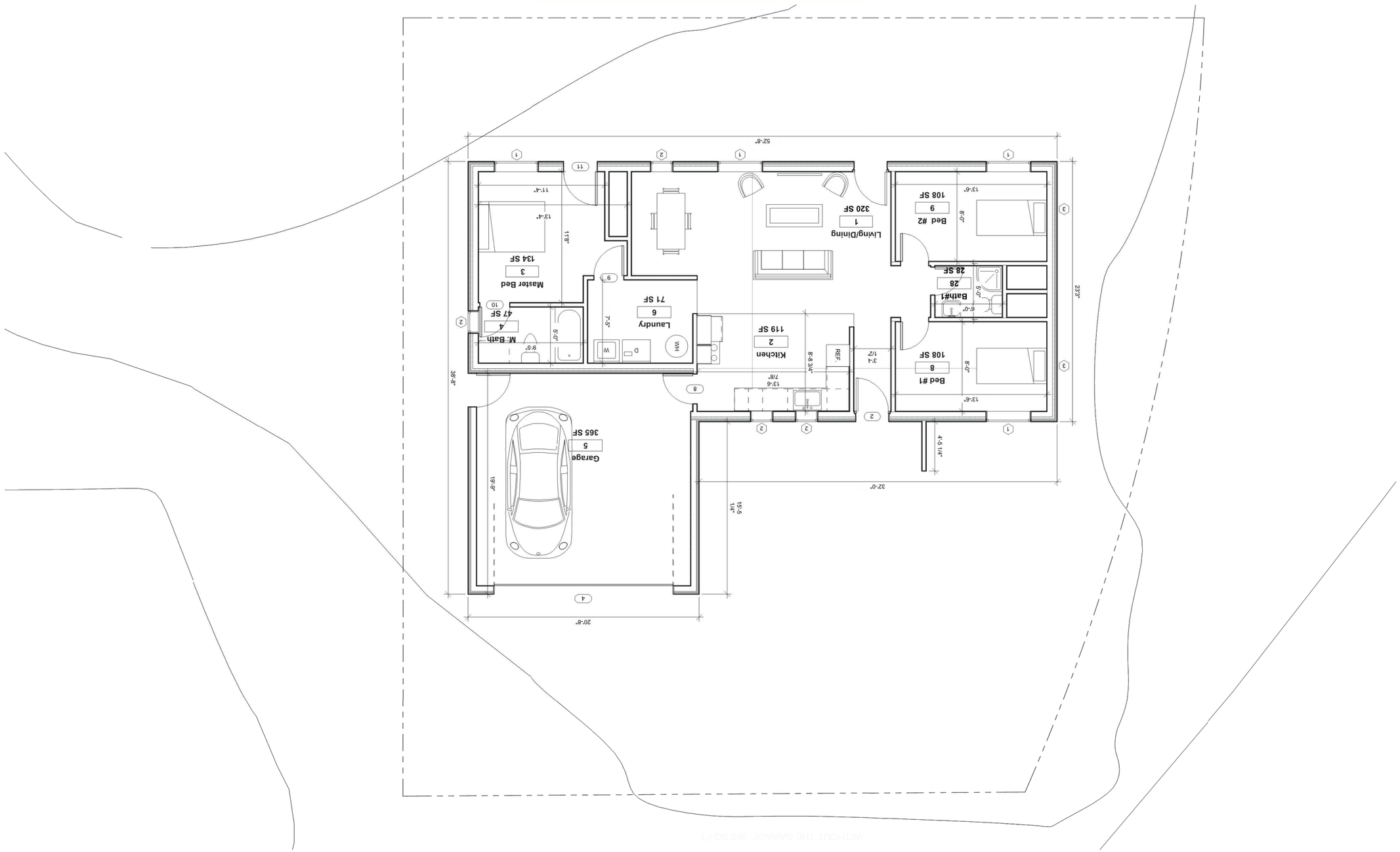
Constraints

The project was constrained to a 1.3 acre site on which there was previously a library, head start program, and an emergency food pantry. These facilities were destroyed after the Boles fire mentioned above. The site was then subdivided into seven lots by GNS for single-family residences and duplexes. The residences were required to be sized at 1200-1500 square feet, contain three bedrooms, two bathrooms, and placed within the setbacks established by the city of Weed. Finally, an especially challenging constraint was prefabrication: the residences had to be partially prefabricated and shipped to Weed after being built by Construction Management students on Cal Poly's campus.

Client Interaction

Throughout this project, the design teams met with the client (GNS) through face-to-face meetings and conference calls. These meetings gave direction for the project and uncovered the goals, objectives, and values of the client that were to be implemented in the design.

Dimension Plan



Elevations

NORTH

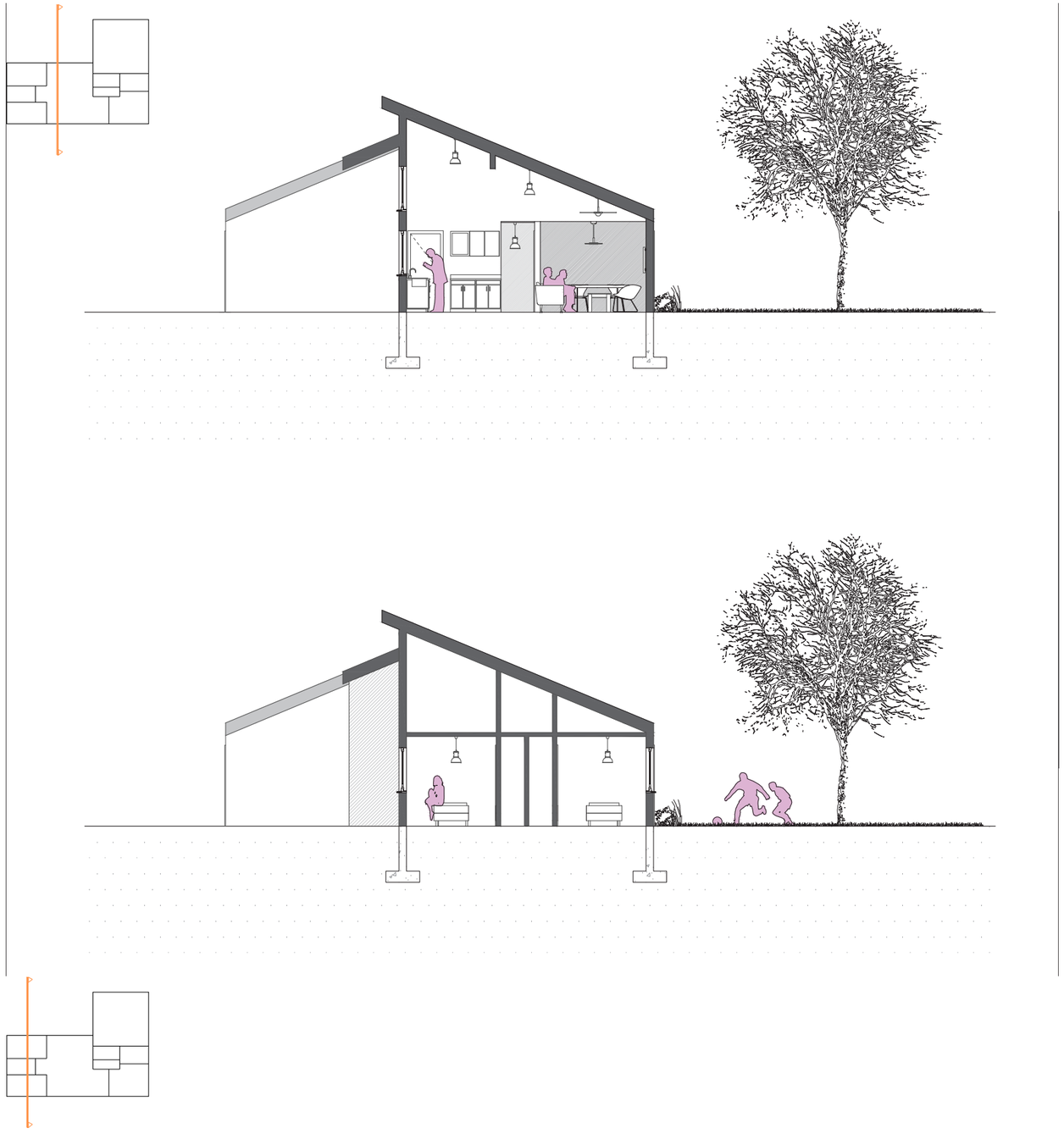
SOUTH

Elevations

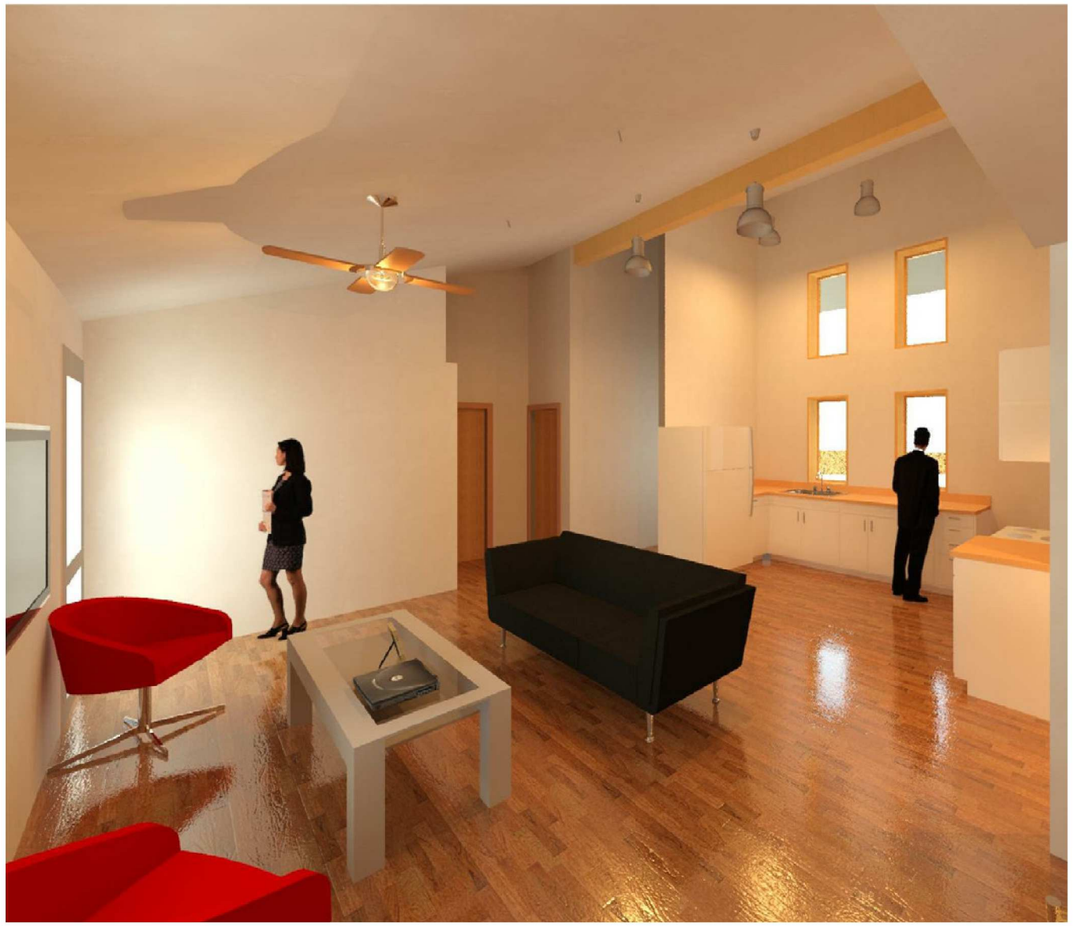
EAST

WEST

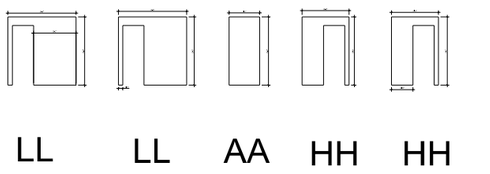
Sections

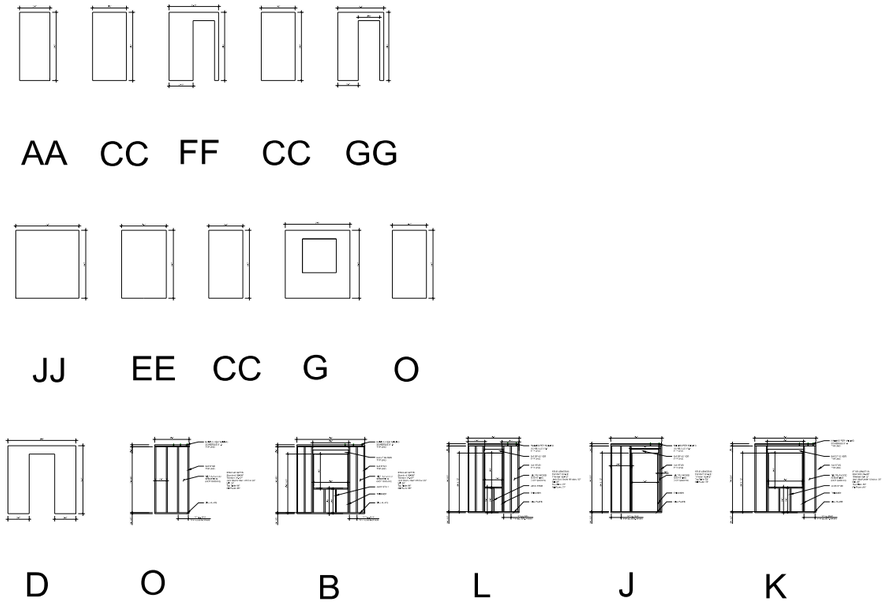


Renders



Panelization





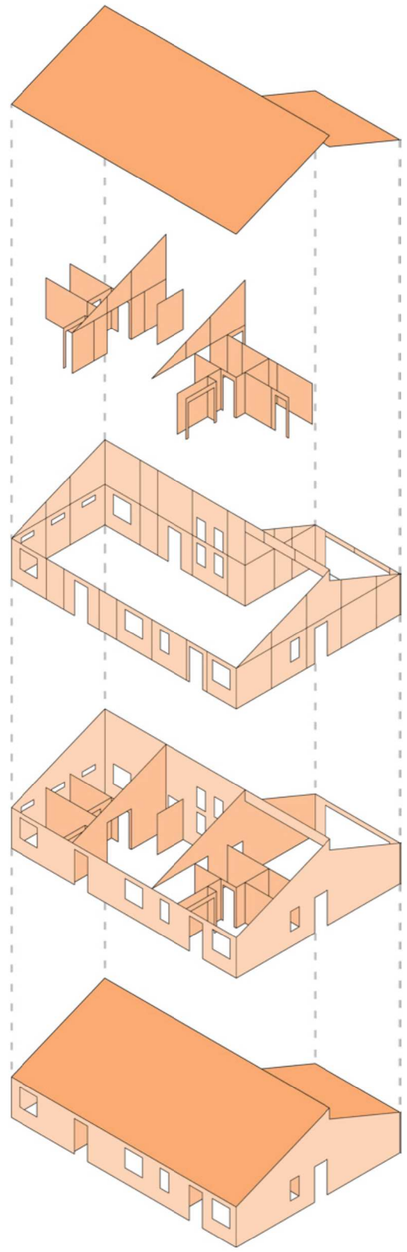
ROOFING

INTERIOR PANELS

EXTERIOR PANELS

INTERIOR + EXTERIOR

WALLS + ROOFING

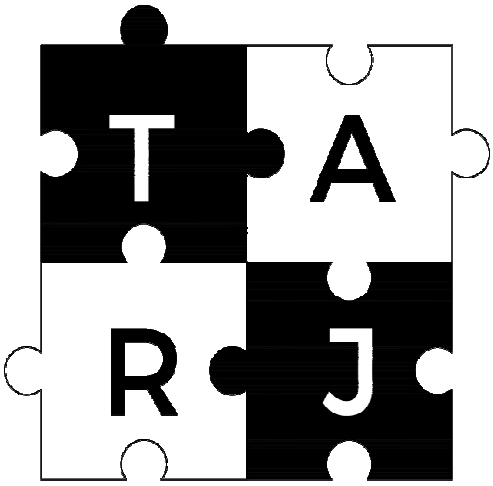


Weed Housing: GNS Davis Ave. Weed, Ca 96094

BY: TARJ Estimator

SF: 1,543

Level 1	Level 2	Level 3	Low Price	Target Price	High Price
Major Group Elements	Group Elements	Individual Elements	Totals	Totals	Totals
A. Substructure	A10 Foundations	Standard Foundations	$ 10,655.93	$ 11,839.93	$ 13,023.92
		Special Foundations	$ 8,359.18	$ 9,287.98	$ 10,216.77
		Slab on Grade	$ 2,285.76	$ 2,539.74	$ 2,793.71
B. Shell	B10 Superstructure	Floor Construction	$ 5,682.37	$ 6,313.74	$ 6,945.11
		Roof Construction	$ 3,041.84	$ 3,379.83	$ 3,717.81
	B20 Exterior Encloure	Exterior Walls	$ 26,367.89	$ 29,297.65	$ 32,227.42
		Exterior Windows	$ 3,770.65	$ 4,189.61	$ 4,608.57
		Exterior Doors	$ 6,099.94	$ 6,777.71	$ 7,455.48
	B30 Roofing	Roof Coverings	$ 12,786.26	$ 14,206.96	$ 15,627.66
		Roof Openings	$ -	$ -	$ -
C. Interiors	C10 Interior Constructio	Partitions	$ 2,349.59	$ 2,610.66	$ 2,871.73
		Interior Doors	$ 1,127.79	$ 1,253.10	$ 1,378.41
		Fittings	$ 205.00	$ 225.50	$ 248.05
	C30 Interior Finishes	Wall Finishes	$ 8,046.00	$ 8,940.00	$ 9,834.00
		Floor Finishes		$ 3,938.65	$ 4,332.52
		Ceiling Finishes	$ 3,987.50	$ 3,987.50	$ 4,386.25
D. Services	D10 Plumbing	Plumbing Fixtures	$ 3,909.63	$ 4,344.03	$ 4,778.43
		Domestic Water Dist.	$ 725.76	$ 806.40	$ 887.04
		Sanitary Waste	$ 141.75	$ 157.50	$ 173.25
		Rain Water Drainage	$ 1,666.10	$ 2,000.00	$ 2,200.00
		Other Plumbing Systems	$ 2,569.83	$ 2,855.37	$ 3,140.91
	D30 HVAC	Energy Supply	$ 2,700.00	$ 3,000.00	$ 3,300.00
		Heat Generating Systems	$ 1,704.60	$ 1,894.00	$ 2,083.40
		Cooling Generating Systems	$ -	$ -	$ -
		Distribution Systems	$ 249.88	$ 277.64	$ 305.40
		Controls & Instrumentations	$ 134.38	$ 147.82	$ 162.60
	D40 Fire Protection	Sprinklers	$ 6,126.78	$ 6,126.78	$ 6,739.46
	D50 Electrical	Electrical Service & Distributi	$ 2,934.90	$ 3,261.00	$ 3,587.10
		Lighting and Branch Wire	$ 8,000.00	$ 8,800.00	$ 9,680.00
		Communications & Security	$ 754.42	$ 838.24	$ 922.06
		Total	$ 126,383.73	$ 143,297.32	$ 157,627.05



STRUCTURAL CALCULATIONS

PERTAINING TO

IPD SENIOR PROJECT

IN WEED, CA

FOR

GREAT NORTHERN SERVICES

FINAL SUBMITTAL

PREPARED BY: RORY DE SEVILLA

TABLE OF CONTENTS

PROJECT: GNS Weed Housing	DATE: 12/1/17
PREPARED BY: Rory de Sevilla	SHEET: 1
NOTES: N/A	

GENERAL CRITERIA

Risk Category:	II	*Per ASCE 7-10 Table 1.5-1*	
Seismic Importance Factor	1	*Per ASCE 7-10 Table 1.5-2*	
Wind Importance Factor	1	*Per ASCE 7-10 Table 1.5-2*	
Seismic Design Category	D	*Per ASCE 7-10 Table 11.6-1*	
Exposure Category	C		
Wind Speed (V)	110	MPH	*Per ASCE 7-10 Figure 26.5-1A*

SOILS CRITERIA

Bearing Capacity:	**1500**	PSF
Bearing Capacity (Seismic):	**2000**	PSF
Coefficient of Friction:	**0.25**	
Frost Line Depth:	**12**	IN.
Minimum ftg. Depth Below Frost Line:	**12**	IN.

