TO:
Mr. Craig Browne, Building Official
Weber County Building Inspection Department
2380 Washington Blvd, Ste 240
Ogden, UT 84401
T (801) 399-8374

RE:
Ninebark Townhomes for Summit Powder Mountain
1st Plan Review Resubmittal Narrative

DATE:
March 10, 2017

STUDIO MA
ARCHITECTURE & ENVIRONMENTAL DESIGN

130 N CENTRAL AVENUE, NO.300
PHOENIX, ARIZONA 85004
TELEPHONE 602 251 3800
FAX 602 251 3100

STUDIOMA.COM

DEAR MR. BROWNE:

Below, please find a narrative of revisions, additions, and clarifications for the above noted project per the WC3 First Review dated February 13, 2017. Please do not hesitate to contact me should you have any questions.

Note that per the revised plat, the lot numbers have changed as follows:

Original:	60A	60B	61A	61B	62R
Revised:	138	139	140	141	142

COMMENT	DESCRIPTION
A1.	Sheet S3.11: As a matter of consideration for field practices, it is this reviewer's opinion that these highlighted details below are extremely difficult to correctly apply in the field. Typical field practices don't allow for j-bolts and horizontal re-enforcing bar extensions mid-wall or below top of form because it would become necessary for the foundation contractor to cut or drill the forms. The design and installation is not ipso facto against code and no correction is required, but it would serve well, if these details do remain, to note that drilling and epoxying require special inspection and test on all anchors installed post foundation casting.
Response	*Noted.*
A2.	Please clearly indicate on the plans the requirements for safety glazing and where it is to be provided. IRC R308.4 requires that all shower compartments, sliding glass doors, windows adjacent to doors, and other hazardous areas have safety glazing.
Response	*Requirements for safety glazing are noted in the specifications, sheet A0.21, in the general notes on sheets A6.10 and A6.11, and have been added to sheets A0.10 where clouded.*
A3.	Please provide details and dimensions for stairs, including rise and run dimensions.

Response	*Stair dimensions and tread/riser dimensions are noted for the typical unit on sheet A2.10; and on A2.11 for unique unit 62R.*
A4.	Please provide details and dimensions for guardrails to be provided at the interior and exterior stairs, and at the edge of the exterior deck. Guards should meet the minimum requirements of IRC R312.1.
Response	*Details for the exterior guardrails are provided as per drawings 3 and 4 on sheet A5.07, and dimensions for the exterior guardrail are indicated on the building sections on sheets A2.00, A2.01, and A2.03. Dimensions for the interior stair guardrail and handrails are provided on drawing 3/A2.10, and details are provided as drawings 6 and 7 on sheet A2.10, and as drawings 10, 11, 13, and 14 on sheet A5.11. There are no exterior stairs within this project's scope of work.*
A5.	Please show on the plans how the building address will be listed as required by IRC R319.1.
Response	*Address signage requirement is noted on code analysis sheet A0.10, and has been added to specifications, sheet A0.22.*
A6.	Provide notes or details on the plans showing the required separation between the garage and the dwelling with a minimum of ½ inch gypsum board as required by IRC R302.6, and 5/8 type X at the ceiling where living space is located above the garage.
Response	*Requirements per R302.6 are noted on code analysis sheet A0.10: Dwelling/Garage Fire Separation [TABLE R302.6]: Walls: 1/2" thk. gypsum board applied to the garage side. Ceiling: 5/8" thk. Type X gypsum board.*
M2.*	Please provide details/cut sheets on fireplaces in order to determine code compliance.
*Response** provided by Architect based on specification scope.	*Refer to attached product information indicating UL compliance.*

Sincerely,

Daniel J Hoffman

Dan Hoffman, RA
Studio Ma, Inc.

END OF MEMORANDUM

GLHN ARCHITECTS & ENGINEERS, INC
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Plan Review Comment Responses

Project: **Summit Powder Mountain Townhomes**
GLHN No. 1530.00

Plan Reviewer: **WC3**
Project No. 217-525-008

Architect: **Studio MA**
130 N. Cewntral Ave. #300
Phoenix AZ 85004
602-251-3800

Reviewers Rewiew: **February 13, 2017**

Contact: **Tim Keil**

Issued to Engineer: **Feb. 14, 2017 via e-mail**

Reviewer: **Mike Molyneux**

The following are comments and or concerns with the sealed contract documents as reviewed by the above mentioned office having jurisdiction to do so. As indicated below, the adjustments have been made and incorporated into the sealed contract documents. The acceptance of these adjustments incorporated into the sealed documents shall be approved by the plan reviewer mentioned above and made available to the contractor for construction. **GLHN** shall not be responsible for any documents issued to contractor for bidding purposes if differ from these plans and do not carry the approval for construction signatures from the office having jurisdiction.

The below represents **GLHN's** response to the plan reviewer's comments. Please refer to your office copy for actual comment.

Mechanical Comments

1. Total dryer duct lengths have been added to the plans. Original design included booster fans and installations in accordance with IRC M1502.4.4.1

2. Fire place details and cut sheets have been provided.

Plumbing Comments

1. Comply – Added "maximum flow rates" to fixture schedule.

2. Comply – Added "vacuum breaker" to FPH on fixtureschedule. FPHB has integral vacuum breaker already called out.

Electrical Comments

1. Added "GFCI" device symbol sheet e1.21 and e1.23.

2. Added 1- GFCI rated (keynote #8) above counter receptacle location sheet e1.21. Adjusted spacing and deleted 1- device sheet e1.23.

3. Revised FA symbol identification to include a combination Carbon Monoxide sensor and Ioniztion Smoke Alarm. Added and adjust Fire Alarm devices sheets e1.20, e1.21, e1.22, and e1.23

4. Added comment to sheet e1.20 General Note "X".

5. Refer to General Note "S" sheet e1.20.

6. Added comment to General Note "P" sheet e1.20.

End of review comments.