SEISMIC CRITERIA (ASCE 7 Ch.12.8)

S_1	0.328	*(VALUES FROM*	Ts	0.64	s	Cs max	0.27
S_{DS}	0.594	*USGS WEBISTE)*	0.8*Ts	0.51	s	Cs min	0.03
S_S	0.736		Ta	0.15	s	R =	6.50
S_{MS}	0.891		T	0.21	s	Cs =	0.09
S_{D1}	0.381		Ct	0.02			
S_{M1}	0.572		Hn	15.00	ft	*(ASCE 7 Ch. 12.8)*	
			x	0.75			
			Cu	1.40			

SNOW CRITERIA (ASCE 7-10 Ch.7)

Ground Snow Loads	**86**	PSF
Flat Roof Snow Loads	**60**	PSF
Sloped Snow Loads (Cold Attic)	**48**	PSF
Sloped Snow Loads (Warm Attic)	**42**	PSF

PROJECT: GNS Weed Housing	DATE: 12/1/17
PREPARED BY: Rory de Sevilla	SHEET: 2
NOTES: N/A	

ROOF DEAD LOAD TAKE OFF

Metal Roofing	1.0	psf
Sheathing (1/2" Plywood)	2.0	psf
Insulation (6" Fiberglass Batting)	1.0	psf
Sheathing (1/2" Plywood)	1.5	psf
MEP & Misc. (Sprinklers/Felt)	3.0	psf
Joists/Trusses	3.0	psf
	11.5	psf (Joists, Trusses)
Beam	1.0	psf
	12.5	psf (Beam)
Column	1.0	psf

Total Roof Dead Load	13.5	psf
Roof Live Load	20.0	psf
roof snow load (warm attic)	42.0	psf
roof snow load (cold attic)	48.0	psf
Roof Area	1790	sf
Roof Weight	24165	lb

FLOOR DEAD LOAD TAKE OFF

Flooring (Wood Laminate Flooring)	1.5	psf
Sheathing	2.5	psf
Insulation	3.0	psf
MEP & Misc.	3.0	psf
Joists	3.0	psf
	13.0	psf (Joists)
Beams	3.0	psf
	16.0	psf (Girders)
Column	3.0	psf

Total Floor Dead Load	19.0	psf
Residential Live Load	40.0	psf
floor area	1509	sf
Floor Weight	28671	lb

PROJECT: GNS Weed Housing	DATE: 12/1/17
PREPARED BY: Rory de Sevilla	SHEET: 3
NOTES: N/A	

WALL WEIGHT TAKE OFF

6-Inch Masonry Wall	81.0 psf

EXTERIOR WOOD WALLS

1/2" Gypsum Board	2.5 psf
1/2" Plywood	1.5 psf
2x6 Studs @ 16" o.c.	1.6 psf
Cement Fiberboard	3.0 psf
Insulation (2" Rigid StyroFoam)	1.0 psf
MEP & Misc.	1.5 psf
Total Unit Weight	**11 psf**
Wall Area	**1821 ft^2**
Total Weight	**20213 lb**

INTERIOR WOOD WALLS

1/2" Gypsum Board (x2)	5.0 psf
MEP & Misc.	1.0 psf
2x4 Studs @ 16" o.c.	1.0 psf
Total Unit Weight	**7 psf**
Wall Area	**1186 ft^2**
Total Weight	**8302 lb**

TOTAL WEIGHT OF BUILDING

Roof Weight	24165.0	lbs
Exterior Wall Weigh	10106.6	lbs
Interior Wall Weight	4151.0	lbs
Total Weight Of Buil	**38422.6**	lbs

WEIGHT = W

BASE SHEAR

V = Cs * W = (0.091)*(44,705) lbs = **3511.2 K**

PROJECT: GNS Weed Housing	**DATE:** 12/1/17
PREPARED BY: Rory de Sevilla	**SHEET:** R1
NOTES: N/A	

Indicates User Input

Member: Critical roof rafters (Cold attic)

RB01

NOTE -->5:12 slope increases D load

STEP 1 - General Information

D (Sloped)	=	12.46 psf
Lr (Reducible)	=	20 psf
S (snow)	=	48 psf
Span	=	18 ft
Spacing	=	1.333333 ft
Def max (S)	=	0.9 in
Def max (D+S)	=	1.2 in
w (S)	=	63.999984 plf
w (0.5D+S)	=	72.305537 plf
w (D+S)	=	80.611091 plf
w (S)/ Def (S)	=	853.33312
w (0.5D+S)/ Def(D +S	=	723.05537
Species and Type:		***DF #1***
Fb	=	1000 psi
Fv	=	180 psi
E	=	1700000 psi
* This case governs for deflection		

STEP 2 - Trial Size (Def Driven)

$E' = E * C_M * C_T * C_I$

C_M	=	1.00	
C_T	=	1.00	
C_I	=	1.00	
E'	=	1700000 psi	

Trial Member

I_{req}	=	98.80	in4
Member Size:		***2 x 12***	
Actual		1.5 in	11.3 in
Ixx	=	178.0 in4	
S	=	31.6 in3	
A	=	16.9 in2	

Ixx > Ireq? YES

STEP 3 - Moment Capacity

$F'_b = F_b * C_D * C_M * C_T * C_L * C_F * C_{fu} * C_I * C_R$

C_D =	1.15	C_L =	1.00	C_I =	1.00		
C_M =	1.00	C_F =	1.00	C_R =	1.15		
C_T =	1.00	C_{fu} =	1.00	F'_b =	1323 psi		

$f_b = (M_u * 12)/S$

M_u	=	3265 lbft
S	=	31.6 in3
f_b	=	1238 psi

$F'_b > f_b$? YES

STEP 4 - Shear Capacity

$F'_v = F_v * C_D * C_M * C_T * C_I$

C_D =	1.15	C_T =	1.00	V_u =	725.5 lbs
C_M =	1.00	C_I =	1.00	A =	16.9 in2
		F'_v =	207 psi	f_v =	64.5 psi

$f_v = 1.5 * (V/A)$

$F'_v > f_v$? YES

PROJECT: GNS Weed Housing	**DATE:** 12/1/17
PREPARED BY: Rory de Sevilla	**SHEET:** R2
NOTES: N/A	

Indicates User Input

Member: Roof rafters (over living/dining)

RB02

NOTE -->5:12 slope increases D load

STEP 1 - General Information

D (Sloped)	=	12.46 psf
Lr (Reducible)	=	20 psf
S (snow)	=	48 psf
Span	=	14 ft
Spacing	=	1.33333 ft
Def max (S)	=	0.7 in
Def max (D+S)	=	0.9333333 in
w (S)	=	63.99984 plf
w (0.5D+S)	=	72.305375 plf
w (D+S)	=	80.61091 plf
w (S)/ Def (S)	=	1097.1401
w (0.5D+S)/ Def(D +S	=	929.64053
Species and Type:		*DF #1*
Fb	=	1000 psi
Fv	=	180 psi
E	=	1700000 psi

* This case governs for deflection

STEP 2 - Trial Size (Def Driven)

$E' = E * C_M * C_T * C_I$

C_M	=	1.00	
C_T	=	1.00	
C_I	=	1.00	
E'	=	1700000 psi	

Trial Member

I req	=	46.49	in4
Member Size:		*2 x 12*	
Actual		1.5 in	11.3 in
Ixx	=	178.0 in4	
S	=	31.6 in3	
A	=	16.9 in2	

Ixx > Ireq? YES

STEP 3 - Moment Capacity

$F'_b = F_b * C_D * C_M * C_T * C_L * C_F * C_{fu} * C_I * C_R$

C_D =	1.15	C_L =	1.00	C_I =	1.00		
C_M =	1.00	C_F =	1.10	C_R =	1.15		
C_T =	1.00	C_{fu} =	1.00	F'_b =	1455 psi		

$f_b = (M_u * 12)/S$

M_u	=	1975 lbft
S	=	31.6 in3
f_b	=	749 psi

$F'_b > f_b$? YES

STEP 4 - Shear Capacity

$F'_v = F_v * C_D * C_M * C_T * C_I$

$f_v = 1.5 * (V/A)$

C_D =	1.15	C_T =	1.00	V_u =	564.3 lbs	
C_M =	1.00	C_I =	1.00	A =	16.9 in2	
		F'_v =	207 psi	f_v =	50.2 psi	

$F'_v > f_v$? YES

PROJECT: GNS Weed Housing	**DATE:** 12/1/17
PREPARED BY: Rory de Sevilla	**SHEET:** R3
NOTES: N/A	

Indicates User Input

Member: **Roof Beam (over kitchen/entrance)**

RB03

NOTE -->5:12 slope increases D load

STEP 1 - General Information

D (Sloped)	=	13.54 psf
Lr (Reducible)	=	20 psf
S (snow)	=	42 psf
Span	=	17.5 ft
Trib Width	=	11 ft
Def max (S)	=	0.875 in
Def max (D+S)	=	1.1666667 in
w (S)	=	462 plf
w (0.5D+S)	=	536.47917 plf
w (D+S)	=	610.95833 plf
w (S)/ Def (S)	=	6336
w (0.5D+S)/ Def(D +S	=	5518.0714
Species and Type:		*DF #1*
Fb	=	1350 psi
Fv	=	170 psi
E	=	1600000 psi
* This case governs for deflection		

STEP 2 - Trial Size (Def Driven)

E' = E * Cm * Ct * Ci

Cm	=	1.00	
Ct	=	1.00	
Ci	=	1.00	
E'	=	1600000 psi	
Emin	=	580000 psi	
Trial Member			
I req	=	696.38	in4
Member Size:		*6 x 16*	
Actual		5.5 in	15.5 in
Ixx	=	1706.8 in4	
S	=	220.2 in3	
A	=	85.3 in2	

Ixx > Ireq? YES

Reactions:

Left: 5346

Right 5346

STEP 3 - Moment Capacity

F'b = Fb * Cd * Cm * Ct * CL * CF * Cfu * Ci * Cr

fb = (Mu * 12)/S

Cd =	1.15	CL =	see blw	Ci =	1.00			Mu	=	23388	lbft
Cm =	1.00	CF =	0.97	Cr =	1.00			S	=	220.2	in3
Ct =	1.00	Cfu =	1.00	Fb* =	1509	psi		fb	=	1274	psi
				F'b =	1457	psi				F'b > fb?	YES

STEP 4 - Shear Capacity

F'v = Fv * Cd * Cm * Ct *Ci

fv = 1.5 * (V/A)

Cd =	1.15	Ct =	1.00	Vu =	5345.9	lbs
Cm =	1.00	Ci =	1.00	A =	85.3	in2
		F'v =	195.5 psi	fv =	94.1	psi

F'v > fv? YES

For CL:		Fb* =	
Leff =	32.4 ft	Fbe/fb* =	2.3
Rb =	14.1	CL =	0.965
Fbe =	3494 psi		

PROJECT: GNS Weed Housing	**DATE:** 12/1/17
PREPARED BY: Rory de Sevilla	**SHEET:** R4
NOTES: N/A	

Indicates User Input

Member: **Garage Header**

RB04

STEP 1 - General Information

D (Sloped)	=	12.5 psf
Lr (Reducible)	=	20 psf
S (snow)	=	48 psf
Span	=	16 ft
Trib Width	=	8 ft
Def max (S)	=	0.8 in
Def max (D+S)	=	1.0666667 in
w (S)	=	384 plf
w (0.5D+S)	=	434 plf
w (D+S)	=	484 plf
w (S)/ Def (S)	=	5760
w (0.5D+S)/ Def(D +S	=	4882.5
Species and Type:		*DF #1*
Fb	=	1350 psi
Fv	=	170 psi
E	=	1600000 psi
* This case governs for deflection		

STEP 2 - Trial Size (Def Driven)

$E' = E * C_M * C_T * C_I$

C_M	=	1.00	
C_T	=	1.00	
C_I	=	1.00	
E'	=	1600000 psi	
Emin	=	580000 psi	
Trial Member			
I req	=	442.37	in4
Member Size:		*6 x 14*	
Actual		5.5 in	13.5 in
Ixx	=	1127.7 in4	
S	=	167.1 in3	
A	=	74.3 in2	

Ixx > Ireq? YES

STEP 3 - Moment Capacity

$F'_b = F_b * C_D * C_M * C_T * C_L * C_F * C_{fu} * C_I * C_R$

$f_b = (M_u * 12)/S$

C_D =	1.15	C_L =	see blw	C_I =	1.00			Mu	=	15488	lbft
C_M =	1.00	C_F =	0.99	C_R =	1.00			S	=	167.1	in3
C_T =	1.00	C_{fu} =	1.00	F_b* =	1532	psi		fb	=	1112	psi
				F'_b =	1494	psi				F'_b > fb?	YES

STEP 4 - Shear Capacity

$F'_v = F_v * C_D * C_M * C_T * C_I$

$f_v = 1.5 * (V/A)$

C_D =	1.15	C_T =	1.00	Vu =	3872.0	lbs
C_M =	1.00	C_I =	1.00	A =	74.3	in2
		F'_v =	195.5 psi	fv =	78.2	psi

F'_v > fv? YES

For C_L:

Leff =	29.5 ft	Fbe/fb* =	2.9
Rb =	12.6	C_L =	0.975
Fbe =	4412 psi		

PROJECT: GNS Weed Housing	**DATE:** 12/1/17
PREPARED BY: Rory de Sevilla	**SHEET:** R5
NOTES: N/A	

Indicates User Input

Member: **Worst Typical Header**

RB05

STEP 1 - General Information

D	=	12.5 psf
Lr (Reducible)	=	20 psf
S (snow)	=	5 psf
Span	=	4 ft
Trib Width	=	9 ft
Def max (S)	=	0.2 in
Def max (D+S)	=	0.2666667 in
w (S)	=	45 plf
w (0.5D+S)	=	101.25 plf
w (D+S)	=	157.5 plf
w (S)/ Def (S)	=	2700
w (0.5D+S)/ Def(D +S	=	4556.25
Species and Type:		***DF #2***
Fb	=	750 psi
Fv	=	170 psi
E	=	1300000 psi

* This case governs for deflection

STEP 2 - Trial Size (Def Driven)

$E' = E * C_M * C_T * C_I$

Cm	=	1.00	
Ct	=	1.00	
Ci	=	1.00	
E'	=	1300000 psi	
Emin	=	370000 psi	
Trial Member			
I req	=	1.00	in4
Member Size:		***6 x 6***	
Actual		5.5 in	5.5 in
Ixx	=	76.3 in4	
S	=	27.7 in3	
A	=	30.3 in2	

Ixx > Ireq? **YES**

STEP 3 - Moment Capacity

$F'_b = F_b * C_D * C_M * C_T * C_L * C_F * C_{fu} * C_i * C_r$

$f_b = (M_u * 12)/S$

Cd =	1.15	CL =	see blw	Ci =	1.00			Mu =	315	lbft	
Cm =	1.00	CF =	1.00	Cr =	1.00			S =	27.7	in3	
Ct =	1.00	Cfu =	1.00	Fb* =	863	psi		fb =	136.3	psi	
				F'b =	861	psi			F'b > fb?		YES

STEP 4 - Shear Capacity

$F'_v = F_v * C_D * C_M * C_T * C_i$

$f_v = 1.5 * (V/A)$

Cd =	1.15	Ct =	1.00	Vu =	315.0	lbs
Cm =	1.00	Ci =	1.00	A =	30.3	in2
		F'v =	195.5 psi	fv =	15.6	psi

F'v > fv? **YES**

For Cl:

Leff =	7.9 ft	Fbe/fb* =	30
Rb =	4.15	Cl =	0.998
Fbe =	25775.8 psi		

PROJECT: GNS Weed Housing	**DATE:** 12/1/17
PREPARED BY: Rory de Sevilla	**SHEET:** R6
NOTES: N/A	

Indicates User Input

Member: **Beam Under Cripple Wall**

RB06

STEP 1 - General Information

D (Sloped)	=	12.5 psf
Lr (Reducible)	=	20 psf
S (snow)	=	48 psf
Span	=	20 ft
Trib Width	=	9 ft
Def max (S)	=	1 in
Def max (D+S)	=	1.3333333 in
w (S)	=	432 plf
w (0.5D+S)	=	488.25 plf
w (D+S)	=	544.5 plf
w (S)/ Def (S)	=	5184
w (0.5D+S)/ Def(D +S	=	4394.25
Species and Type:		*DF #1*
Fb	=	1350 psi
Fv	=	170 psi
E	=	1600000 psi

* This case governs for deflection

STEP 2 - Trial Size (Def Driven)

$E' = E * C_M * C_T * C_I$

Cm	=	1.00	
Ct	=	1.00	
Ci	=	1.00	
E'	=	1600000 psi	
Emin	=	580000 psi	
Trial Member			
I req	=	972.00	in4
Member Size:		*6 x 16*	
Actual		5.5 in	15.5 in
Ixx	=	1706.8 in4	
S	=	220.2 in3	
A	=	85.3 in2	

Ixx > Ireq? **YES**

STEP 3 - Moment Capacity

$F'_b = F_b * C_D * C_M * C_T * C_L * C_F * C_{fu} * C_i * C_R$

$f_b = (M_u * 12)/S$

Cd =	1.15	CL =	see blw	Ci =	1.00		Mu =	27225	lbft		
Cm =	1.00	CF =	0.97	Cr =	1.00		S =	220.2	in3		
Ct =	1.00	Cfu =	1.00	Fb* =	1509	psi	fb =	1483	psi		
				F'b =	1446	psi		F'b > fb?		YES	

STEP 4 - Shear Capacity

$F'_v = F_v * C_D * C_M * C_T * C_i$ | $f_v = 1.5 * (V/A)$

Cd =	1.15	Ct =	1.00	Vu =	5445.0	lbs
Cm =	1.00	Ci =	1.00	A =	85.3	in2
		F'v =	195.5 psi	fv =	95.8	psi

F'v > fv? **YES**

For CL:

Leff =	36.5 ft	Fbe/fb* =	2.1
Rb =	15	CL =	0.958
Fbe =	3103 psi		

PROJECT: GNS Weed Housing	**DATE:** 12/1/17	
PREPARED BY: Rory de Sevilla	**SHEET:** F1	
NOTES: N/A		

Indicates User Input

Member: CRITICAL FLOOR JOISTS UNDER MASTER

FB01

STEP 1 - General Information

D	=	13 psf
L (Nonreducible)	=	40 psf
Span	=	18 ft
Spacing	=	1.33333 ft
D max (L)	=	0.9 in
D max (D+L)	=	1.2 in
w (L)	=	53.3332 plf
w (0.5D+L)	=	61.999845 plf
w (D+L)	=	70.66649 plf
w (L)/ D (L)	=	711.10933
w (0.5D+L)/ D (D +L	=	619.99845
Species and Type:		DF #1
Fb	=	1000 psi
Fv	=	180 psi
E	=	1700000 psi
* This case governs for deflection		

STEP 2 - Trial Size (D Driven)

E' = E * Cm * Ct * Ci

Cm	=	1.00	
Ct	=	1.00	
Ci	=	1.00	
E'	=	1700000 psi	

Trial Member

I req	=	82.33	in4
Member Size:		2 x 12	
Actual		1.5 in	11.3 in
Ixx	=	178.0 in4	
S	=	31.6 in3	
A	=	16.9 in2	

Ixx > Ireq? YES

STEP 3 - Moment Capacity

F'b = Fb * Cd * Cm * Ct * CL * CF * Cfu * Ci * Cr

Cd =	1.00	CL =	1.00	Ci =	1.00		
Cm =	1.00	CF =	1.00	Cr =	1.15		
Ct =	1.00	Cfu =	1.00	F'b =	1150 psi		

fb = (Mu * 12)/S

Mu	=	2862 lbft
S	=	31.6 in3
fb	=	1085 psi

F'b > fb? YES

STEP 4 - Shear Capacity

F'v = Fv * Cd * Cm * Ct *Ci

Cd =	1.00	Ct =	1.00	Vu =	636.0 lbs
Cm =	1.00	Ci =	1.00	A =	16.9 in2
		F'v =	180 psi	fv =	56.5 psi

fv = 1.5 * (V/A)

F'v > fv? YES

PROJECT: GNS Weed Housing	**DATE:** 12/1/17	
PREPARED BY: Rory de Sevilla	**SHEET:** F2	
NOTES: N/A		

Indicates User Input

Member: CRITICAL FLOOR JOISTS UNDER MASTER

FB02

STEP 1 - General Information

D	=	13 psf
L (Nonreducible)	=	40 psf
Span	=	14 ft
Spacing	=	1.333333 ft
D max (L)	=	0.7 in
D max (D+L)	=	0.9333333 in
w (L)	=	53.33332 plf
w (0.5D+L)	=	61.999985 plf
w (D+L)	=	70.666649 plf
w (L)/ D (L)	=	914.28549
w (0.5D+L)/ D (D +L	=	797.14266
Species and Type:		DF #2
Fb	=	900 psi
Fv	=	180 psi
E	=	1600000 psi
* This case governs for deflection		

STEP 2 - Trial Size (D Driven)

E' = E * Cm * Ct * Ci

Cm	=	1.00
Ct	=	1.00
Ci	=	1.00
E'	=	1600000 psi

Trial Member

I req	=	41.16	in4
Member Size:		2 x 12	
Actual		1.5 in	11.3 in
Ixx	=	178.0	in4
S	=	31.6	in3
A	=	16.9	in2

Ixx > Ireq? YES

STEP 3 - Moment Capacity

F'b = Fb * Cd * Cm * Ct * CL * CF * Cfu * Ci * Cr

Cd =	1.00	CL =	1.00	Ci =	1.00		fb = (Mu * 12)/S	
Cm =	1.00	CF =	1.00	Cr =	1.15		Mu =	1731 lbft
Ct =	1.00	Cfu =	1.00	F'b =	1035 psi		S =	31.6 in3
							fb =	656.6 psi

F'b > fb? YES

STEP 4 - Shear Capacity

F'v = Fv * Cd * Cm * Ct *Ci

Cd =	1.00	Ct =	1.00		fv = 1.5 * (V/A)	
Cm =	1.00	Ci =	1.00	Vu =	494.7	lbs
		F'v =	180 psi	A =	16.9	in2
				fv =	44.0	psi

F'v > fv? YES

PROJECT: GNS Weed Housing	**DATE:** 12/1/17
PREPARED BY: Rory de Sevilla	**SHEET:** F3
NOTES: N/A	

Indicates User Input

Member: Roof Beam (over kitchen/entrance)

FB03

STEP 1 - General Information

D	=	16.00 psf
L (Reducible)	=	40 psf
Span	=	17 ft
Trib Width	=	11 ft
Def max (S)	=	0.85 in
Def max (D+S)	=	1.1333333 in
w (L)	=	440 plf
w (0.5D+L)	=	528 plf
w (D+L)	=	616 plf
w (S)/ Def (S)	=	6211.7647
w (0.5D+S)/ Def(D	=	5590.5882
Species and Type:		DF #1
Fb	=	1350 psi
Fv	=	170 psi
E	=	1600000 psi
* This case governs for deflection		

STEP 2 - Trial Size (Def Driven)

E' = E * Cm * Ct * Ci

Cm	=	1.00	
Ct	=	1.00	
Ci	=	1.00	
E'	=	1600000 psi	
Emin	=	580000 psi	
Trial Member			
I req	=	607.98	in4
Member Size:		6 x 16	
Actual		5.5 in	15.5 in
Ixx	=	1706.8 in4	
S	=	220.2 in3	
A	=	85.3 in2	

Ixx > Ireq? YES

STEP 3 - Moment Capacity

F'b = Fb * Cd * Cm * Ct * CL * CF * Cfu * Ci * Cr

fb = (Mu * 12)/S

Cd =	1.00	CL =	see blw	Ci =	1.00			Mu	=	22253	lbft
Cm =	1.00	CF =	0.97	Cr =	1.00			S	=	220.2	in3
Ct =	1.00	Cfu =	1.00	Fb* =	1312	psi		fb	=	1213	psi
				F'b =	1277	psi				F'b > fb?	YES

STEP 4 - Shear Capacity

F'v = Fv * Cd * Cm * Ct *Ci

fv = 1.5 * (V/A)

Cd =	1.00	Ct =	1.00	Vu =	5236.0	lbs
Cm =	1.00	Ci =	1.00	A =	85.3	in2
		F'v =	170 psi	fv =	92.1	psi

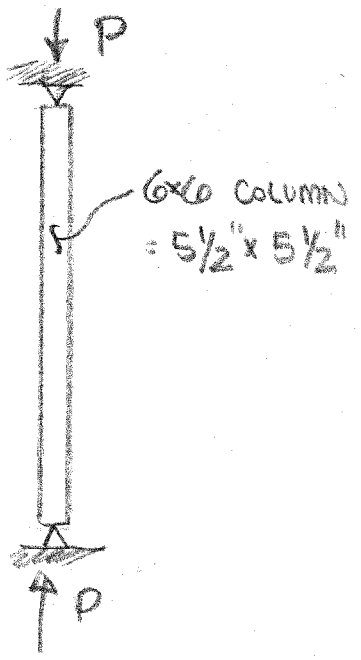
F'v > fv? YES

For CL:		Fb* =	
Leff =	31.6 ft	Fbe/fb* =	2.7
Rb =	13.9	CL =	0.973
Fbe =	3584 psi		

GRAVITY FRAMING

6x6 Column, Grid Intersection 4C

CRITICAL COLUMN



SECTION PROPERTIES:

$A = 30.25 in.^2$
$S_{xx} = 27.73 in.^3$
$I_{xx} = 76.26 in.^4$

Column Capacity:

$F'_c = F_c \cdot C_D \cdot \cancel{C_m}^{1.0} \cdot \cancel{C_t}^{1.0} \cdot C_F \cdot \cancel{C_i}^{1.0} \cdot C_P$

$1000psi \rightarrow C_D = 1.15$ (Snow)

$C_F = 1.0$

$F'_c = 1000psi(1.15)(C_P)$

Find C_P:

$$C_P = \frac{1 + F_{cE}/F_c^*}{2c} - \sqrt{\left[\frac{1+(F_{cE}/F_c^*)}{2c}\right]^2 - \left[\frac{F_{cE}/F_c^*}{c}\right]}$$

$c = 0.8$ for sawn lumber

$$F_{cE} = \frac{0.822\, E'_{min}}{(l_e/d)^2}$$

- From NDS, $E'_{min} = 580{,}000 psi \underbrace{(C_m C_t C_i C_T)}_{1.0} = 580{,}000 psi$
- $l_e = K_e l_u = 1(13') = 13'$

$$F_{cE} = \frac{0.822(580{,}000psi)}{\left[(13' \times 12''/1')/(5.5'')\right]^2} = 592.60\ psi$$

- $F_c^* = 1000psi(1.15) = 1150psi$

NEXT PAGE

- $F_{cE}/F_c^* = 592.6\text{psi} / 1150\text{psi} = \underline{0.52}$

- $C_p = \dfrac{1+(0.52)}{2(0.8)} - \sqrt{\left[\dfrac{1+(0.52)}{2(0.8)}\right]^2 - \left[\dfrac{0.52}{0.8}\right]} = 0.444$

$\Rightarrow \underline{C_p = 0.444}$

Find F'_c:

- $F'_c = F_c^* \cdot C_p = 1150\text{psi}(0.444) = \boxed{510.925\text{psi}}$

Find Max P for 13' height:

- $F'_c = \dfrac{P}{A}$

$\rightarrow P_{max} = F'_c A = 510.925\text{psi}(30.25\text{in}^2) = \boxed{15.46^K}$

CHECK BEARING ON RB03 (BEAM APPLYING LOAD):

6x16 — BEARING AREA = 30.25in^2

6+6

- $F'_{c\perp} = F_{c\perp}(C_M \times C_t \times C_i \times C_b)$

$\boxed{F'_{c\perp} = 625\text{psi}}$

$C_b = 1$ because $< 3"$ away from beam end.

CHECK MAX P from BEARING:

$F'_{c\perp} A = 625\text{psi}(30.25\text{in}^2) = \boxed{18{,}906\ \#} = P_{max}$

(Comp. Still Governs)

LOAD TO COLUMN 4C (FROM ROOF)

- SEE TRIB KEYPLAN
- TOTAL $P_{ROOF} = (D_R + 0.75S)(A_{Trib}) + D_{wall}(A_{wall})$

$$P_{ROOF} = (13.5psf + 36\ psf)([10' \times 12'] + [6' \times 1.33']) + (7psf \cdot 8' \times 3')$$

$$P_{ROOF} = 6440\# = 6.44^K < 15.46^K \ (OK)$$

BEARING CAP OF 6 x 16 (Governs)

LOAD TO COLUMN 4C (FROM FLOOR)

- SEE TRIB KEYPLAN
- TOTAL $P_{FLOOR} = (D_{FL} + 0.75L)(A_{trib})$

$$P_{FLOOR} = (19psf + (0.75 \cdot 40psf))([10' \times 12'] + [6' \times 1.33'])$$

$$P_{FLOOR} = 6272\# = 6.27^K$$

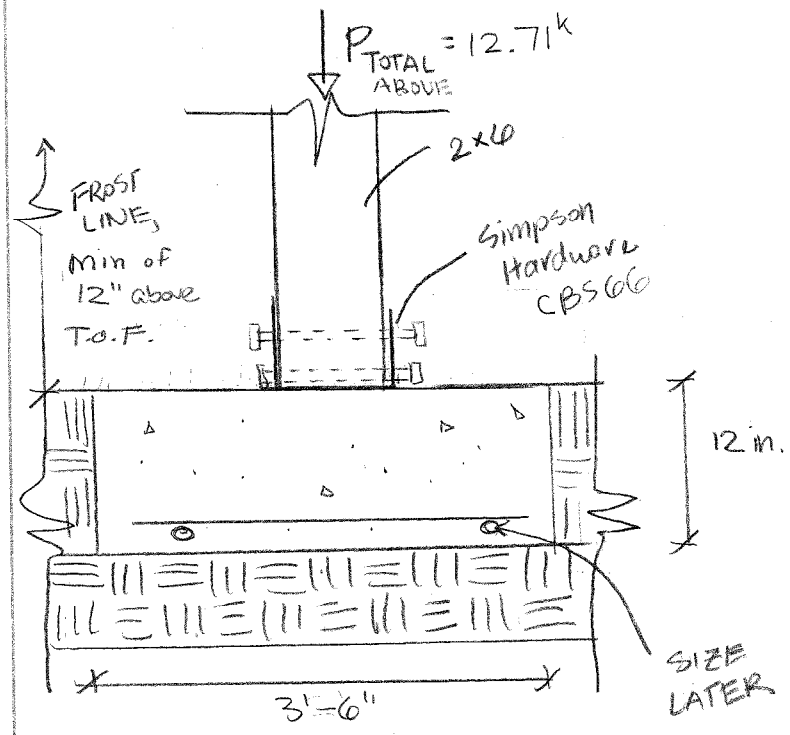
TOTAL LOAD TO COLUMN:

$$P_{ROOF} + P_{Floor} = 6440\# + 6272\# = 12.71^K < 15.46^K$$

OK

COMPRESSION CAPACITY OF 13' 6x6 Column

PAD FOOTING FOR COLUMN 3C (interior footing)



CONCRETE WEIGHT:

Assume $\rho_{conc.} = 150pcf$

$150pcf(3.5')^2(1') = 1837.5\#$

TOTAL LOAD:

$P_{TOT} + P_{conc} = P_{TOTAL}$

$P_{TOTAL} = 12.71^k + 1.838^k$

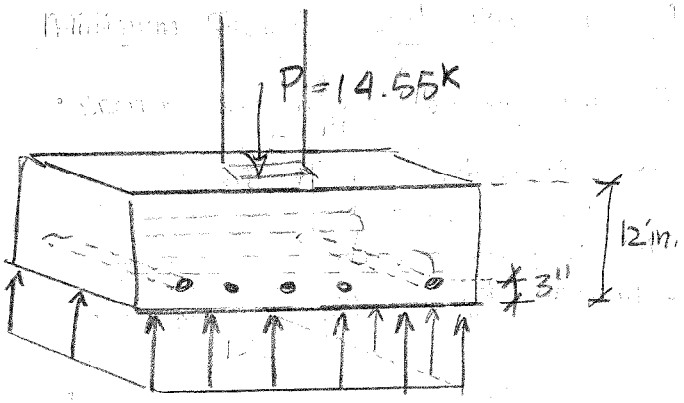
$P_{TOTAL} = \underline{14.55^k}$

• BEARING CAPACITY

$$f_{bearing} = \frac{14.55^k}{(3.5')^2} = \boxed{1.2^{ksf} < 1.5^{ksf} \ (ok)}$$

BEARING CAPACITY OF SOIL

• REINFORCEMENT

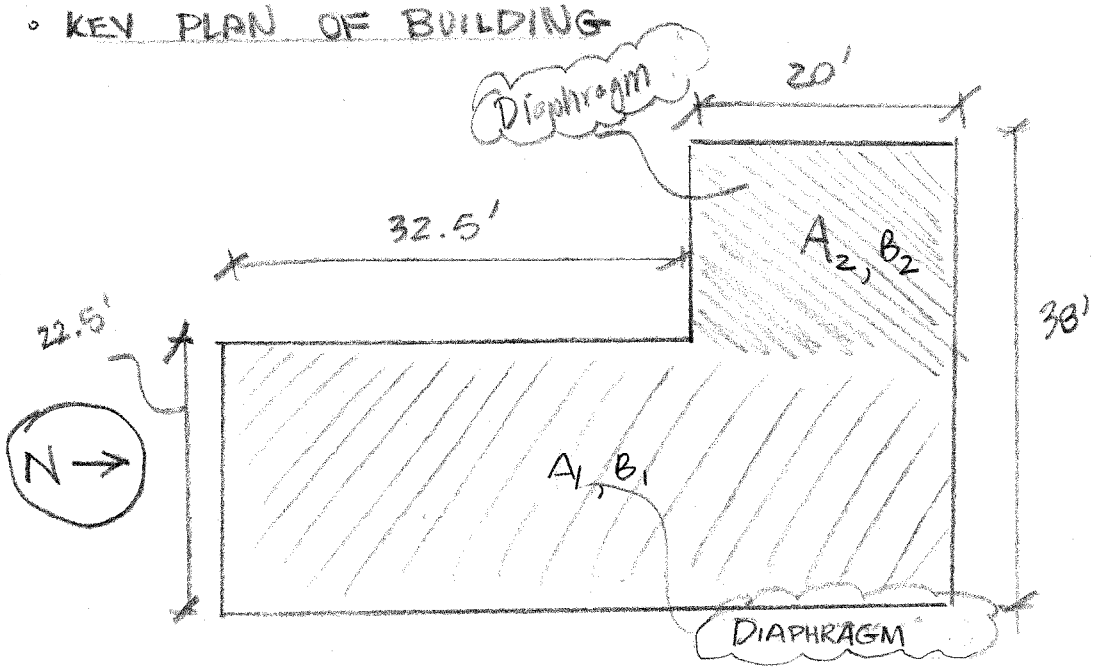


CHECK:

1. ONE-WAY BEAM SHEAR
2. TWO-WAY PUNCHING SHEAR
3. BENDING

WIND LOADING (N/S)

- KEY PLAN OF BUILDING



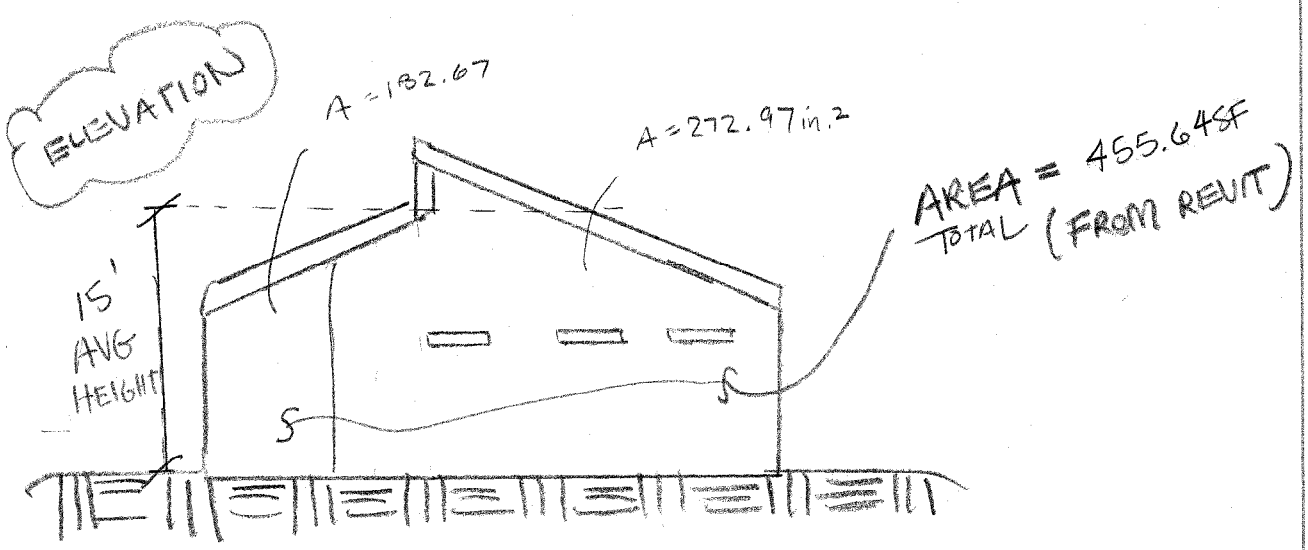
- TABLE 27.5-1, ASCE 7-10 (DIRECTIONAL PROCEDURE)
  - STEP (1): Risk Category II (Table 1.5-1)
  - STEP (2): V = 110 mph (Figure 26.5-1)
  - STEP (3): Wind Load Parameters
    - Exposure Category C
    - Topographic Factor $K_{zt} = 0$ (No hills or escarpments)
    - ENCLOSURE CLASSIFICATION (26.2)
      - ⇒ ENTIRELY ENCLOSED BUILDING
  - STEP (4): NET PRESSURES PER TABLE 27.6-1

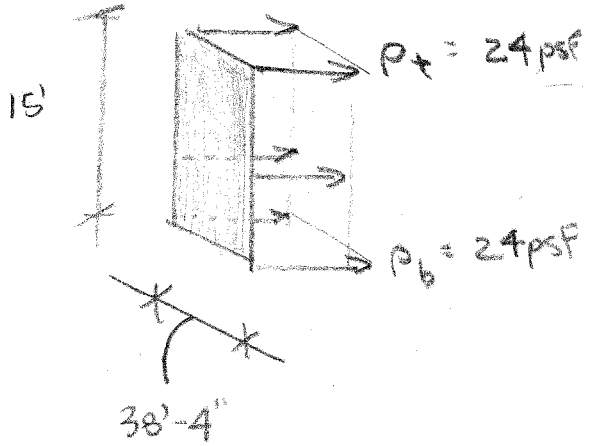
L/B ratio = 52.5/38 = 1.38

⇒ USING TABLE, $p_{top} = p_{bottom} = 24$ psf

WIND LOADING (N/S) CONT.

- SOUTH FACE HAS THE GREATEST WALL AREA.





$F_{wind} = 24psf(15' \times 38.33')$

$F_{wind} = 13,799^k$

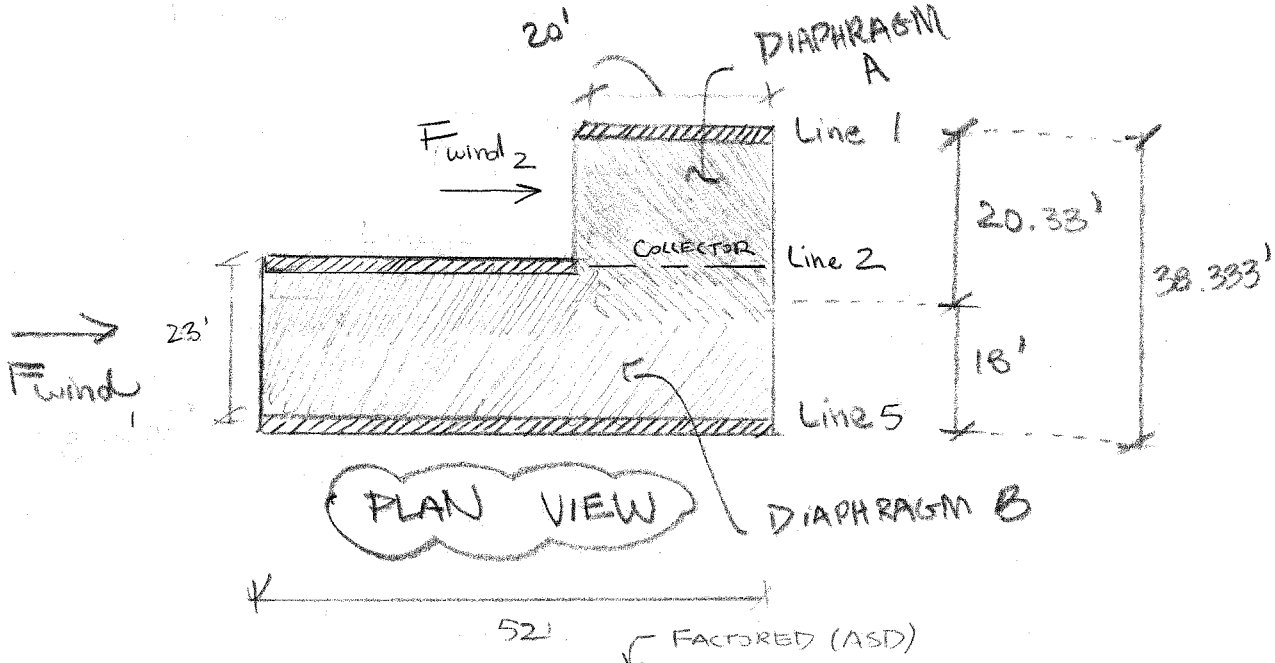
$F_{wind\ factored} = 13,799^k(.06) = \boxed{8,279^k}$

$V_{BASE\ SHEAR} = 3,511.2^k$ (From Spreadsheet)

SINCE $F_{wind} > V_{BASE\ SHEAR}$, WIND GOVERNS IN N/S DIR.