GLHN Architects & Engineers

Stanley Yellowhair (M)
Mechanical Designer – Project Manager

Ross Mellencamp (P)
Plumbing Designer

John Gomez III (E)
Senior Electrical Designer

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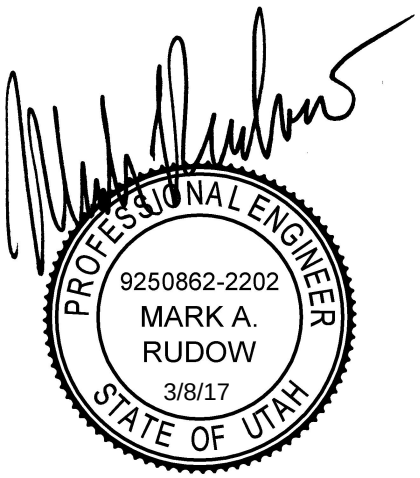
job name:	Copper Crest East			pg
job number:	15105			of
designed by:	MAR	date:	3/2017	
checked by:		date:		

Summit Powder Mountain Townhomes
Copper Crest East
WC^3 Project #217-525-008



Response to Plan Review Comments

Architectural Comment A1: Sheet S3.11: As a matter of consideration for field practices, it is this reviewer's opinion…
Response: We generally agree with your assessment and are planning to discuss alternate post-installed fasteners with the contractor once he has been selected and can tell us what type of fastener he prefers to use (adhesive, expansion, screw, etc). For now we will leave the drawings with the cast-in anchors.

Structural Comments:

Structural Drawings:

Comment S1: Please provide the geotechnical report by IGES referenced in the plans for further review.
Response S1: The geotechnical report is included as an appendix to the structural calculations that were submitted. Please refer to sheet 448 of 484 of the calculation pdf.

Comment S2: Sheet s0.12: The Simpson strap schedule has not been provided with marks as shown on the plans. Please address.
Response S2: The missing marks were inadvertently left off the schedule and have now been added to it.

Comment S3: Sheet s0.13: The special inspection requirements shown are per the 2009 IBC. Significant changes have been made to the special inspection requirements especially the steel inspections per Section N5.4 of AISC 360-10. Please address.
Response S3: The statement of special inspections has been revised to reflect the current code requirements.

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checked by:		date:		

Comment S4: The footing calculations (page 333) for SW-5 show that a 3.67 foot wide footing is required while the drawings show a WF3 which is a 3-foot wide footing. Please coordinate.
Response S4: Refer to calculation sheet 282. The wall SW5 is the first wall west (right) of grid B on the plan (just right of the concrete basement wall). This wall is noted as WF6 which is a 3'-8" footing matching the calculations. The wall with the WF3 mark is wall SW3 which is correct per calculation sheet 329.

Comment S5A: Sheet S1.23: Detail 5/S4.14 is shown at gridline B at the entry level. This detail has not been provided. Please address.
Response S5A: The missing detail has been added to the plans as requested.

Comment S5B: Sheet S1.23: Beam TH11 at the entry level does not match what is shown in the calculations. The calculations show 6-3/4x10-1/2 glu-lam beam while the drawings show a 6-3/4x9 glu-lam beam. Please address.
Response S5B: We have changed the size on the plans to match the calculations.

Comment S6: Sheet s1.24: Please provide the size of the header near beam FB8 that is supporting FB10. This is not shown in the plans or detail 1/s4.14.
Response S6: The header size has been added to the framing plan.

Comment S7: Sheet s4.10: Multiple floor framing details reference a Simpson hanger per schedule. Please clarify where this schedule can be found on the plans.
Response S7: The hanger schedule has been added to sheet S0.12.

Comment S8: Sheet s4.13: Detail 9 does not show end wall blocking at the floor joists which run parallel to the foundation walls. Please provide a detail showing the blocking requirements as required by Section 12.11.2.2 of ASCE 7-10.
Response S8: The blocking requirements have been added to the plan and detail.

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Structural Calculations:

Comment S9: There is a nonparallel irregularity as defined in Table 12.3-1 of ASCE 7-10. This occurs at the end unit. Please address.
Response S9: The lateral analysis for unit 62R has been revised to satisfy ASCE 7-10. The unit has been modelled in RISA3d using stiffness coefficients per NDS for all shear walls and floor/roof diaphragms. Forces have been determined based on the worst case of 100% of one direction plus 30% of the orthogonal direction as required. All calculations have been modified and the drawings adjusted as required. See the attached sheets S6 through S70 for the revised analysis.

Comment S10: The beam calculations are per the 2012 IBC and 2012 NDS. Please verify that the calculations meet the requirements of the 2015 IBC and all referenced standards.
Response S10: We have reviewed all beam calculations for the 2015 IBC and 2015 NDS requirements and have found that none of the members need to be changed.

Comment S11A: Please clarify whether the simplified or equivalent lateral force method was used.
Response S11A: The equivalent lateral force method was used.

Comment S11B: Please show how the forces at each level were determined (vertical distribution of forces, etc. as applicable). Calculations for Unit 62R show the vertical distribution of forces. Please provide for the other units.
Response S11B:

For both the 62R and Typical 20 foot units: The seismic coefficients are determined on page 20 using the Equivalent Lateral Force method of ASCE 7-10 section 12.8. On pages 22-24 the wind pressures are determined using the ASCE sections noted. Wind pressures are broken into pressure and suction (windward and leeward) to allow a more accurate load determination due to the split floor levels.

For the typical 20 foot wide unit:

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designed by:	MAR	date:	3/2017	
checked by:		date:		

On page 280, dead loads are determined for each floor and roof level and seismic redistribution is performed to determine the adjusted percentage of total seismic force going to each level and the resulting force at each level. On page 281 the resulting ASD seismic loads at each level are shown (without rho applied). The loads are shown as total seismic loads on a given floor section.

On pages 282 and 283, the wind loads at each floor and roof level are developed based on tributary heights to each floor/roof level. Tributary heights were determined graphically using the building elevations and sections due to the complexity of the building profile and the pressure/suction effects of the split levels.

On pages 284 and 285, wind and seismic loads (with rho added) are determined for each wall using tributary widths and simple (flexible) diaphragm spans between shear walls.