SHEAR WALL LENGTHS: (N/S)

(Flexible Diaphragm)



FACTORED (ASD)

- $F_{wind_1} = 24psf\left(272.97in.^2\right)(0.6) = 3930.77\#$

  → $F_{Line\ 5} = \frac{3930.77\#}{2} = \underline{1965.39\#}$

- $F_{wind_2} = 24psf\left(182.67in.^2\right)(0.6) = 2630.45\#$

  → $F_{Line\ 1} = \frac{2630.45\#}{2} = \underline{1315.22\#}$

  → $F_{Line\ 2} = (1965.39\#) + (1315.22\#) = \underline{3280.61\#}$

- UNIT SHEARS

  Line 5: 2 Shear walls → 8'-0" and 7'-6"

  LOAD TO 8'-0" SHEAR WALL: $\left(\frac{8}{8+7.5}\right)(1965.39\#) = 1014.39\#$

  LOAD TO 7'-6" SHEAR WALL: $\left(\frac{7.5}{8+7.5}\right)(1965.39\#) = 950.995\#$

  8' PLF = $\frac{1014.39\#}{8'} = \underline{126.80\ plf}$

  7.5' PLF = $\frac{950.995\#}{7.5'} = \underline{126.80\ plf}$

- UNIT SHEARS CONT.

LINE 2:

(1) - 8' WALL, 18' high

$PLF = \frac{3280.6\#}{8'} = 410.01\ plf$

ASPECT RATIO (NDS 4.3.4)

$WSP = 1.25 - 0.125\frac{h}{b_s} = 1.25 - 0.125\left(\frac{18'}{8'}\right) = 0.969$ for 18' wall

LINE 1:

(2) - 2' walls

LOAD TO EACH WALL = $\frac{1315.22\#}{2} = 657.61\#$

$\frac{657.61\#}{2'} = 328.81\ plf$

ASPECT RATIO (NDS 4.3.4):

- PROVIDE 1' curb under 2' walls.

$Ratio = \frac{h}{b_s} = \frac{8'-1'}{2'} = 3.5$ (OK)

$WSP = 1.25 - 0.125\frac{h}{b_s} = 1.25 - 0.125(3.5)$

$\Rightarrow WSP = 0.8125$ (ASPECT RATIO FACTOR)

MULTIPLY TABULATED SHEAR CAPACITY BY WSP = 0.8125

SHEAR WALL CAPACITY:

USE 15/32" STRUCT 1 PANELS, 8d nails, 6" Edge Nail Spacing PER NDS TABLE 4.3A (Blocked Shear Walls).

$V_w = \frac{785\ plf}{2} = 392.5\ plf$

(A) WALL

SHEAR WALL CAPACITY CONT.

Line 4: WORST SHEAR = 126.80plf < 392.5plf (OK)

(NOTE: No capacity reduction due to aspect ratos because 8' and 7.5' walls)

Line 2: WORST SHEAR = 410.01 plf

CAPACITY OF 8' wall = 392.5 plf (0.969) = 380.33plf (WSP)

→ 410.01plf > 380.33plf (N.G.) ∴ DECREASE NAIL SPACING TO 4 in.o.c.

→ NEW CAP = 1205plf/2 = 602.5plf > 410.01 plf

Line 1: WORST SHEAR = 328.81 plf

CAPACITY = 392.5plf (0.8125) = 318.91 plf

⇒ 328.81 plf > 318.91 plf (N.G.)

∴ Decrease nail spacing to 4 in.

⇒ NEW CAPACITY = $\frac{1205\text{plf}}{2}$ = 602.5plf

∴ USE 15/32", 8d, 4in Edge Nailing → (B) WALL

V_w = 602.5plf > 328.81 plf (OK)

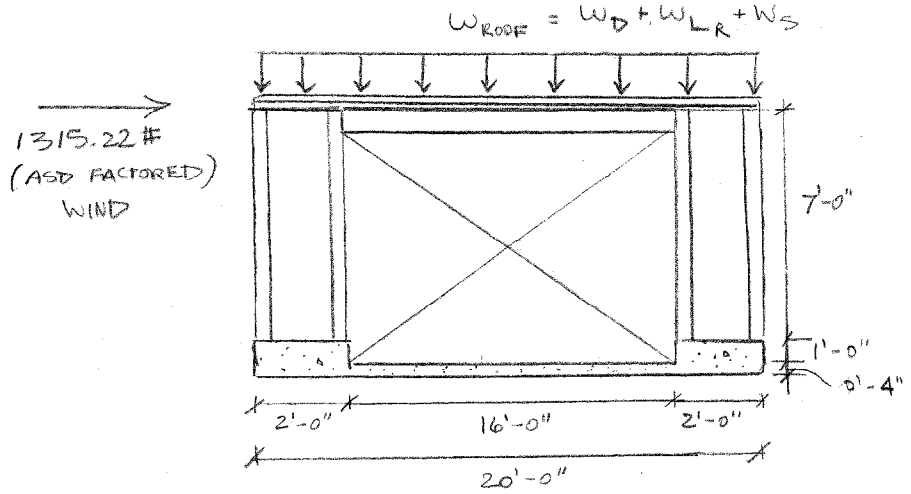
SUMMARY OF SHEAR WALLS (N/S Dir.)

- USE 15/32" STRUCT 1 PANELS, 8d nails, 6 in E.N. Blocked Shear Walls For all walls except 2' walls on Grid (1), and Wall on Grid (2)

- USE 15/32" STRUCT 1 PANELS, 8d nails, 4 in. E.N. BLOCKED FOR 2' walls on Grid (1) w/ 1' curb

OVERTURNING AT SHEAR WALLS (Grid ①)

- GRID 1 SHEAR WALLS (Garage)



- LOAD COMBOS:

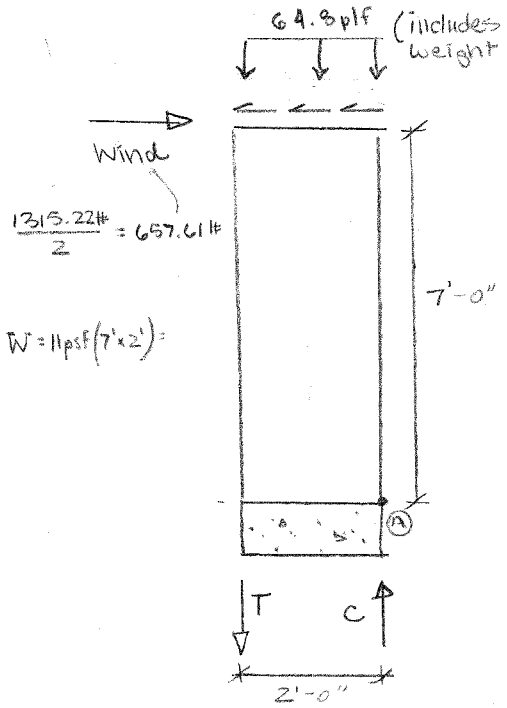
WORST L.C. FOR OVERTURNING IS THE L.C. WITH THE LEAST AMOUNT OF GRAVITY LOADING:

7) $0.6D + 0.6W$

$= 0.6\left(13.5psf \cdot \frac{16'}{2}\right) + 1,315.22\#$

$= 64.8\,plf + 1,315.22\#$

- EQUILIBRIUM:



- $\sum M_{(A)} = 0$:

$64.8plf(2')\left(\frac{2'}{2}\right) - 657.61\#(7') + T(2') = 0$

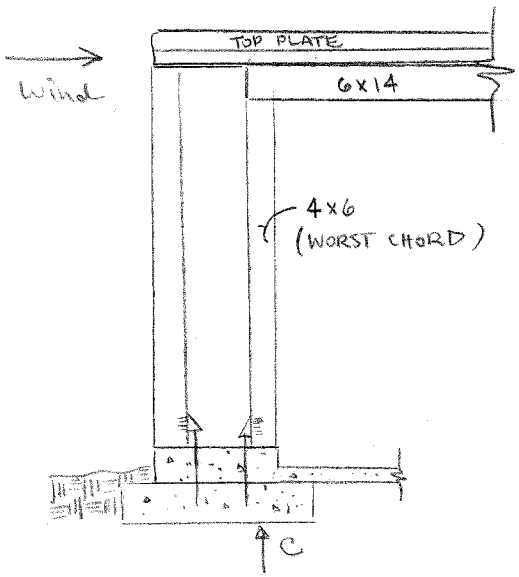
$\Rightarrow$ **$T = 2,236.84\#$**

- CAPACITY OF HDU2 FROM SIMPSON:

HDU2 = 3075 # > 2,237 # (OK)

$\Rightarrow$ **USE (2) HDU2 PER WALL ON GRID 1**

SHEAR WALL CHORD DESIGN (Try 4x6 DF #1)



- LOAD COMBOS: (ASCE 7-10)

3) D + S → $C_D = 1.15$

5) D + 0.6W

6a) D + 0.75(0.6W) + 0.75(S) } → $C_D = 1.6$

- USING L.C. #3:

D+S = 13.5psf + 48psf = 61.5psf

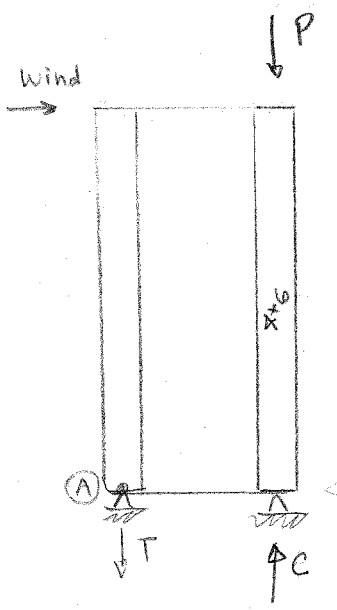
- USING L.C. #5:

D+0.6W = 13.5psf + 657.61#

- USING L.C. #6:

D+0.75(0.6W)+0.75(S) = 13.5psf + (0.75 · 657.61#) + 0.75(48psf)

= 49.5psf + 493.208#



- P = LOAD FROM ROOF FRAMING (FROM 6x14 HDR)
- A_{Trib} for $P = \frac{16'}{2}\left(\frac{20'}{2}\right) = 80\,SF$

$\Sigma M_A = 0$:

$$C(2') - P(2') - W(7') = 0$$

$$\Rightarrow C = \frac{7(\text{wind}) + 2P}{2} = \left(3.5(\text{wind}) + P\right)\#$$

LC #3:

$C = 3.5(0) + (61.5psf \cdot 80SF) = \boxed{4,920\#}$

LC #5:

$C = 3.5(657.61\#) + (13.5psf \cdot 80\,SF) = \boxed{3381.64\#}$

LC #6:

$C = 3.5(493.208\#) + (49.5psf \cdot 80SF) = \boxed{5,686.23\#}$

CHECK AGAINST C_D:

P/C_D:

LC #3: $\frac{4920\#}{1.15} = 4278.26\#$

LC #6: $\frac{5686.23\#}{1.6} = 3553.89\#$

∴ LC #3 GOVERNS

SHEAR WALL CHORD DESIGN CONT. (@ GARAGE, GRID ①)

- CHECK BEARING:

$$F'_{c\perp} = F_{c\perp} C_M C_t C_i C_b = 625psi(1)(1)(1)(1) = 625psi$$

$$f_{c\perp} = \frac{P}{A} = \frac{4{,}920\#}{19.25"} = 255.58\,psi < 625psi \quad (OK)$$

- CHECK COMPRESSION:

PER SIMPSON CATALOG POST CAPACITIES,

8' height, DF #2 4x6 CAPACITY = 20,925# >> 4920# (OK)

NOTE: USING A DF-L #1 4x6 POST, which provides an even greater capacity.

4x6 #1 DF OK

SHEAR TRANSFER (@ GARAGE ROOF., GRID ①)

$$V_{ASD} = \frac{1315.22\#}{20'} = 65.76\,plf$$

From simpson catalog, H3 clip: CAP = 125 #

$$\text{Spacing} = \frac{125\,\#/clip}{65.76plf} = 1.901ft/clip = 22.81"/clip$$

Spacing = 16" o.c. < 22.81"
TO MATCH RAFTER SPACING

→ **USE H3 CLIPS @ 16" O.C. FOR GARAGE RAFTERS**

SHEAR WALL TO FOUNDATION (Garage, GRID 1)

$V_{ASD} = \frac{1315.22\#}{4'} = 328.81\ plf$

→ TABLE 12E NDS ⟹ Choose DF-L, 2x sill plate, 1/2" bolt:

$Z_{11} = 650\ \#$

$Z'_{11} = Z C_D C_m C_t C_g C_\Delta C_{eg}$

$C_D = 1.6$

$Z'_{11} = 650\ \# (1.6) = 1040\ \#$

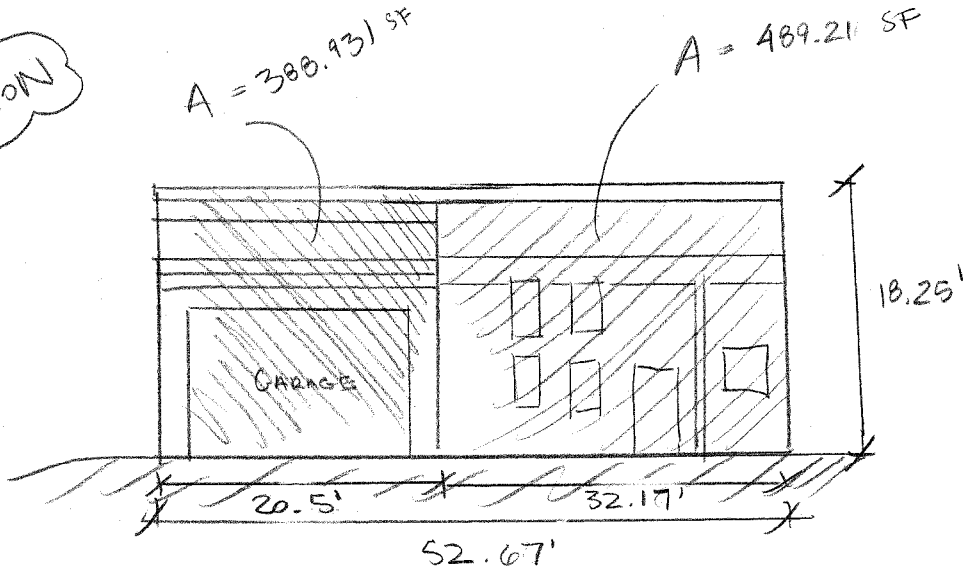
$\frac{1040\ \#/bolt}{328.81\ plf} = 3.16'\ o.c.$

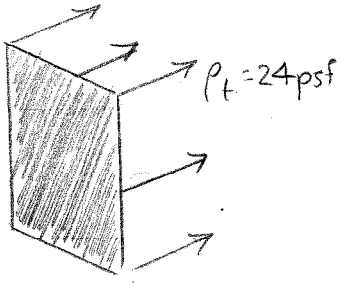
→ 32" BECAUSE STUDS ARE AT 16" O.C.

⟹ USE 1/2" A.B. @ 32" O.C.

WIND LOADING (E/W)

ELEVATION





$F_{wind} = 24psf(52.67')(18.25')$

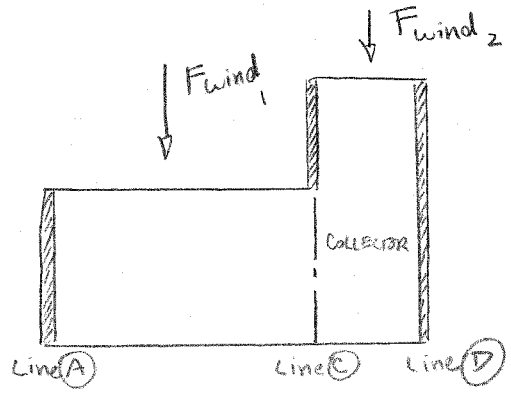
$F_{wind} = 23.0695^K$

$F_{wind_{ASD}} = 23.07^K/0.6 = \boxed{13.84^K}$

$V_{BASE\ SHEAR}$ =

Since $F_{wind} > V_{BASE\ SHEAR}$,
WIND GOVERNS IN E/W Direction

SHEAR WALLS (E/W)



PLAN VIEW

ASD

$F_{wind_1} = 24psf(388.931SF)(0.6) = 5600.61\#$

$\rightarrow F_{line\ (A)} = \frac{5600.61\#}{2} = \underline{2,800.3\#}$

$F_{wind_2} = 24psf(489.21SF)(0.6) = 7044.62\#$

$\rightarrow F_{line\ (D)} = \frac{7044.62\#}{2} = \underline{3522.31\#}$

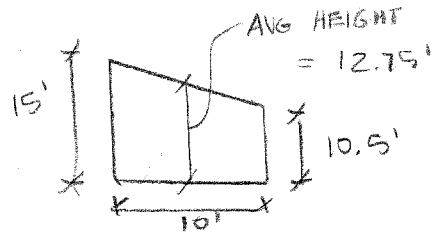
$\rightarrow F_{line\ (C)} = 2,800.3\# + 3522.31\#$

$F_{line\ (C)} = \underline{6,322.62\#}$

UNIT SHEARS: (E/W)

LINE A:

10' length



- LOAD TO SHEAR WALL = $F_{line\,A}$ = 2800.3#
- PLF = $\frac{2800.3}{10'}$ = **280.0 plf**
- ASPECT RATIO (NDS 4.3.4): NOT >2:1, so N/A.

LINE C:

(1) - 15' wall

- LOAD TO WALL = $F_{line\,C}$ = 6,322.62#
- PLF = $\frac{6322.62\#}{15'}$ = **421.52 plf**
- ASPECT RATIO (NDS 4.3.4)

→ N/A because wall length = 15'

LINE D:

(1) - 12'
(1) - 16'
→ Sum = 28'

(12' wall)

- LOAD TO EACH WALL: $F_{line\,D}\left(\frac{L_1}{L_{total}}\right) = 3522.31\#\left(\frac{12'}{28'}\right) = 1509.56\#$

$F_{16'\,wall} = F_{line\,D}\left(\frac{16'}{28'}\right) = 3522.31\#\left(\frac{16'}{28'}\right) = 2012.75\#$

- UNIT SHEAR = $\frac{1509.56\#}{12'}$ = **125.80 plf**
- ASPECT RATIO → N/A

SHEAR WALL CAPACITIES (E/W)

LINE A: WORST SHEAR = 400.05plf

PER SHEAR WALL SCHEDULE:

(A) WALL CAP = 392.5plf < 400.05plf

(B) WALL CAP = 602.5plf (WSP)

= 602.5plf (0.964) = 580.81plf > 400.05plf (OK)

↑ FOR 3.5' WALL

∴ USE (B) WALLS FOR LINE A

LINE C: WORST SHEAR = 421.52plf

↳ SEE LINE A above

421.52plf < (B) WALL CAP = 602.5plf

NO ASPECT RATIO REDUCTION B.C. 15' wall

∴ USE (B) WALLS FOR LINE C

LINE D: WORST SHEAR = 125.80plf

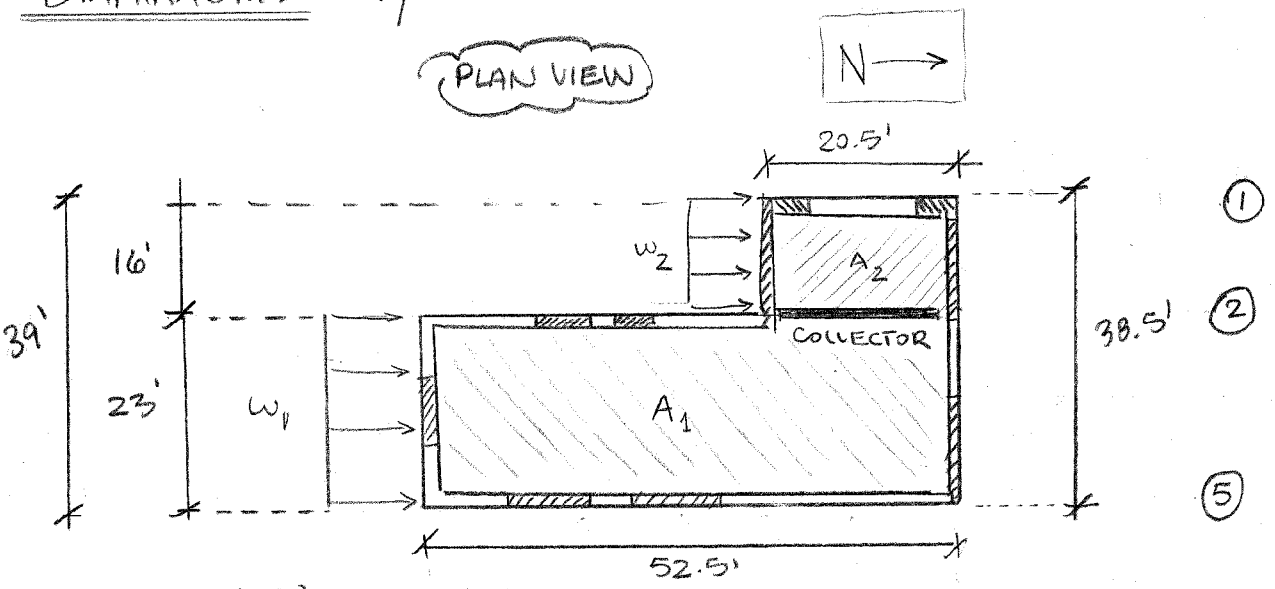
PER SHEAR WALL SCHEDULE:

(A) WALL CAP = 392.5plf > 125.80plf (OK)

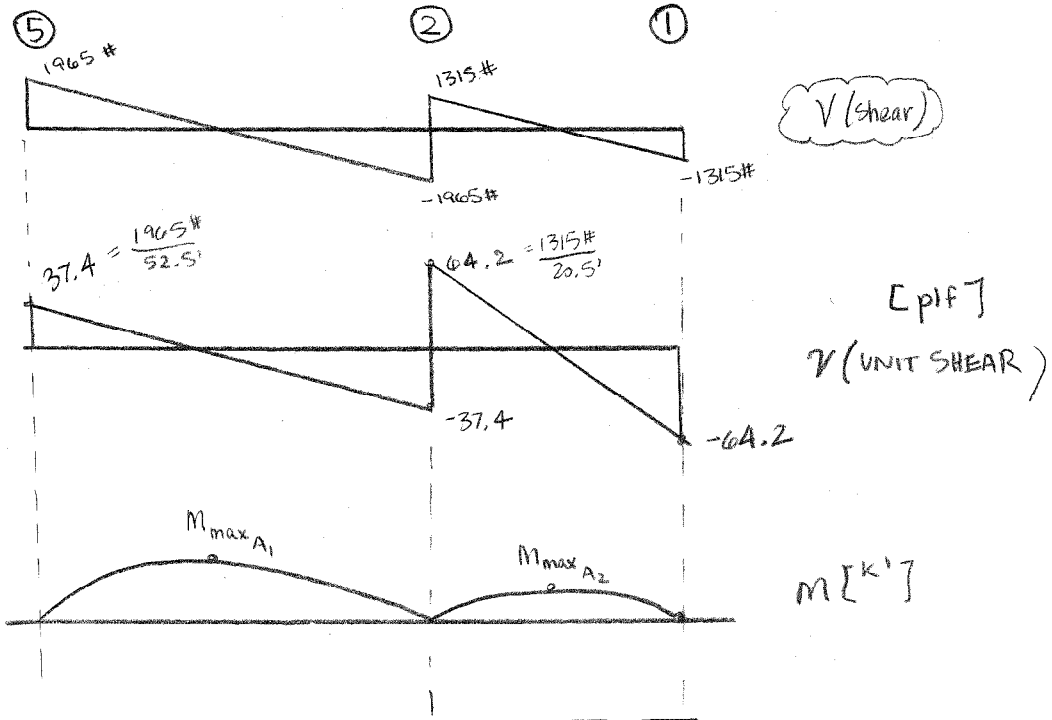
NO ASPECT RATIO REDUCTION B.C. 12' end 16' length walls.

∴ USE (A) WALLS FOR LINE D

DIAPHRAGMS : N/S



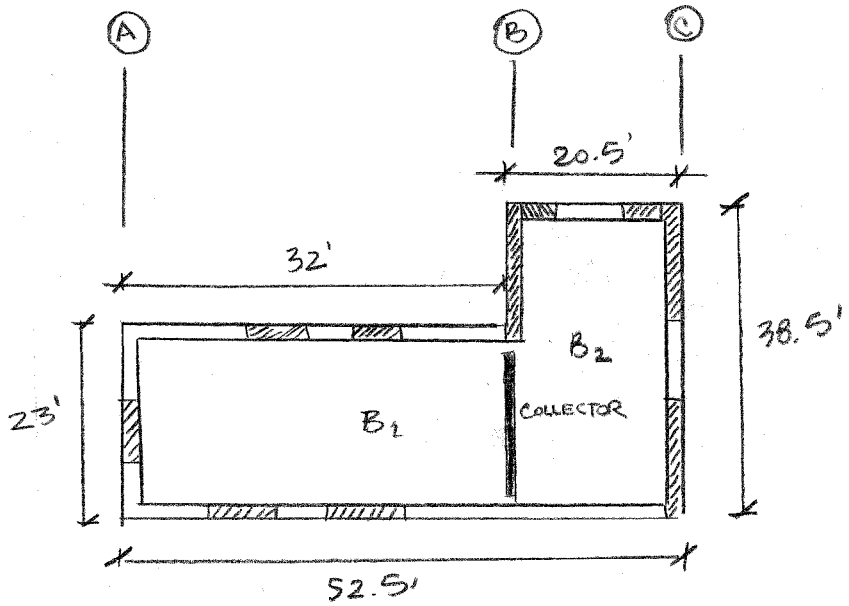
- $w_1 = \frac{3930.77\#}{23'} = \boxed{170.90\ plf}$ (ASD) $\Rightarrow \frac{w_1 L}{2} = \frac{170.90\ plf\,(23')}{2} = 1965.35\#$
- $w_2 = \frac{2630.45\#}{16'} = \boxed{164.40\ plf}$ (ASD) $\Rightarrow \frac{w_2 L}{2} = \frac{164.40\ plf\,(16')}{2} = 1315.22\#$

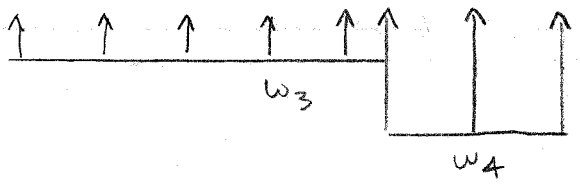


- $M_{max_{A_1}} = 1965\#(23')(0.5)(0.5) = 11{,}300\#' = \boxed{11.3^{K'}}$
- $M_{max_{A_2}} = 1315\#(16')(0.5)(0.5) = 5261\#' = \boxed{5.26^{K'}}$

DIAPHRAGMS E/W

PLAN VIEW



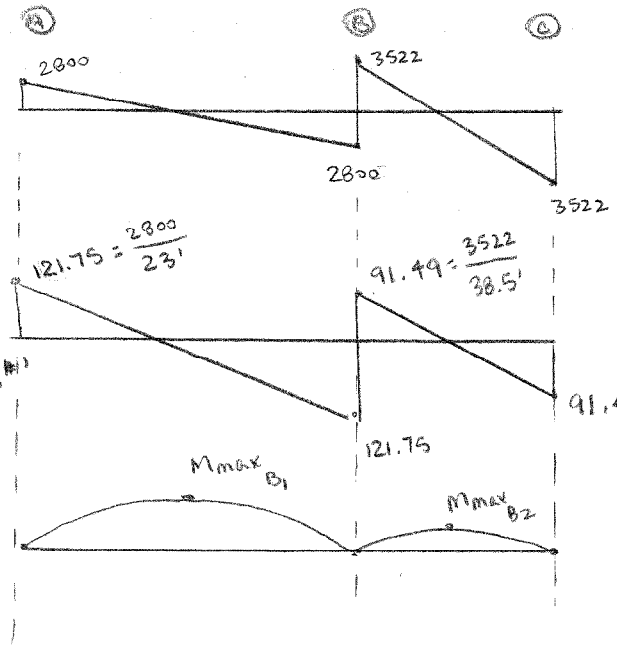


(ASD)

$w_3 = \frac{5600.61\#}{32'} = \boxed{175.02 plf} \Rightarrow \frac{w_3 L}{2} = \frac{175.02 plf\,(32')}{2} = 2800.32\#$

(ASD)

$w_4 = \frac{7044.62\#}{20.5'} = \boxed{343.64 plf} \Rightarrow \frac{w_4 L}{2} = \frac{343.64 plf\,(20.5)}{2} = 3522.31\#$



$M_{max_{B1}} = \frac{2800\#\left(\frac{32'}{2}\right)}{2} = 22,403\#'$

$M_{max_{B1}} = \boxed{22.40 k'}$

$M_{max_{B2}} = \frac{3522\left(\frac{20.5'}{2}\right)}{2}$

$Max_{B2} = \boxed{18.05 k'}$

DIAPHRAGM DESIGN

NOTES: PER TABLE 4.2C (unblocked diaphragms)

- USE 8d Nails, 6in. E.N. and B.N., 12" F.N., (same as shear walls)
- USE 15/32" Panels, Structural 1 (same as shear walls)
- USE min nominal width of 2"
- USE CASE 2/4

$\rightarrow V_w = \frac{505plf}{2} = \boxed{227.5plf}$

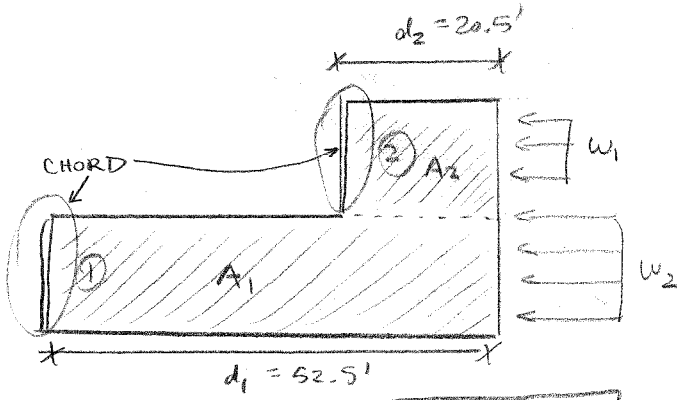
DIAPHRAGM A_1:
Max Diaphragm Shear = 37.4plf
DIAPHRAGM A_2:
Max Diaphragm Shear = 64.2plf
DIAPHRAGM B_1:
Max Diaphragm Shear = 121.75plf
DIAPHRAGM B_2:
Max Diaphragm Shear = 91.49plf

$< 227.5plf$ (OK)

∴ USE 15/32" STRUCT 1 PANELS, UNBLOCKED
8d nails, 6in. o.c. B.N., 6in. o.c. E.N, 12in. o.c. F.N.
FOR ALL ROOF DIAPHRAGMS

DIAPHRAGM CHORD DESIGN (N/S)

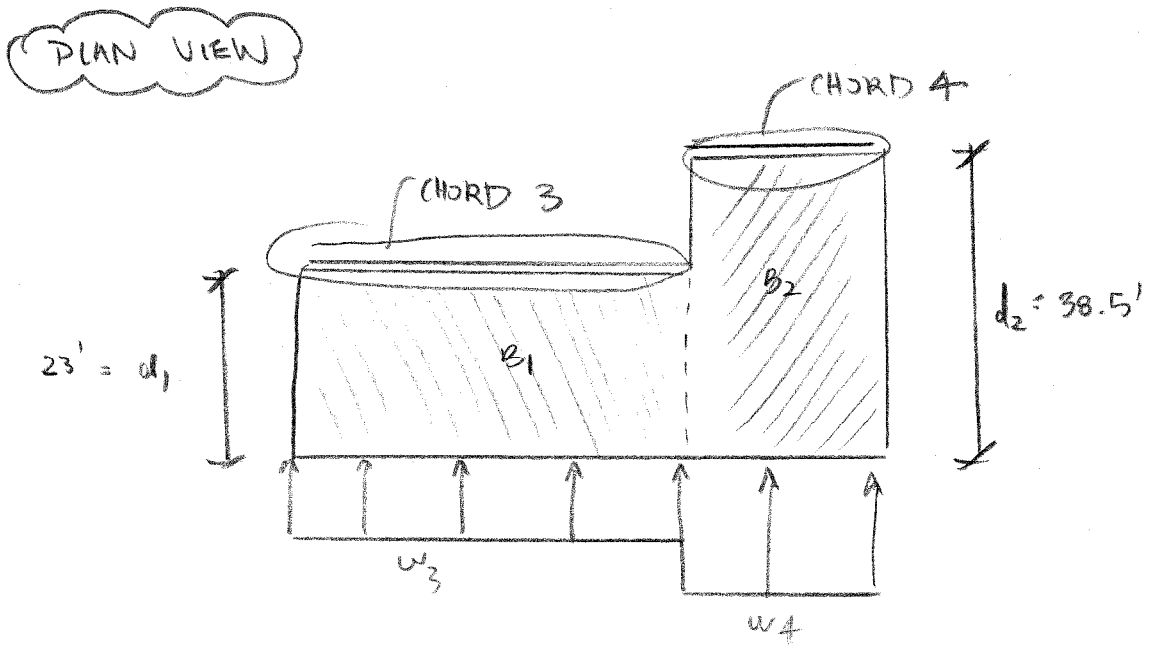
PLAN VIEW



FROM PREV. PAGE:

- $M_{max\,A_1} = 11.3^{k'} \Rightarrow \frac{M_{max}}{d_1} = \frac{11.3^{k'}}{52.5'} = 0.215^k = \boxed{215\# = T_{CHORD\,1}}$
- $M_{max\,A_2} = 5.26^{k'} \Rightarrow \frac{M_{max}}{d_2} = \frac{5.26^{k'}}{20.5'} = 0.257^k = \boxed{257\# = T_{CHORD\,2}}$

DIAPHRAGM CHORD DESIGN (E/W)



FROM PREV. PAGE:

- $M_{MAX\ B_1} = 22.40^{k'} \Rightarrow \frac{M_{max}}{d_1} = \frac{22.40^{k'}}{23'} = 0.974^{k} = \boxed{974\# = T_{CHORD\ 3}}$

- $M_{Max\ B_2} = 18.05^{k'} \Rightarrow \frac{M_{max}}{d_2} = \frac{18.05^{k'}}{38.5'} = 0.469^{k} = \boxed{469\#\ T_{CHORD\ 4}}$

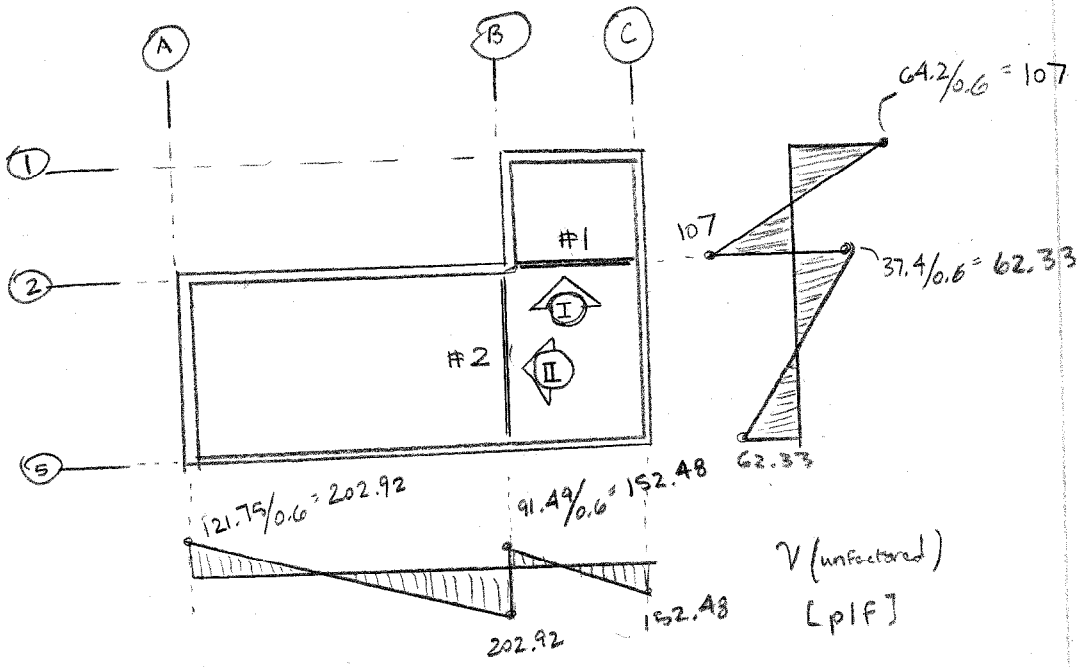
SUMMARY OF CHORD DESIGN:

CRITICAL TENSION = $T_{CHORD\ (3)} = 974\#$ ← Factored, ASD wind.

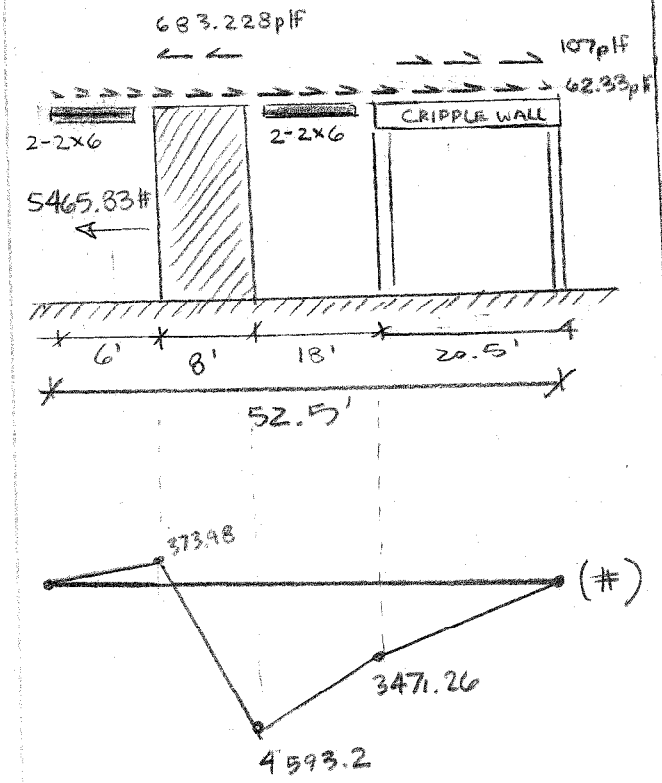
FROM SIMPSON CATALOG, $T_{capacity}$ of a 2x6 #2 w/ 1" bolt = 7960# >> 974#

By inspection, 2-2x6 #2 top plate OK

COLLECTOR DESIGN



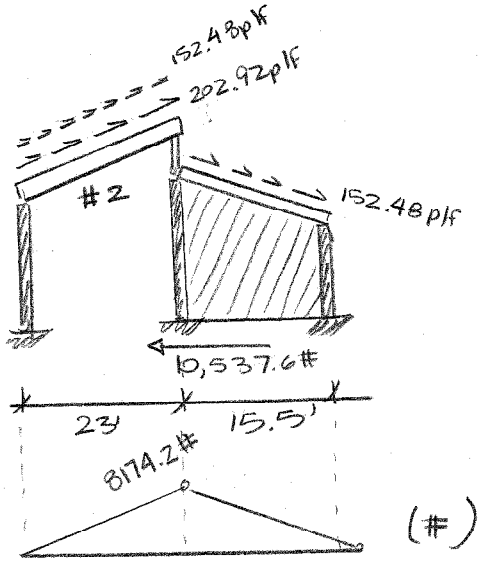
- ELEVATION (I):



AXIAL FORCE DIAGRAM

(UNFACTORED)

- ELEVATION (II):



AXIAL FORCE DIAGRAM

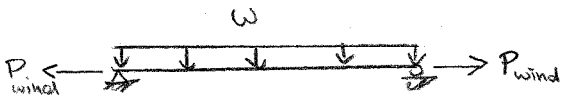
(UNFACTORED)

COLLECTOR DESIGN (#1 FROM PREV. PAGE)

Axial Tension + Bending

Check 6x16 #1 from gravity calculations.

TENSION



- $F'_t = F_t C_D C'_m C'_t C_F C'_i = 675psi(1.6)(0.969) = \underline{1046.52\ psi}$
  - C_D: 1.6
  - F_t: 675psi
  - C_F: $\left(\frac{12}{16}\right)^{1/9} = 0.969$
- $f_t = \frac{2008\#}{74.25in^2} = 27.04psi < 1046.52psi$ (OK)

BENDING

F'_b from Spreadsheet = 1446psi (see calc for RB06)

∴ USE $C_D = 1.6$ NOW: $\frac{1446psi}{1.15}(1.6) = 2011.83psi = F'_b$

(C_D for snow)

Check $\frac{f_t}{F'_t} + \frac{f_b}{F'_b{}^*} < 1.0$

Also from spreadsheet

$\frac{27.04}{1046.52} + \frac{1483psi}{2011.83psi} = \underline{0.76 \leq 1.0\ (OK)}$

Check $\frac{f_b - f_t}{F_b^{**}} \leq 1.0$

$\frac{1483psi - 27.04psi}{2011.83psi} = \underline{0.72 < 1.0\ (OK)}$

6X16 #1 OKAY AS COLLECTOR

COLLECTOR DESIGN (#2 FROM PREV. PAGE)

- AXIAL TENSION + BENDING

Check 2x12 #1

TENSION

- $F'_t = F_t C_D C_M C_t C_F C_i = 675psi(1.6) = \underline{1080\,psi}$

  (F_t: 675, C_D: 1.6, C_M: 1.0)

- $F_t = \frac{(8174.2\#)(0.6)}{16.88in^2} = 290.552 < 1080psi$ (ok)

  (0.6 ← ASD)

BENDING

- SEE RB01 CALCULATION FROM SPREADSHEET

$F'_b = 1323psi\left(\frac{1.6}{1.15}\right) = 1840.7\,psi$

(C_D Factors)

$F_b = 1238psi$ FROM SPREADSHEET.