For the 62R Unit:

Due to the non-regular shape of the unit, the simpler method used for the typical unit can't be followed for unit 62R. On page 286, seismic loads at each level are shown in the form of varying line loads applied to the diaphragm edges. These loads are developed on pages 287-288 based on the floor and roof weights and the varying building dimensions. The total seismic load at each level is calculated based on the varying line loads and building dimensions. On page 289, the seismic redistribution is performed based on the seismic load at each level and the height of the level above the ground. An adjustment factor is determined at each level that reflects the magnification or reduction of the seismic load at each level due to the redistribution exercise. (For example the roof seismic load of 8774# is 42.4 percent of the total seismic force, however the redistributed roof seismic load is 61.1 percent. This results in an adjustment factor of 1.44 (61.1/42.4) for the seismic loads at the roof level.) On pages 290 and 291, the seismic forces at each shear wall are determined. The diaphragms have been modeled as simple beam elements with the varying line loads applied to determine the forces going to each wall. We did not include these analyses in the calculation book but can provide them if you want them. The resulting shear wall forces are then adjusted using the adjustment factors determined in the seismic force redistribution exercise described earlier.

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checked by:		date:		

On pages 292-295, wind loads are developed using the method described for the typical 20 foot unit.

Comment S12: The roof snow load is listed as 263psf. Please confirm that a percentage of the snow was considered in the seismic weight of the structure as required by Section 1605.3.1 and 1605.3.2 of the Utah Amended Code

Response S12: The ground snow load is 263 psf. The roof snow load is 203 psf as calculated on page 17. Twenty percent of the roof snow load is included in the effective seismic weight as required by ASCE section 12.7.2.4. This can be seen on page 280 for example in the calculation of the roof dead load at the top of the sheet where 0.2 x 203 is added to the typical 30 psf DL allowance.

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job name: CCE/Ninebark pg
job number: 15105 of
designed by: MAR date: 3/17
checked by: date:

Revised Lateral Analysis at Unit 62R

To account for the non-orthogonal systems irregularity, the unit has been redesigned using RISA3D to model the semi-rigid floor and roof diaphragms and shear walls.

Seismic Loads from previous load calculations (see calculation page 289):

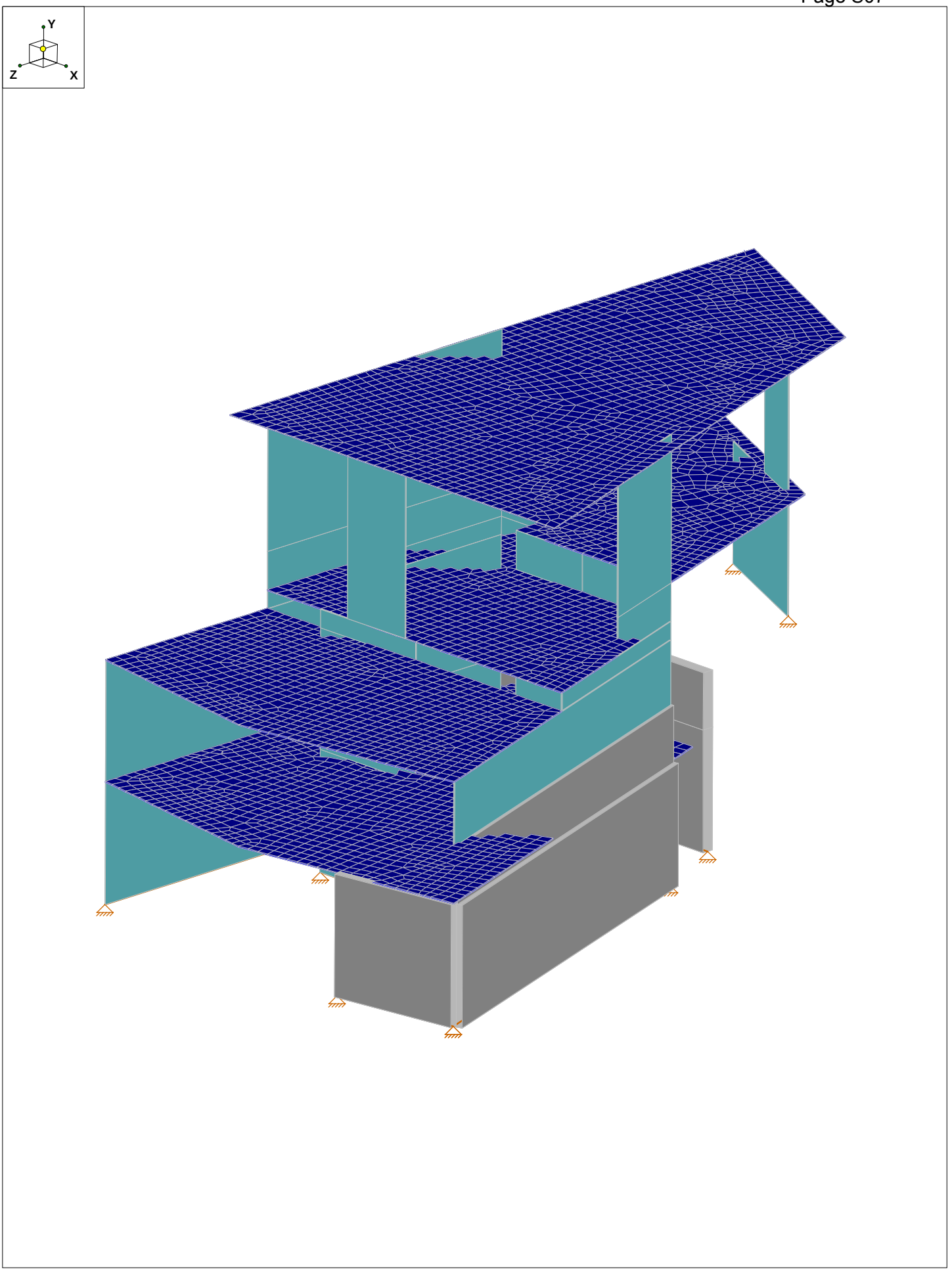
Roof (Level 5)
V = 8774 * 1.44 = 12591#
Roof Area = 1391 ft2
Load to plate elements in model = 9.05 psf