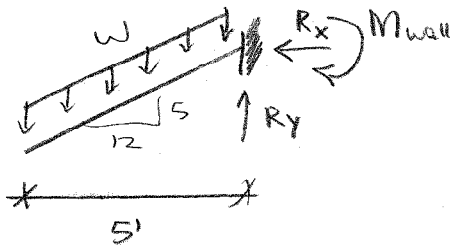
CHECK $\frac{f_t}{F'_t} + \frac{f_b}{F'^{*}_b} = \frac{290.552}{1080psi} + \frac{1238psi}{1840.7psi} = \underline{0.942 < 1.0\ (ok)}$

CHECK $\frac{f_b - f_t}{F_b^{**}} = \frac{1238psi - 290.552}{1840.7psi} = \underline{0.515 < 1.0\ (ok)}$

∴ 2x12 #1 OKAY AS COLLECTOR

USE 2-2x12 #1 SO THAT STRAPS CAN FIT ALONG THE TOP OF THE COLLECTOR.

DETAIL 203 (ENTRANCE OVERHANG)



LOADS

$w = w_D + w_{Lr} + w_S$ (NOTE: Ignoring w_{wind} because it would help by applying negative pressures on the underside of the overhang)

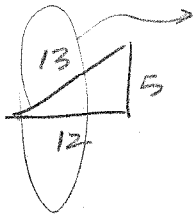
USE LOAD COMBOS FROM ASCE 7-10 (ASD):

2) will govern by observation, which is D+S

$$w_D + w_S = \left[(11.5psf) + (48psf)\right](1.33') = 79.33plf$$

(11.5psf): SEE LOAD TAKE OFF; (1.33'): Trib width

ADjust FOR SLOPE:



Slope Factor $= \frac{13}{12}$

$$\Rightarrow w = 79.33plf\left(\frac{13}{12}\right) = \boxed{85.94plf}$$

$$\Sigma M_{support} = -M_{wall} + w(L)\left(\frac{L}{2}\right) = -M_{wall} + (85.94plf)(5')\left(\frac{5'}{2}\right) = 0$$

$$\Rightarrow \boxed{M_{wall} = 1074.31\,lb'} \text{ (Max Moment)}$$

$$\Sigma F_y = R_y - (wL) = 0$$

$$\Rightarrow R_y = 85.94plf(5') = \boxed{429.72\,lb} \text{ (Max Shear)}$$

CHECK 2x8 #1 @ 16 in. O.C.

DEFLECTION:

- $\Delta_{max} = \frac{wL^4}{8E'I} = \frac{85.94 plf (5')^4 (12''/1')^4}{8(1{,}700{,}000 psi)(47.63 in.^4)} = \boxed{1.720''}$

  $E' = E C_M C_t C_i$ (all = 1)

- $\Delta_{allow_{TL}} = L/60 = \frac{5'(12''/1')}{60} = 1'' < 1.720''$ (N.G.)

  ↑ SEE CBC TABLE 1604.3, Footnote (a), FOR METAL ROOFING

CHECK 2x10 #1 @ 16 in. O.C.

- DEFLECTION: $= \frac{wL^4}{8E'I} = \frac{85.94 plf (5')^4 (12''/1')^4}{8(1{,}700{,}000 psi)(98.93 in.^4)} = 0.828'' < 1''$ (OK)

- BENDING:

  $F'_b = 1000 psi \cdot C_D^{1.15} \cdot C_M \cdot C_t \cdot C_L \cdot C_F \cdot C_{Fu} \cdot C_I^{1.1} \cdot C_r \cdot C_R^{1.15} = 1454.75 psi$

  $f_b = M/S = \frac{1074.31 lb'(12''/1')}{21.39 in.^3} = 602.70 psi < 1454.75 psi$ (OK)

- SHEAR:

  $F'_v = 180 psi \cdot C_D^{1.15} \cdot C_M \cdot C_t \cdot C_i = 207 psi$

  $f_v = 3/2 \frac{V_{max}}{A} = 3/2 \left(\frac{429.72\#}{13.33 in.^2}\right) = 46.44 psi < 207 psi$ (OK)

2x10 #1 @ 16 in. O.C. WORKS

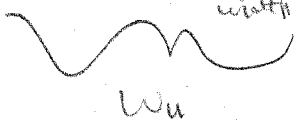
CHECK ANGLE MOMENT CAPACITY REQ.

- Change Loading To LRFD:

$$w_u = 1.2D + 1.6S = 1.2(11.5psf) + 1.6(48psf) = 90.6\ psf$$

$$\left[90.6psf\,(1.333')\left(\frac{13}{12}\right)\right]\left(\frac{L^2}{2}\right) = M_u \qquad L = 5'$$

(1.333' = Trib width; 13/12 = Slope Factor)



$$M_u = 1635.83^{16'}$$

- CHECK YIELDING M_y (Plastic Moment)

$$\phi M_n = (1.5M_y)\phi = M_u = 1.5 Z_x f_y \phi$$

$$\Rightarrow Z_x = \frac{M_u}{1.5 f_y \phi} = \frac{1635.83^{16'}(12^{in}/1')}{1.5(36{,}000psi)0.9} = 0.404\ in.^3$$

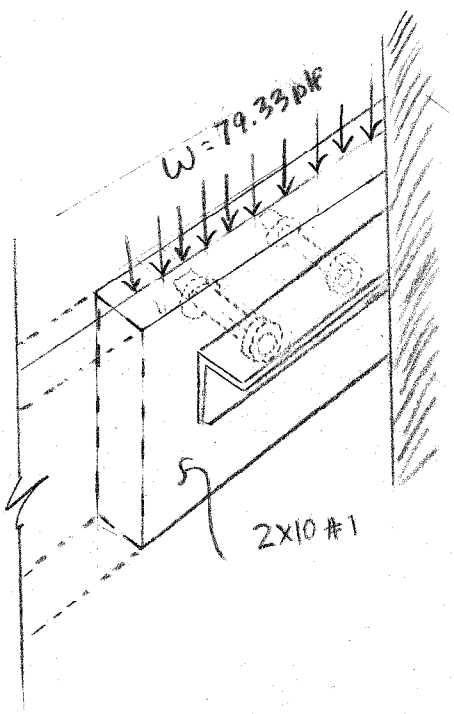
$$L5\times3\times1/4'',\ Z_x = \underline{2.08 in.^3 > 0.404 in.^3}$$

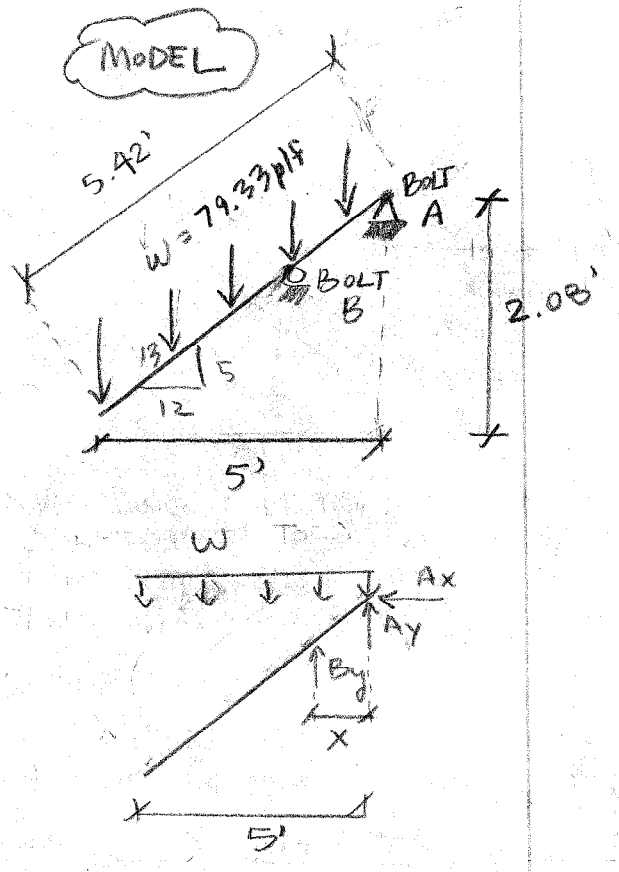
(OK)

- ASSUME LENGTH OF 5' TOO SHORT FOR LTB

- SINCE L5x3x1/4" is compact, don't need to check LOCAL FLG BUCKLING PER AISC CH 16.1, SECTION F10

CHECK BOLTS (DETAIL 203)





Equilibrium:

$\Sigma M_A = 0$:

$$-B_y(x) + wL\left(\frac{L}{2}\right) = 0$$

$$\Rightarrow B_y = \frac{wL^2}{x} \quad \text{where } B_y = \text{SHEAR TO BOLT B}$$

PER NDS, TABLE 12B (single shear):

$Z_\perp = 360\#$ for 5/8" bolt, DF member, 1/4" min steel ✓

$Z'_\perp = Z_\perp C_D C_M C_t C_g C_\Delta C_{eg} C_{Di} C_{tn}$

360 1.15 1 1 1 1 1 1

$Z'_\perp = 360\#(1.15) = 414\# \Rightarrow$ SET AS MAX SHEAR, B_y

$$B_y = \frac{wL^2}{x} \Rightarrow x = \frac{wL^2}{B_y} = \frac{85.94 plf (5')^2}{414\#} = 5.19' \text{ (N.G.)}$$

TRY LARGER BOLT, D = 1":

$Z'_\perp = 510\#(1.15) = 586.5\#$

$$x = \frac{85.94 plf (5')^2}{586.5\#} = 3.66' \text{ (STILL TOO LARGE)}$$

(N.G.)

TRY USING TWO 1" bolts in A ROW (S = 4" o.c.)

$Z'_\perp = Z_\perp C_D C_g$

↳ Per table 11.3.6C (NDS)

A_m = 1.5" x 9.25" = 13.875 in.²

A_s = 2.07 in.² (AISC STEEL MANUAL)

$A_m/A_s = \frac{13.875 in.^2}{1.94} = 7.15 < 12$ (N.G.)

∴ USE Eqn 11.3-1

$$C_g = \left[\frac{m(1-m^{2n})}{n\left[(1+R_{EA}m^n)(1+m)-1+m^{2n}\right]}\right]\left[\frac{1+R_{EA}}{1-m}\right]$$

$\Rightarrow n = 1$

$\Rightarrow \gamma = 270{,}000\,D^{1.5} = 270{,}000(1)^{1.5} = 270{,}000$

$\Rightarrow s = 4"$

$\Rightarrow E_m = 1{,}700{,}000$

$\Rightarrow E_s = 30{,}000{,}000$ psi

$$\Rightarrow u = 1 + \frac{\gamma s}{2}\left[\frac{1}{E_m A_m} + \frac{1}{E_s A_s}\right] = 1 + \frac{270{,}000(4)}{2}\left[\frac{1}{1{,}700{,}000(13.875)} + \frac{1}{30{,}000{,}000(1.94)}\right]$$

$u = 1.03217$

$$\Rightarrow m = u - \sqrt{u^2-1} = 1.032 - \sqrt{1.032^2-1} = 0.776$$

$\Rightarrow R_{EA}$ = LESSER OF { $E_s A_s / E_m A_m = 2.467$; $E_m A_m / E_s A_s = 0.405$ }

$R_{EA} = 0.405$

$$C_g = \left[\frac{0.776(1-0.776^2)}{(1+0.405(0.776))(1+0.776)-1+0.776^2}\right]\left[\frac{1+0.405}{1-0.776}\right]$$

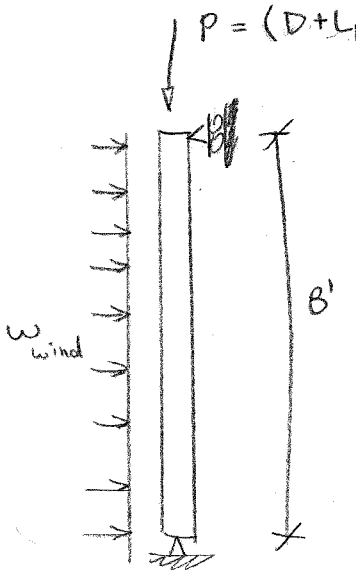
$C_g = 1.0$

$Z'_\perp = 510\#(1.15)(1) = 586.5\#$ PER BOLT

$$\text{MOMENT ARM } X = \frac{85.94\text{plf}\,(5')^2}{2(536.5\#)} = 1.83' \approx 2'$$

∴ USE (2)-1" Diameter Bolts
4D = 4(1") = 4" min. end distance
Spaced 4" apart, 2'-2" from wall and 1'-8" from wall.
(centered @ 2'-0" from wall)

DESIGN OF TYPICAL STUD (NOT FINISHED)



$w_{wind} = 24psf\left(\frac{16''}{12''/1'}\right) = 32plf$

$D_{wall} = 11psf\left(\frac{16''}{12}\right)(8') = 117.33\#$

$D_{roof} = 13.5psf\left(\frac{16''}{12}\right)\left(\frac{18'}{2}\right) = 162\#$

↑ TRIB WIDTH MAX

$L_r = 20psf\left(\frac{16''}{12}\right)\left(\frac{18'}{2}\right) = 240\#$

$S = 48psf\left(\frac{16''}{12}\right)\left(\frac{18'}{2}\right) = 576\#$

- LOAD COMBOS:

3) $D+S = [117.33+162]+[576] = 855.33\#$

6a) $D+0.75L+0.75(0.6W)+0.75(S)$

- CHECK 3): COMPRESSION

855.33# Compression < 8970 # ← CAPACITY OF 8' 2X6 DF #2 PER SIMPSON STRONGTIE CATALOG. BEARING ON 2X6 SILL PLATE FAILURE. WORST CONDITION.

(OK)

GENERAL NOTES

General

1. *Applicable Code:* 2016 California Building Code (CBC).

   A. Design Wind Speed (ASCE 7-10): 110 mph, Exposure C.
   B. Design Seismic Criteria (ASCE 7-10):

Seismic Importance Factor I:	1.0
Short Period MCE Acceleration SS:	0.736
Long Period MCE Acceleration S1:	0.328
Response Modification Coefficient R:	6.5
Soil Profile Type:	D

2. *Governing Code Authority:* Weed CA Building Code.

3. *Design Intent:* Contract documents indicate information sufficient to convey design intent. Review contract documents and verify field and existing conditions. Promptly notify Structural Engineer, prior to proceeding with work, if design intent requires further clarification.

4. *Submittals:* Review for completeness and compliance with contract documents prior to submission to Structural Engineer. Submit prior to fabrication. Submittal review is for general conformance with design intent and does not constitute an authorization to deviate from terms and conditions of contract. When indicated, provide a professional engineer's signature and seal applicable to state where project is located. Maintain at site a copy of reviewed and accepted submittals

5. *Modifications and Substitutions:* Must be accepted in writing by Structural Engineer. No modification or substitution will be accepted via shop drawing review.

6. *Contract Documents Use:* Perform structural related work and develop shop drawings considering contract documents in their entirety. See architectural drawings for top of floor and roof elevations, depressions, slopes, openings, curbs, drains, trenches, slab edge locations, wall overall dimensions and locations of openings not indicated on structural drawings. Any discrepancies between architectural and structural dimensions should be confirmed with the Structural Engineer before starting work.

7. *Construction Means and Methods:* Not a part of contract documents. Perform construction means, methods, techniques, sequences and procedures complying with national, state and local safety ordinances. Site visits (including structural observation) by Structural Engineer do not constitute supervision of construction means and methods.

8. *Typical Details:* Details titled as "Typical" are applicable throughout project and may not be specifically referenced herein. Contractor is responsible for identifying these details and understanding extent of their application prior to performing work.

Reinforcing Steel

1. *Reinforcing Steel:*

All bars unless indicated otherwise	ASTM A615, Grade 60
Bars to be welded	ASTM A706, Grade 60

2. *Wire Reinforcing:*

Smooth welded wire fabric	ASTM A185
Deformed wire stirrups (D4 and larger only)	ASTM A497

3. *Lap Lengths:* As shown on drawings. If lap lengths cannot be determined, verify with Structural Engineer. Lap wire fabric 1-1/2 spaces (1 foot minimum).

4. *Minimum Cast-in-Place Concrete Cover:*

	Min. cover, in.
Slabs on Grade	center of slab
(a) Concrete Exposed to Earth or Weather (Unformed)	3

5. *Chairs or Spacers:* Plastic or plastic coated when resting on exposed surfaces.

6. *Bending:* Bend cold unless otherwise accepted by Architect (Structural Engineer). Do not field-bend reinforcing steel bars embedded in concrete unless otherwise shown on contract documents or pre-approved by Structural Engineer.

Cast-In-Place Concrete

1. *Applicable Standard:* ACI 301.

2. *Portland Cement:* ASTM C150, Type II.

3. *Normal Weight Concrete (145 pcf):* ASTM C33 for aggregates of natural sand and rock. Concrete to attain the following 28-day minimum compressive strength (f_c), of 2000 PSI.

   Maximum Aggregate Sizes: 1-1/2 inches at foundations and slabs on grade and 1 inch elsewhere.

4. *Lean Concrete:* Where specifically indicated, containing 2 sacks of cement per cubic yard of concrete.

5. *Maximum Slump:* 5 inches. 4 inches in flatwork.

6. *Shrinkage:* ASTM C157, limit to 0.055 percent.

7. *Use of Chlorides:* Not permitted.

8. *Construction Joints:* Provide keys unless detailed otherwise. Roughen surface to ¼ inch amplitude. Thoroughly clean, remove laitance and thoroughly wet and remove standing water before placing new concrete.

9. *Concrete Abutting Structural Masonry Walls:* Roughen concrete surface to full amplitude of 1/16 inch.

10. *Curing:* Maintain concrete above 50 degrees Fahrenheit and in a moist condition for a minimum of 7 days after placement unless otherwise accepted by Structural Engineer.

Earthwork and Foundations

1. *Design Assumption:* In lieu of more detailed soils information, existing subgrade is assumed to be class "5" in compliance with CBC Table 1806.2 with allowable bearing pressure of 1,500 psf.

   *These values may be increased 33 percent for seismic or wind loading

2. *Excavations, Backfill and Compaction of Backfill:* Comply with requirements of CBC Section 1804. Contractor is responsible for all excavation, lagging, shoring, underpinning and related procedures.

3. *Minimum Footing Depths:* 12 inches below adjacent grade (excluding landscaping soil) or finish floor, whichever is lower.

4. *Water Exposure at Building Perimeter Footings:* At areas where sidewalks or paving do not immediately adjoin structure, provide positive drainage away from structure at building perimeter. Landscape irrigation is not permitted within five feet of building perimeter footings except when enclosed in protected planters with direct drainage away from structure or which complies with applicable code. Discharge from downspouts, roof drains and scuppers is not permitted onto unprotected soils within five feet of building perimeter. Refer to geotechnical report for complete requirements.

Masonry

1. *Specified Compressive Strength of Masonry (f'_m):* 1500 psi.

2. *Concrete Block:* ASTM C90, medium weight, Grade N-I attaining a minimum compressive strength as required to meet specified compressive strength of masonry (f'_m).

5. *Mortar:* ASTM C270, Type S conforming with CBC Section 2103.2 and attaining a minimum compressive strength at 28 days of 1800 psi. Do not use masonry cement or plastic cement.

3. *Grout:* ASTM C476 or CBC Section 2103.3 attaining a minimum compressive strength as required to meet specified compressive strength of masonry (f'_m). However, in no case shall grout compressive strength be less than 2000 psi at 28 days.

4. *Portland Cement:* Cast-in-place concrete section of general notes.

5. *Aggregates for Mortar and Grout:* ASTM C144 and C404 of natural sand and rock.

6. *Reinforcing Steel:* Reinforcing steel section of general notes unless indicated otherwise.

7. *Reinforcing Steel Splices:* Lap reinforcing steel at splices a minimum of 72 bar diameters, unless noted otherwise. Where clear distance between bars at adjacent splices is 3 inches or less, increase lap length 30 percent unless splices are staggered at least 24 bar diameters.

8. *Placement:* Set cells in vertical alignment.

9. *Grouting:* Grout solid all cells. Mechanically vibrate grout in cells.

   A. Horizontal Construction Joints: Hold grout 1 1/2 inches below top of masonry unit if work is stopped one hour or longer.
   B. Grout Cover Around Reinforcing Steel, Anchor Bolts and Inserts Penetrating Masonry Shell: 1 inch minimum

10.

11. *Typical Details:* Details titled as "Typical" are applicable throughout project and may not be specifically referenced herein. Contractor is responsible for identifying these details and understanding extent of their application prior to performing work.

Nailing Schedule (Portion of CBC Table 2304.10.1)

All nails are common nails unless written acceptance by Architect (Structural Engineer) is attained.