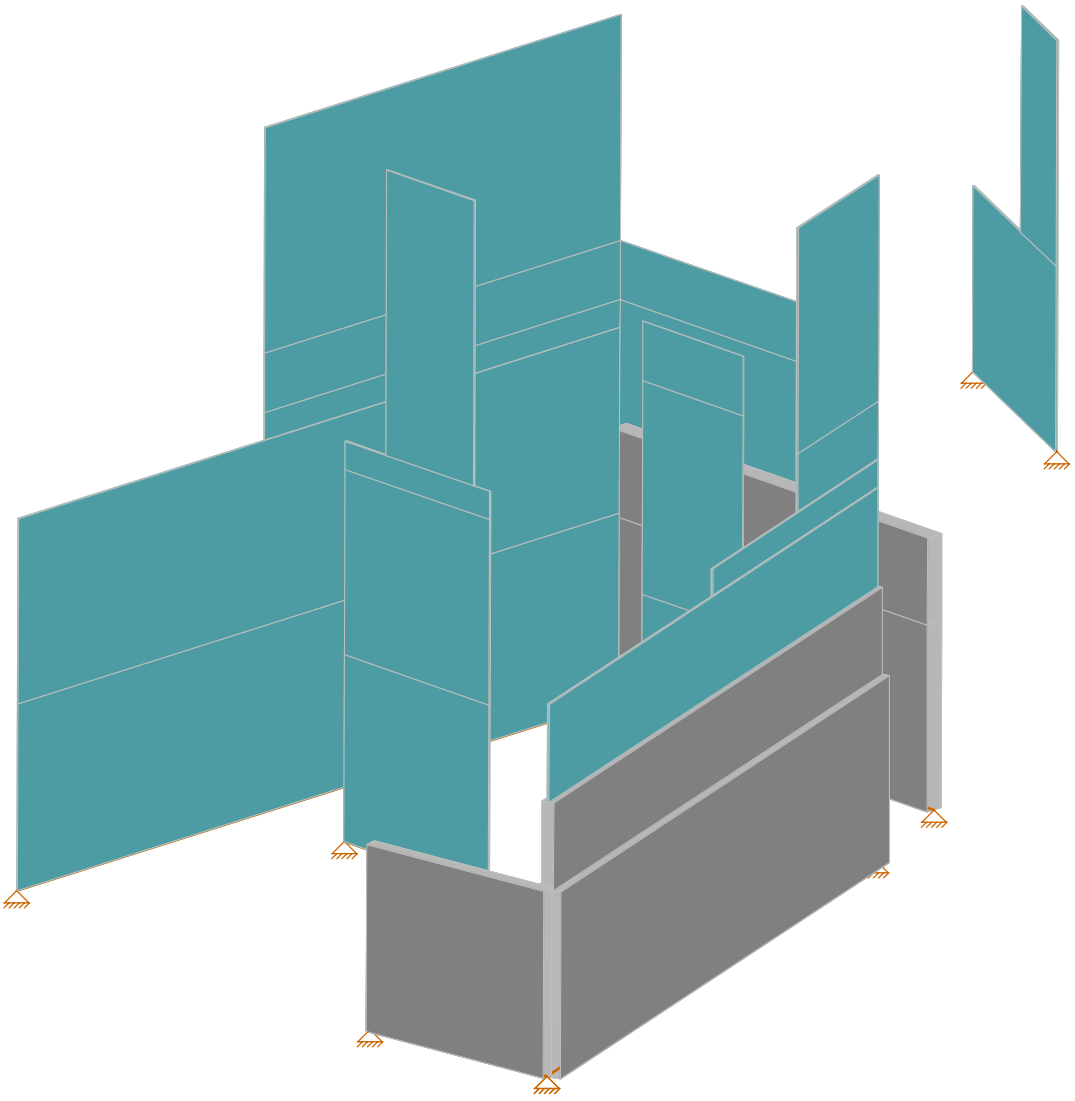
+9-11 (Level 4)
V = 1266 * 0.98 = 1241#
Floor Area = 506 ft2
Load to plate elements in model = 2.45 psf

+6-9 (Level 3)
V = 1617 * 0.81 = 1310#
Floor Area = 586 ft2
Load to plate elements in model = 2.24 psf

+5-3 (Level 2 - Terrace)
V = 4291 * 0.81 = 3476#
Floor Area = 674 ft2
Load to plate elements in model = 5.16 psf

-4-8 (Level 1)
V = 4764 * 0.43 = 2049#
Floor Area = 1322 ft2
Load to plate elements in model = 1.55 psf


Y
Z
X



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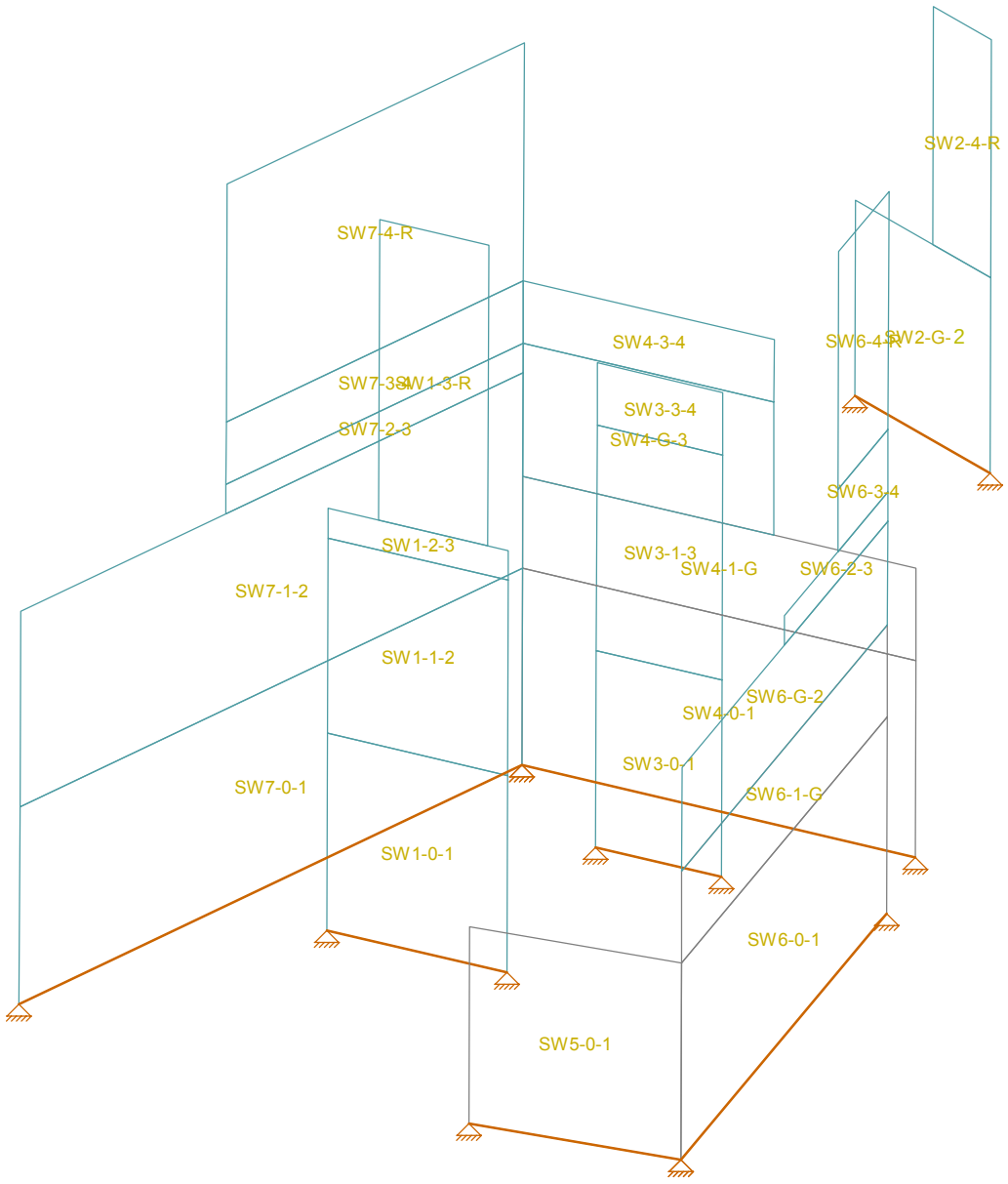
	Unit 62R RISA-3D Model	62R-FULL-05.r3d