1. Joist to sill or girder, toenail 3-8d
2. Bridging to joist, toe nail each end 2-8d
3. 1"x6" subfloor or less to each joist, face nail 2-8d
4. Wider than 1"x6" subfloor to each joist, face nail 3-8d
5. 2" subfloor to joist or girder, blind and face nail 2-16d
6. Sole plate to joist or blocking, typical face nail 16d @ 16" o/c
7. Sole plate to joist or blocking, at braced wall panels 3-16d per 16"
8. Top plate to stud, end nail 2-16d
9. Stud to sole plate 4-8d, toe nail or 2-16d, end nail
10. Double studs, face nail 16d @ 24" o/c
11. Doubled top plates, typical face nail 16d @ 16" o/c
12. Doubled top plates, lap splice 8-16d
13. Blocking between joists or rafters to top plate, toe nail 3-8d
14. Rim joist to top plate, toe nail 8d @ 6" o/c
15. Top plates, laps and intersections, face nail 2-16d
16. Continuous header, two pieces 16d @ 16" o/c along each edge
17. Ceiling joist to plate, toe nail 3-8d
18. Continuous header to stud, toe nail 4-8d
19. Ceiling joists, laps over partitions, face nail 3-16d
20. Ceiling joists to parallel rafters, face nail 3-16d
21. Rafter to plate, toe nail 3-8d
22. 1" brace to each stud and plate, face nail 2-8d
23. 1"x8" sheathing or less to each bearing, face nail 2-8d
24. Wider than 1"x8" sheathing to each bearing, face nail 3-8d
25. 2" planks 2-16d at each bearing
26. Built-up corner studs 16d @ 24" o/c
27. Built-up girder and beams 20d @ 32" o/c at top and bottom and staggered 2-20d at ends and at each splice

Rough Carpentry

1. *Structural Lumber:* Grade marked Douglas Fir-Larch structural lumber complying with Standard Grading Rules No. 17 of the West Coast Lumber Inspection Bureau. Provide air-dry lumber with 19 percent maximum moisture content.

2. *Classifications and Grades:*

Member	*Grade*
2x and 4x Joists and Rafters UNO	Douglas Fir #2
4x Beams, Headers and Stringers UNO	Douglas Fir #2
6x or larger Beams, Headers and Stringers UNO	Douglas Fir #1
4x Posts	Douglas Fir #1
Posts Larger Than 4x	Douglas Fir #1
Studs, Plates and Blocking	Stud Grade

3. *Plywood:* U.S. Product Standard PS x-xx and classified as Exposure 1. Each sheet of plywood shall be identified with appropriate trademark of the American Plywood Association.

4. *Pressure Treat Structural Lumber Bearing on Concrete or Masonry:* See specifications. Provide hot dipped galvanized or stainless steel fasteners and hardware connectors at pressure treated structural lumber.

5. *Nails:* Common nails with dimensional properties complying with CBC 2304. Install nails in compliance with CBC Chapter 23, including Table 2304.10.1

6. *Bolts:* ASTM A307 bolts with standard cut washer under bolt head and nut. Provide holes for bolts 1/32 to 1/16 inch larger than nominal bolt diameter. Re-tighten bolts prior to application of sheathing or finish.

7. *Lag Screws:* ANSI/ASME Standard B18.2.1 for lag screw dimensions. Pre-drill all holes. Hole at shank portion to match diameter of shank. Holes at threaded portion to be 60 to 75 percent of shank diameter and equal to length of threaded portion. Use soap and lubricants to facilitate installation. Driving with hammer is not permitted.

8. *Plate Washers:* Provide under heads or nuts of bolts (including anchor bolts at sill plates) and lag screws of the following sizes when anchoring wood:

1/2" diameter	1/8"x2" sq.	5/8" diameter	1/8"x2-1/2" sq.
3/4" diameter	3/16"x2-3/4" sq.	7/8" diameter	1/4"x3" sq.
1" diameter	5/16"x3-1/2" sq.		

9. *Wood Hardware Connectors:* Manufactured by Simpson Strong-Tie Company, Inc.

10. *Notching or Cutting Structural Lumber:* Not permitted unless specifically detailed or indicated.

11. *Lateral Support for Beams, Rafters and Joists:* CBC Section 2308.4.6.

12. *Wood Studs:*

   A. Top Plate: construct with 2 pieces same width as studs. Splice as indicated.
   B. Stud Wall Bracing in Stud Walls not Plywood Sheathed: Compliance with CBC Section 2308.6
   C. Fire Blocks: CBC Section 706.
   D. Notching or Boring Holes in Wood Studs: CBC Section 2308.5.9. and 2308.5.10.
   E. Partition Support at Floor Framing: Double joists under partitions which are parallel to joists and provide solid full depth blocking under partitions which are perpendicular to joists.

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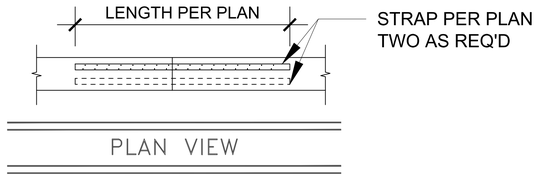
GENERAL NOTES

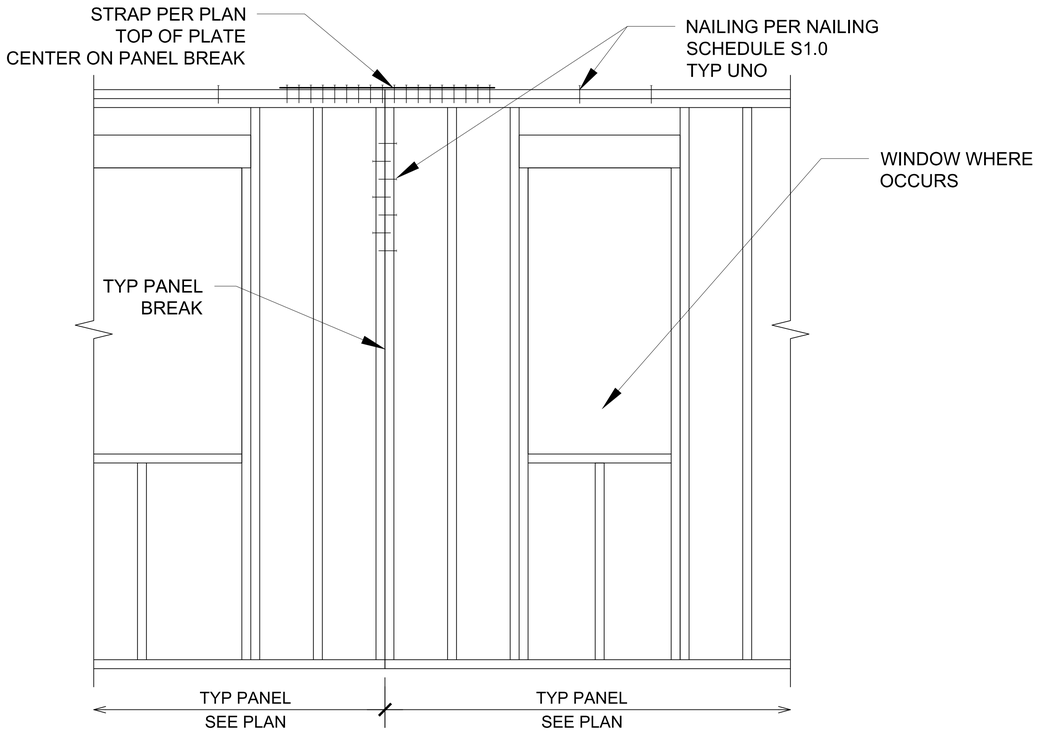
CAPSTONE CLASS
ARCE 415

DATE 12/1/17
SCALE 1" = 1'-0"
DRAWN R.d.S
JOB CAPSTONE
SHEET

S1.0

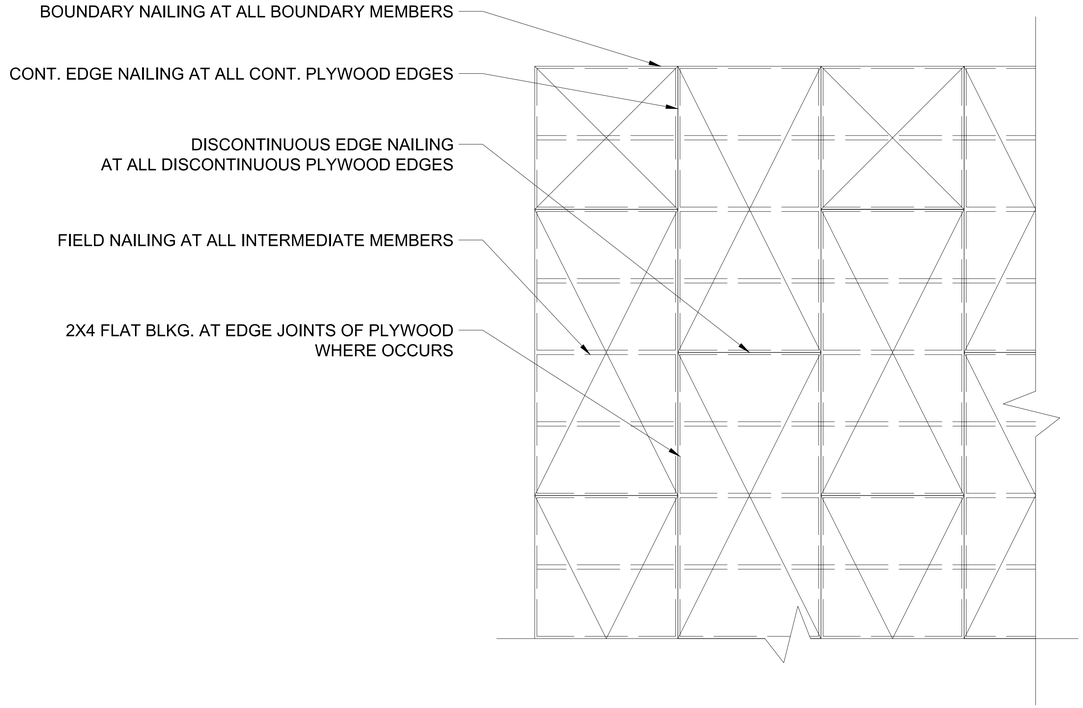
OF **6** SHEETS





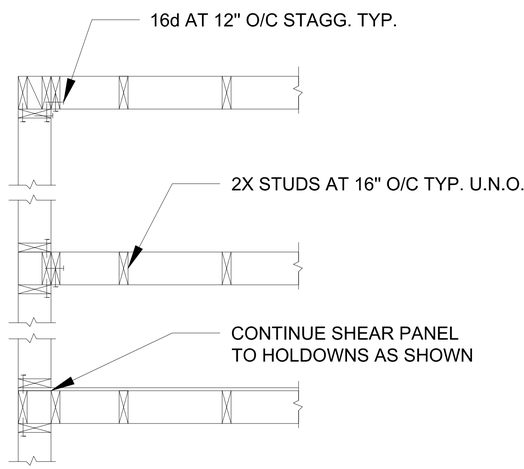
3 STRAP @ TYP WALL PANEL

SCALE: 1/2"=1'-0"



2 TYP ROOF OR WALL PLYWOOD

SCALE: 1/2"=1'-0"



1 WALL INTERSECTIONS

SCALE: 1"=1'-0"

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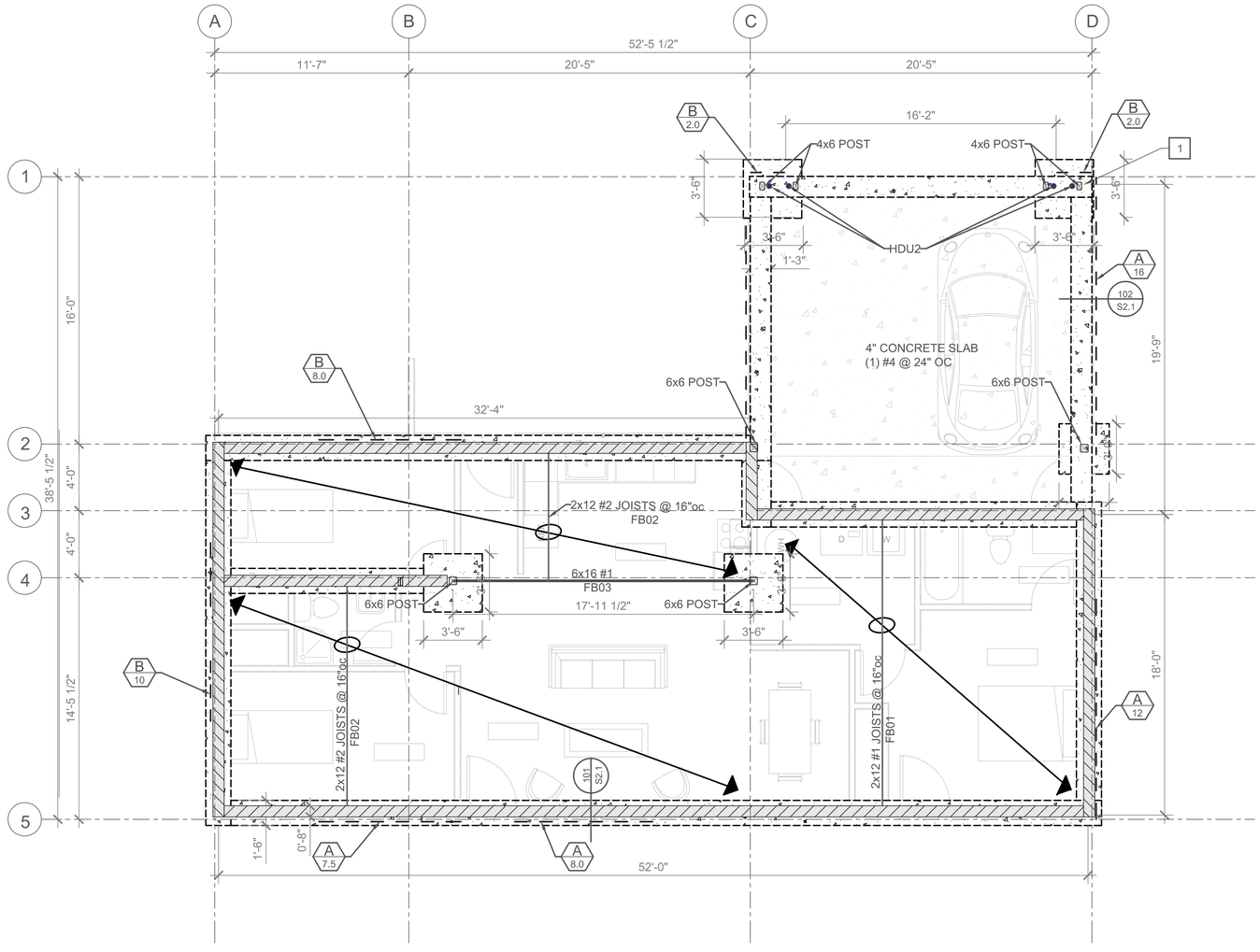
TYPICAL DETAILS

CAPSTONE CLASS
ARCE 415

DATE 12/1/17
SCALE 1" = 1'-0"
DRAWN R.d.S
JOB CAPSTONE
SHEET

S1.1

OF 7
SHEETS



N

FLOOR AND FOUNDATION PLAN

SCALE: 1/4" = 1'-0"

SHEAR WALL SCHEDULE:

- A: $15/32$" STRUCT. 1 PANEL, 8d @ 6" O.C. E.N., BLOCKED
- B: $15/32$" STRUCT. 1 PANEL, 8d @ 4" O.C. E.N., BLOCKED

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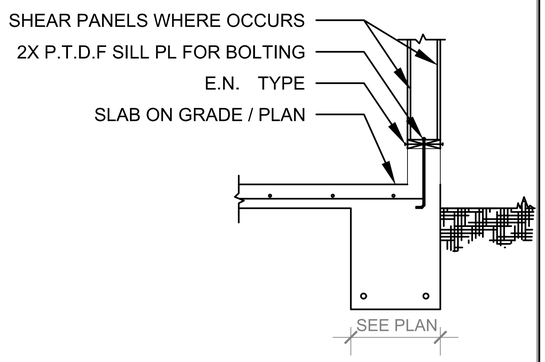
FLOOR FRAMING AND FOUNDATION PLAN

CAPSTONE CLASS
ARCE 415

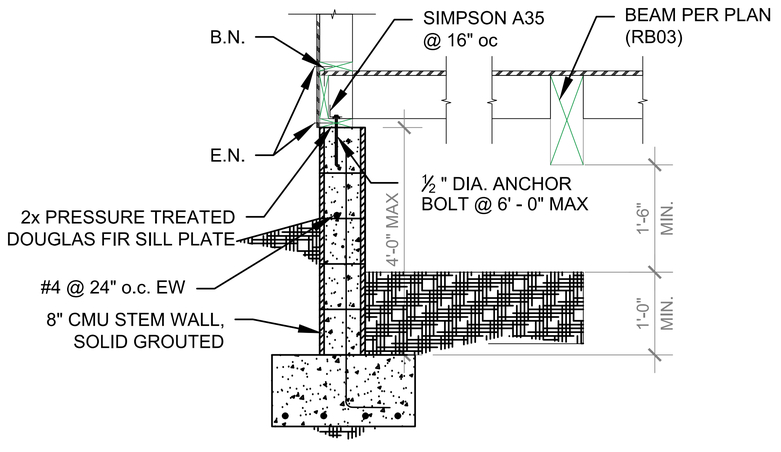
DATE 12/1/17
SCALE 1/4" = 1'-0"
DRAWN R.d.S
JOB CAPSTONE
SHEET S2

OF 6 SHEETS



102 FOUNDATION AT GARAGE

SCALE: 1"=1'-0"



101 FOUNDATION TO FLOOR

SCALE: 1"=1'-0"

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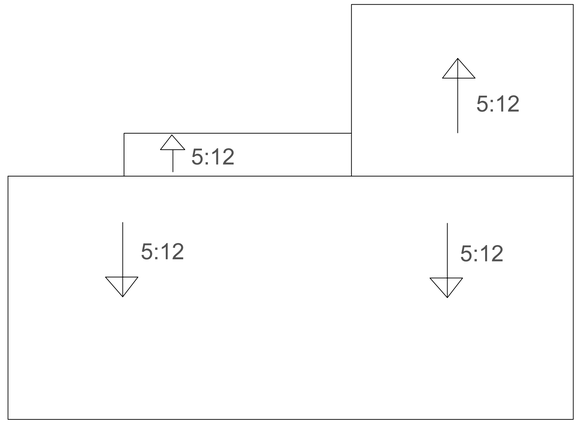
FOUNDATION AND FLOOR DETAILS

CAPSTONE CLASS
ARCE 415

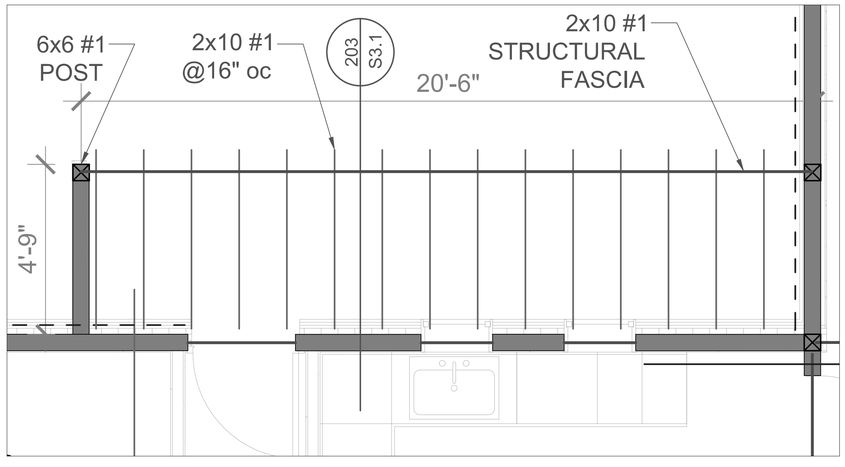
DATE 12/1/17
SCALE 1" = 1'-0"
DRAWN R.d.S
JOB CAPSTONE
SHEET