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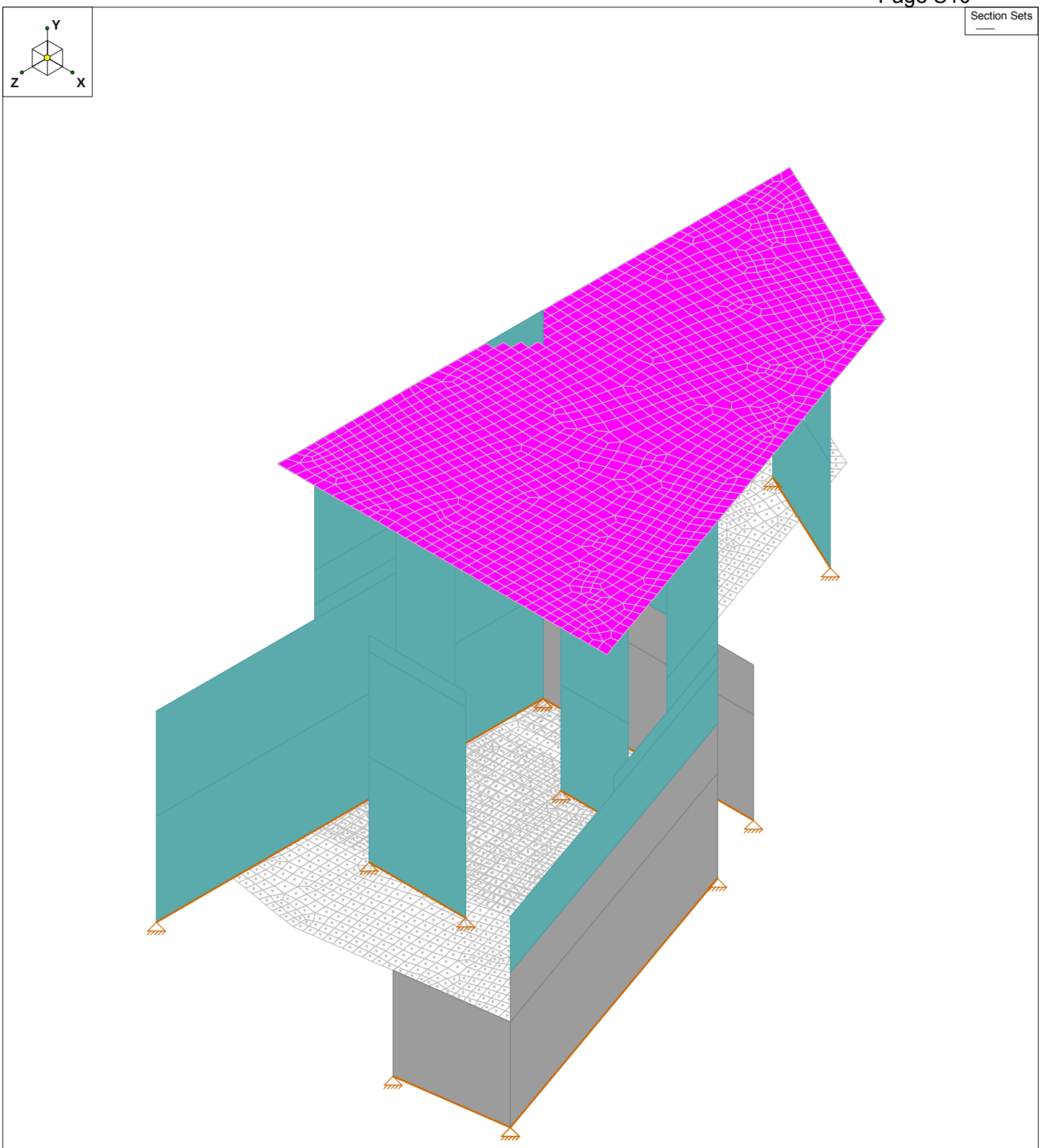
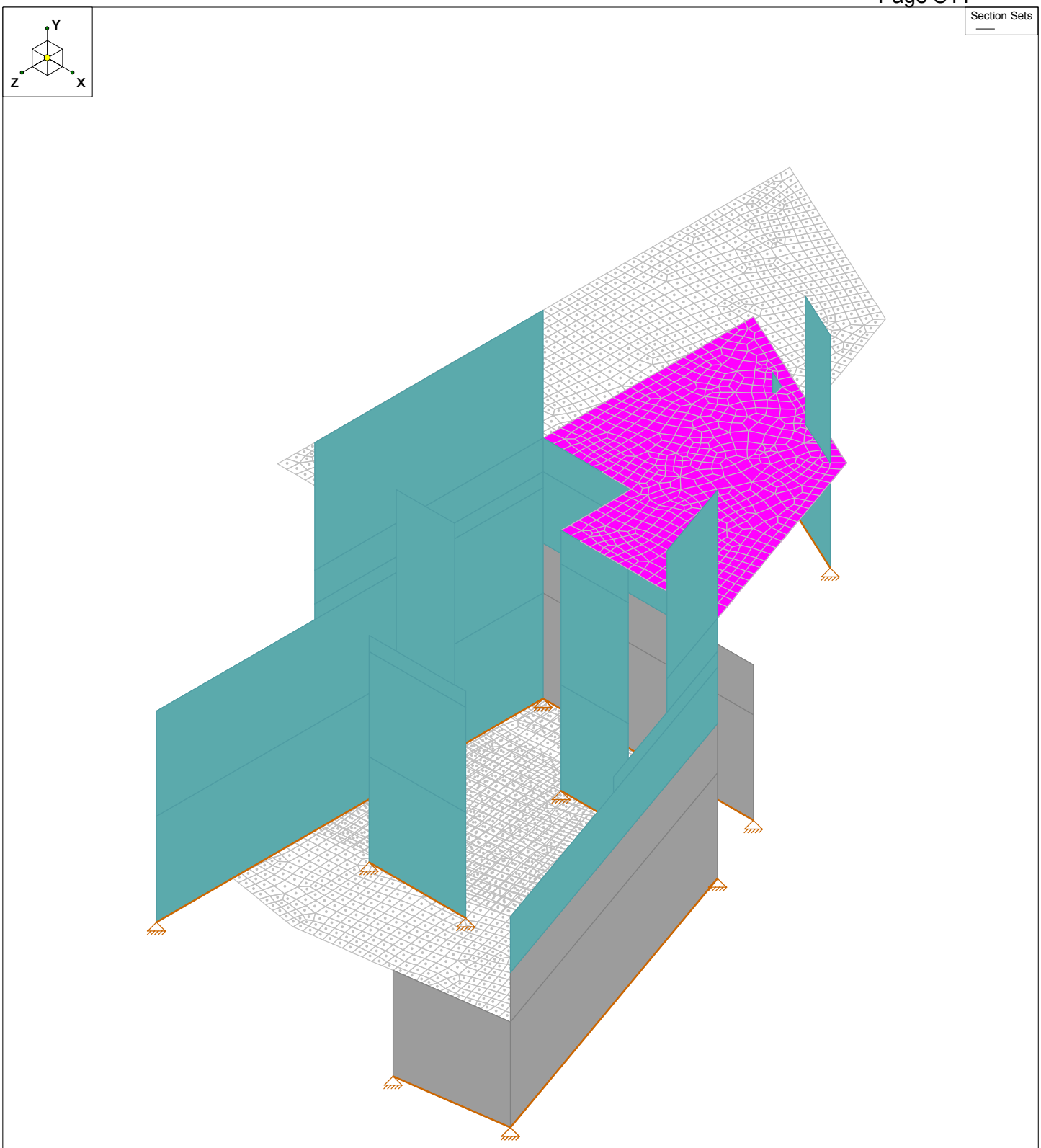
	Unit 62R RISA-3D Model	62R-FULL-05.r3d



		Mar 6, 2017 at 11:25 AM
	Unit 62R Model Roof	62R-FULL-05.r3d



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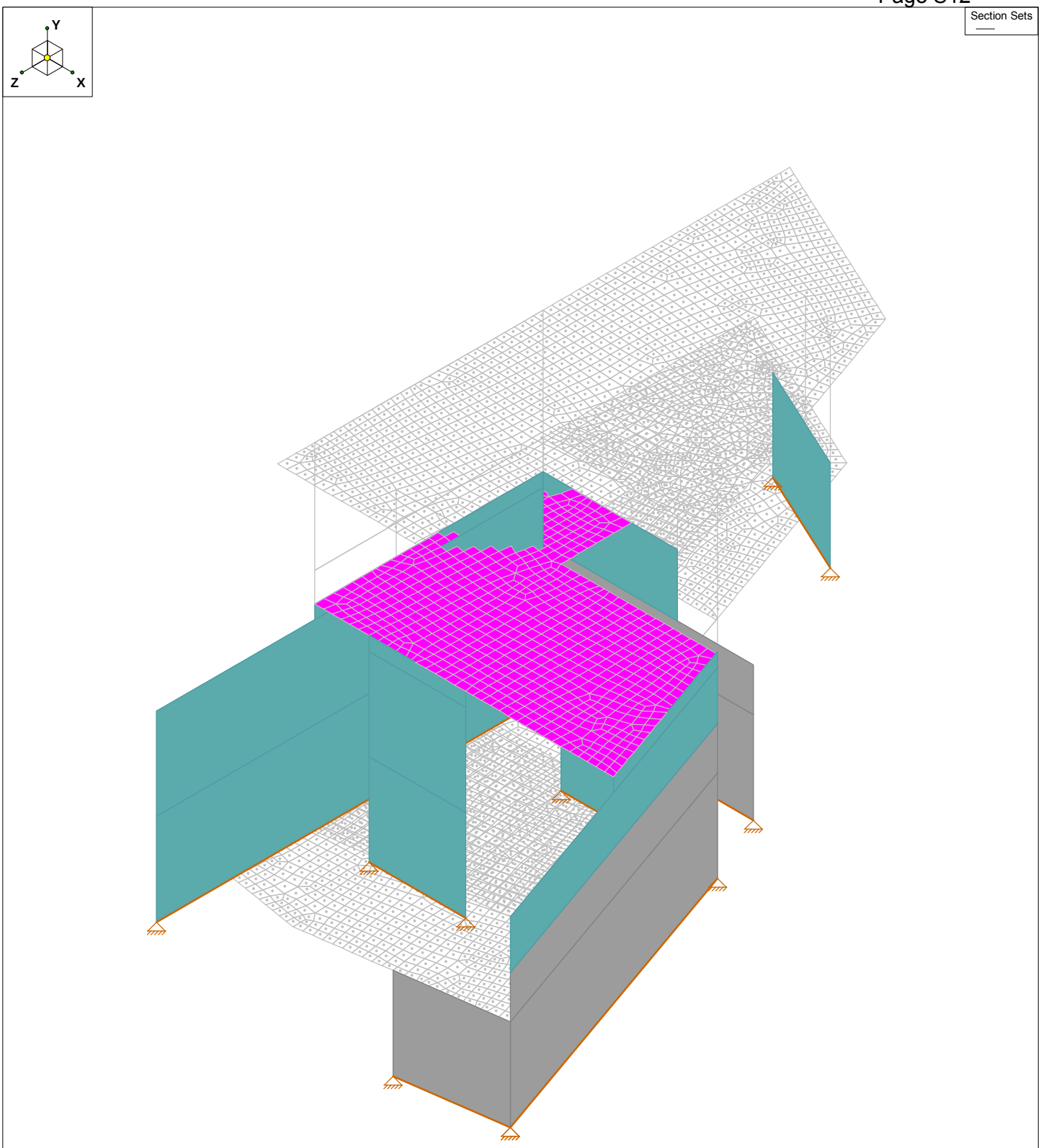
	Unit 62R Model Level 4 (+9-11)	62R-FULL-05.r3d



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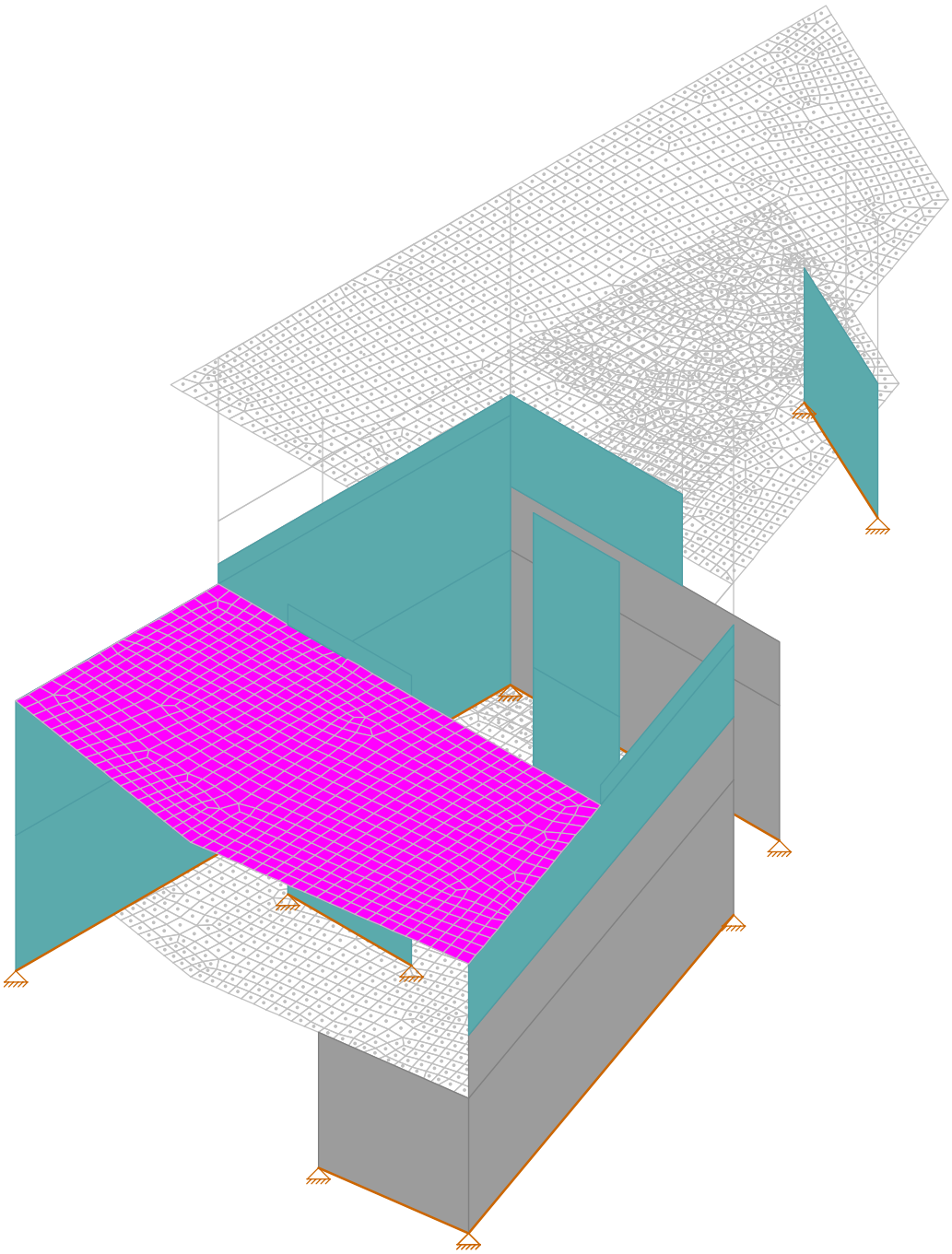
	Unit 62R Model Level 3 (+6-9)	62R-FULL-05.r3d