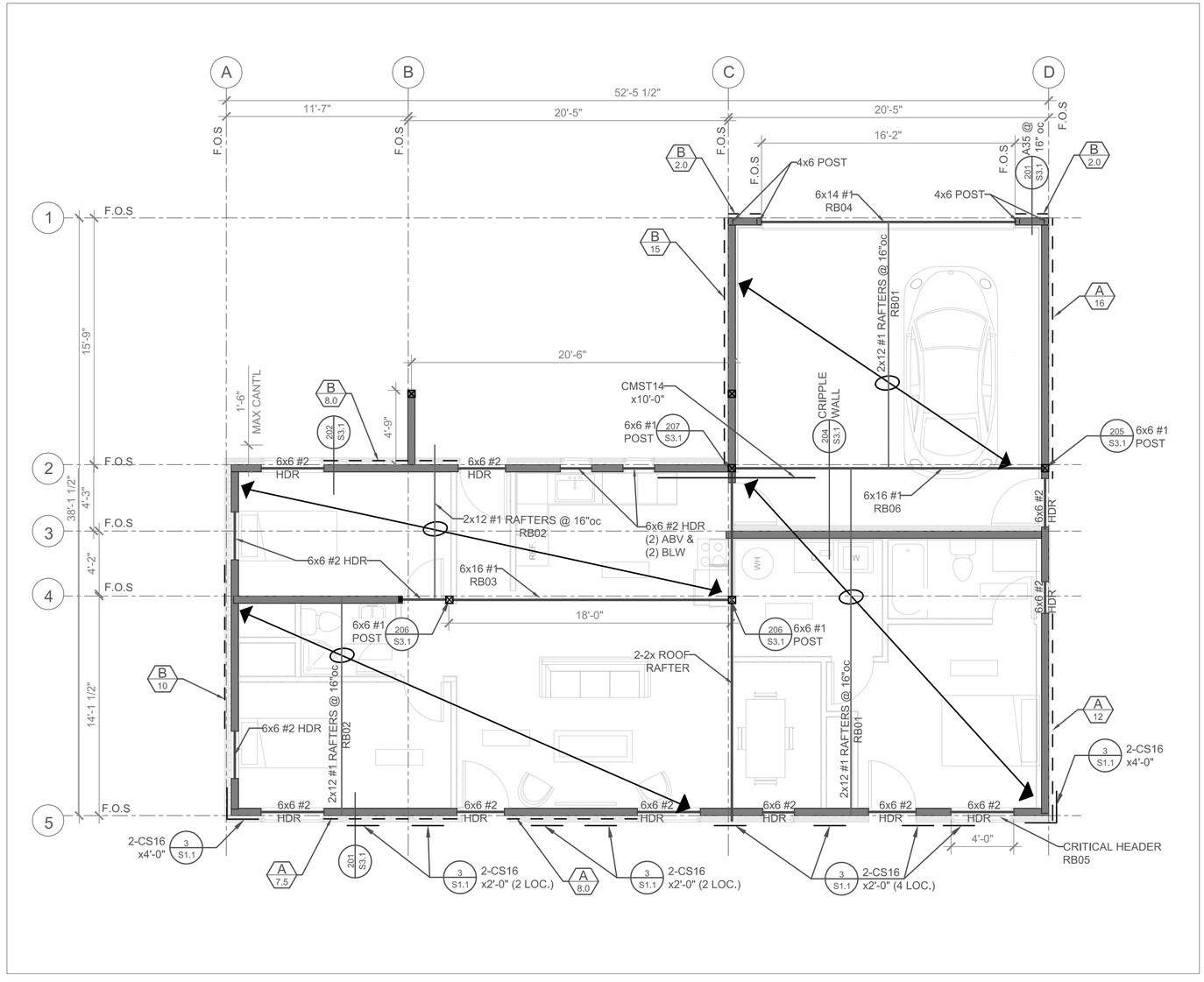
S2.1

OF 6 SHEETS



ROOF SLOPES
NO SCALE





N

ENTRANCE FRAMING PLAN
SCALE: 1/2" = 1'-0"

N

ROOF FRAMING PLAN
SCALE: 1/4" = 1'-0"

SHEAR WALL SCHEDULE:

- A: 15/32" STRUCT. 1 PANEL, 8d @ 6" O.C. E.N., BLOCKED
- B: 15/32" STRUCT. 1 PANEL, 8d @ 4" O.C. E.N., BLOCKED

DIAPHRAGM NAILING:

15/32" STRUCT. 1 PANEL, 8d @ 6" O.C. E.N., UNBLOCKED
(ALL ROOF DIAPHRAGMS)

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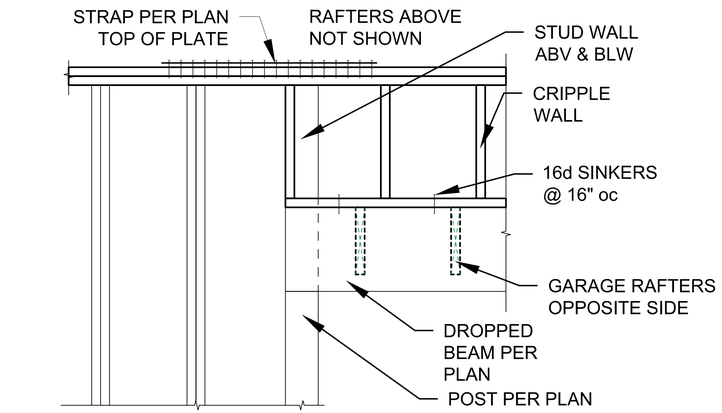
ROOF FRAMING PLAN

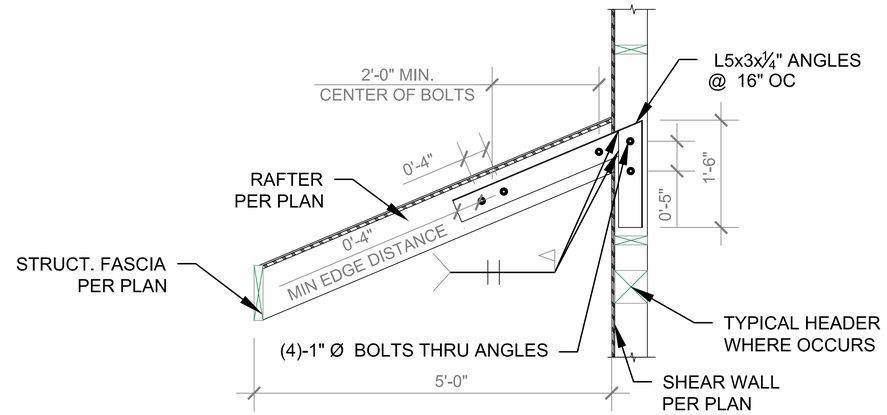
CAPSTONE CLASS
ARCE 415

DATE 12/1/17
SCALE 1/4" = 1'-0"
DRAWN R.d.S
JOB CAPSTONE
SHEET S3
OF 6 SHEETS



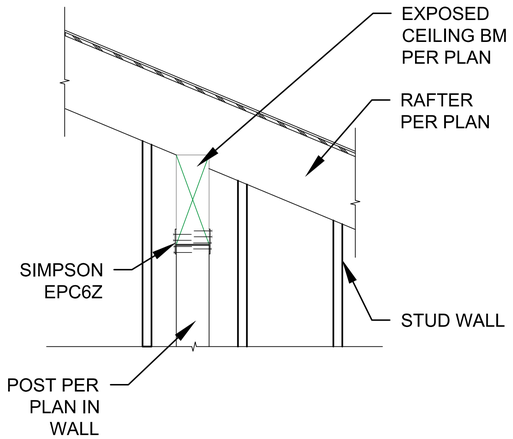
207 POST UNDER CRIPPLE @ STRAP

SCALE: 1"=1'-0"



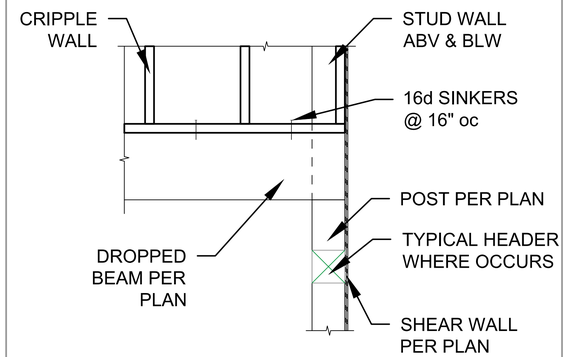
203 ENTRANCE OVERHANG

SCALE: 1"=1'-0"



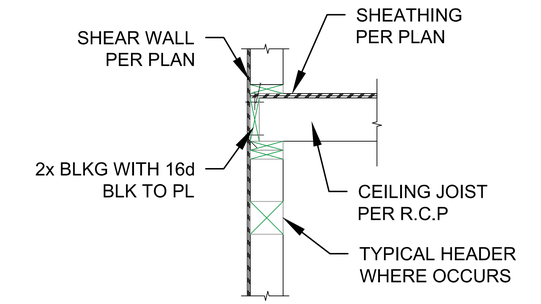
206 CEILING BEAM @ POST

SCALE: 1"=1'-0"



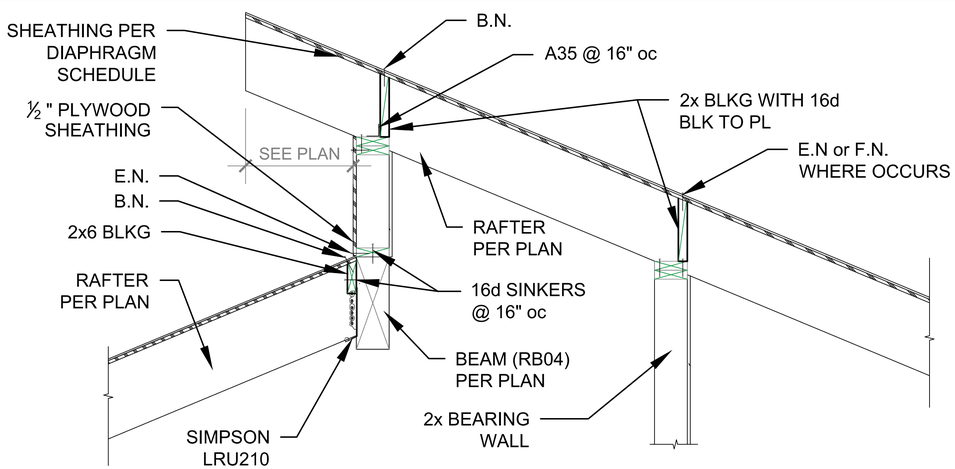
205 POST @ CRIPPLE WALL END

SCALE: 1"=1'-0"



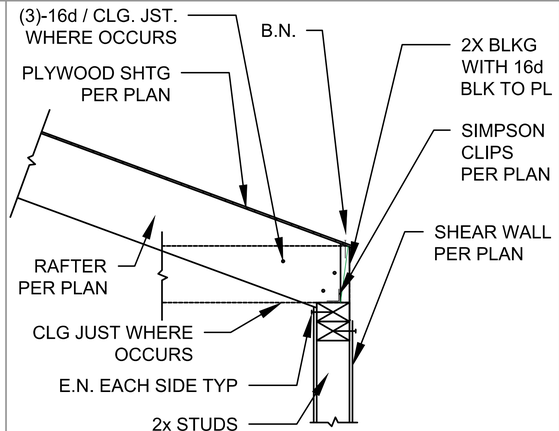
202 CEILING JOIST TO WALL

SCALE: 1"=1'-0"



204 CRIPPLE WALL @ GARAGE

SCALE: 1"=1'-0"



201 ROOF RAFTER TO WALL

SCALE: 1"=1'-0"

RORY DE SEVILLA
rdesevil@calpoly.edu

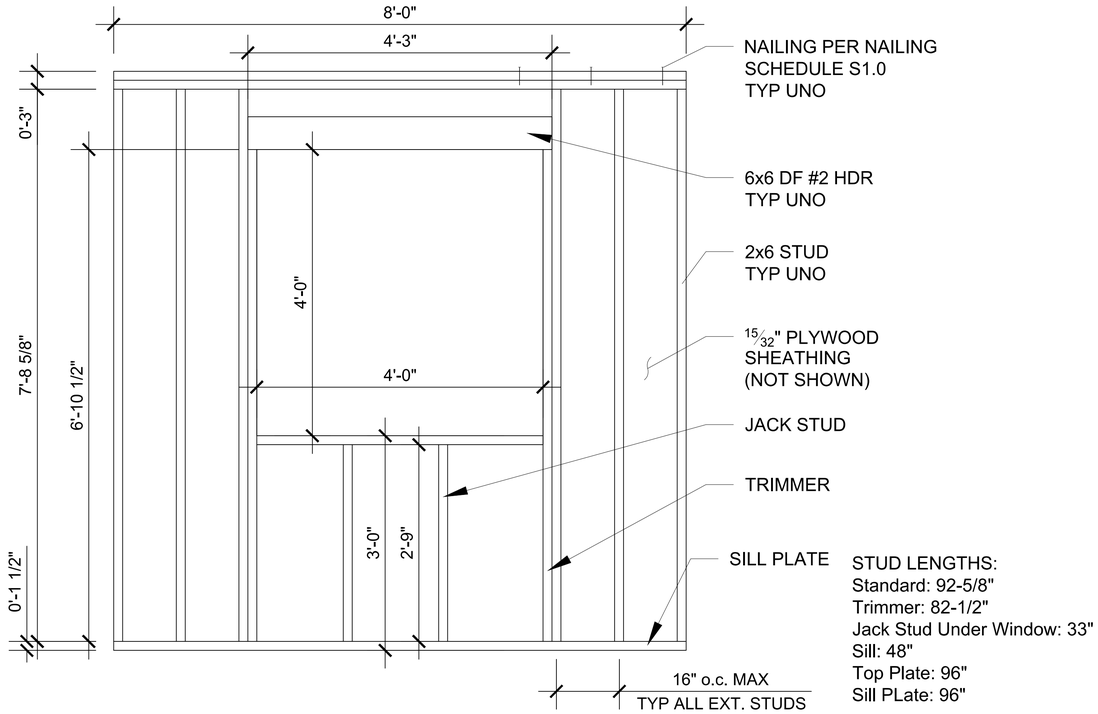
ROOF DETAILS

CAPSTONE CLASS
ARCE 415

DATE 12/1/17
SCALE 1" = 1'-0"
DRAWN R.d.S
JOB CAPSTONE
SHEET

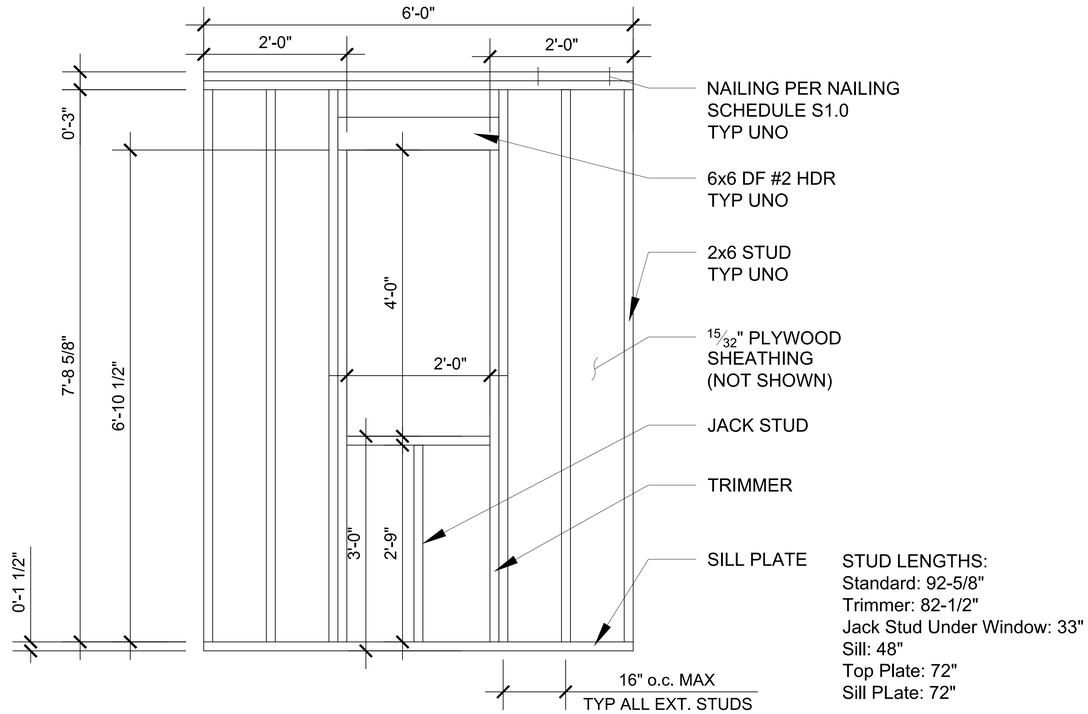
S3.1

OF 6 SHEETS



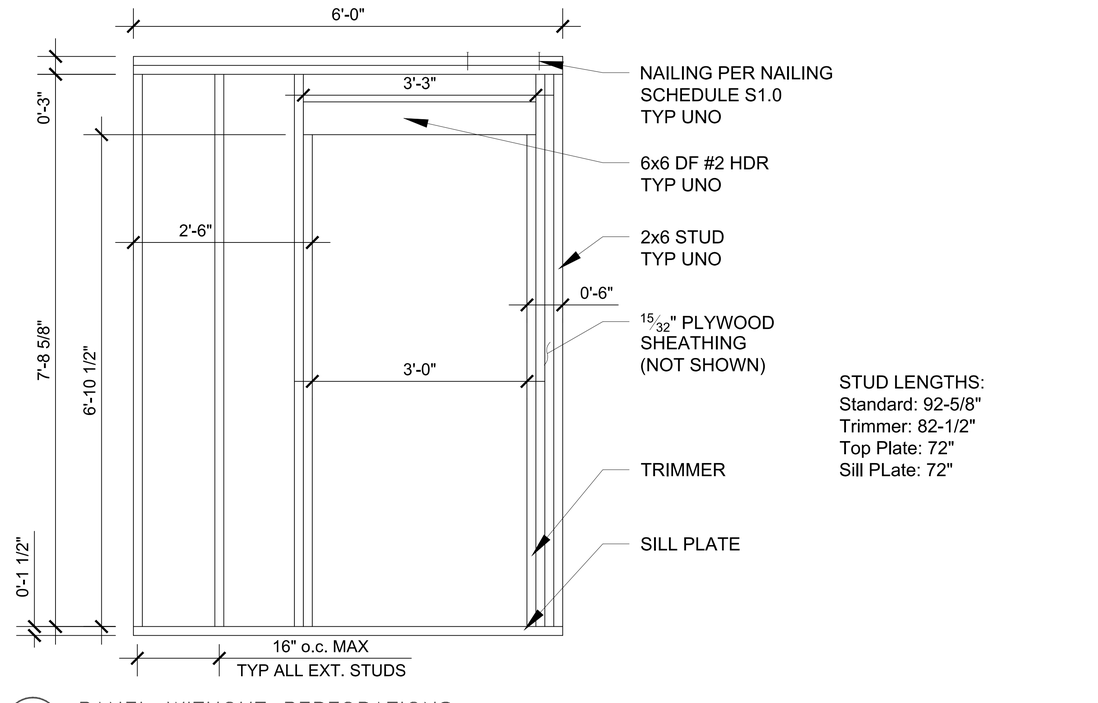
B PANEL WITH SMALLER WINDOW

SCALE: 1"=1'-0"



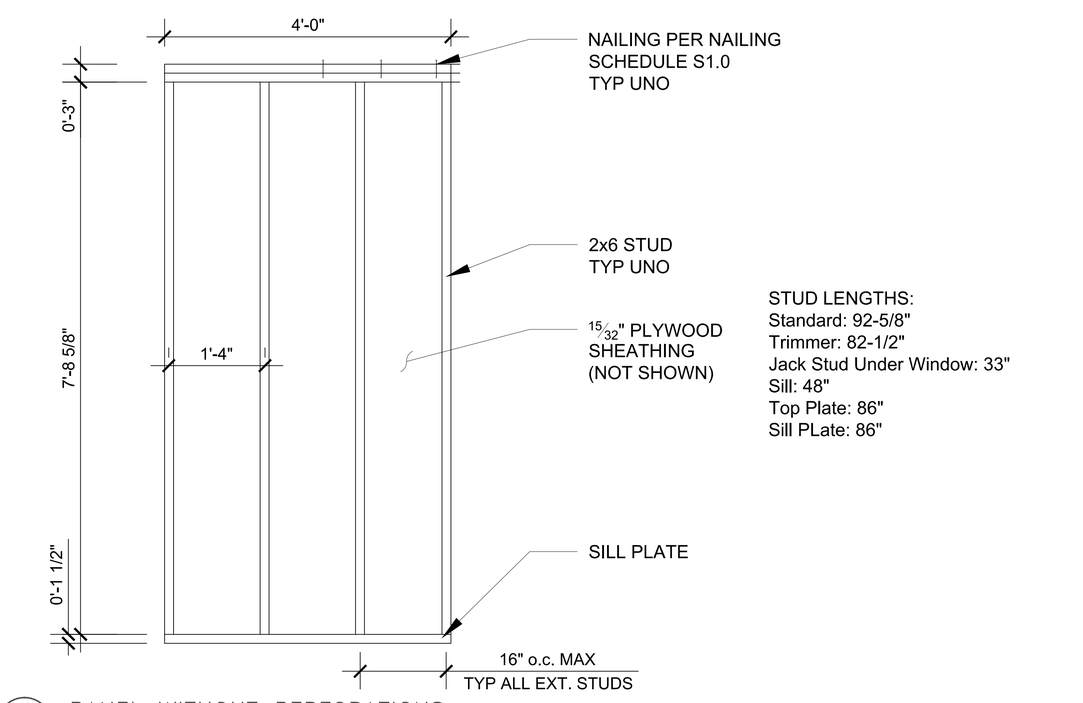
L PANEL WITH SMALLER WINDOW

SCALE: 1"=1'-0"



J PANEL WITHOUT PERFORATIONS

SCALE: 1"=1'-0"



O PANEL WITHOUT PERFORATIONS

SCALE: 1"=1'-0"

RORY DE SEVILLA
rdesevil@calpoly.edu

PANEL DETAILS

CAPSTONE CLASS
ARCE 415

DATE 12/1/17
SCALE 1" = 1'-0"
DRAWN R.d.S
JOB CAPSTONE
SHEET

S4.1

OF 6 SHEETS