Section Sets



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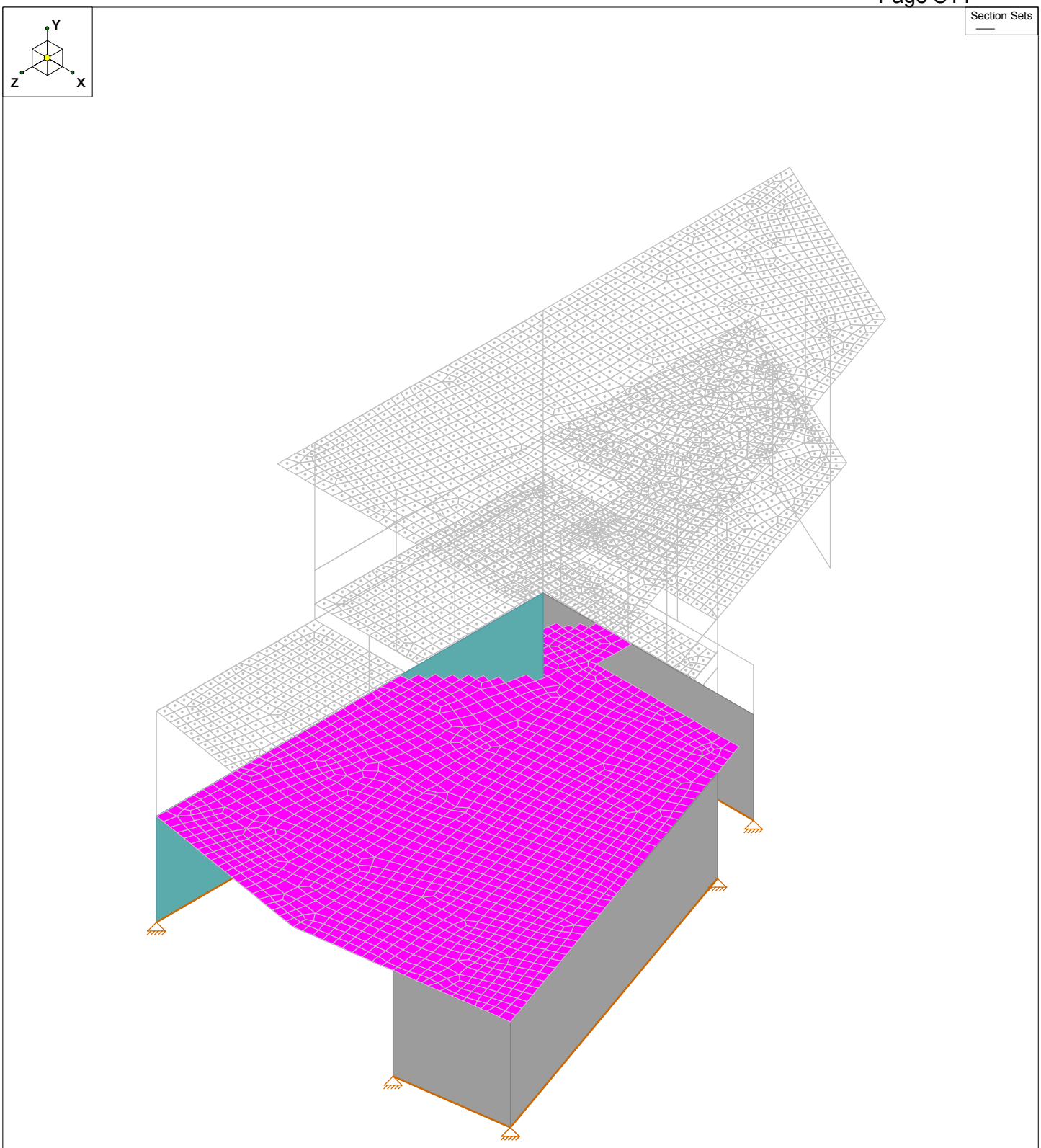
	Unit 62R Model Level 2 (+5-3)	62R-FULL-05.r3d



		Mar 6, 2017 at 11:34 AM
	Unit 62R Model Level 1 (-4-8)	62R-FULL-05.r3d

Wall Panel Forces

	LC	Wall Label	Elevation [ft]	Axial [k]	x Shear [k]	z Shear [k]	x-x Moment [k-ft]	z-z Moment [k-ft]
1	1	SW1-0-1	-14.67	0	.312	-.011	0	-23.264
2	1	SW1-1-2	-4.67	0	.279	.007	.039	-20.147
3	1	SW1-2-3	5.25	0	-.399	-.127	-.152	-17.376
4	1	SW1-3-R	6.75	0	1.179	-.008	.016	-17.975
5	1	SW2-4-R	9.92	0	1.493	-.008	-.018	-18.069
6	1	SW2-G-4	0	0	3.475	-.005	0	-52.575
7	1	SW3-0-1	-14.67	0	.127	-.005	0	-1.199
8	1	SW3-1-3	-4.67	0	.295	-.005	-.001	.075
9	1	SW3-3-4	6.75	0	-1.085	-.01	.002	3.437
10	1	SW4-0-1	-14.67	-5.553	3.32	-.014	0	17.511
11	1	SW4-1-G	-4.67	-4.095	1.078	-.039	-.012	33.94
12	1	SW4-3-4	6.75	-1.005	.267	-.027	0	6.548
13	1	SW4-G-3	0	-2.288	1.073	-.009	0	8.679
14	1	SW5-0-1	-14.67	1.21	-.96	-.008	0	16.501
15	1	SW6-0-1	-14.67	-1.21	18.039	.008	0	-446.861
16	1	SW6-1-G	-4.67	0	14.582	.019	0	-289.014
17	1	SW6-2-3	5.25	0	12.485	.003	-.022	-144.818
18	1	SW6-3-4	6.75	0	10.478	-.038	-.009	-124.514
19	1	SW6-4-R	9.92	0	7.705	.013	.112	-92.927
20	1	SW6-G-2	0	0	14.579	.017	-.01	-221.131
21	1	SW7-0-1	-14.67	5.553	5.29	0	0	-94.986
22	1	SW7-1-2	-4.67	4.095	6.432	.001	-.004	-73.138
23	1	SW7-2-3	5.25	1.288	4.762	-.005	0	-80.154
24	1	SW7-3-4	6.75	1.005	5.451	-.013	-.004	-76.577
25	1	SW7-4-R	9.92	0	5.982	0	.006	-71.912

Wall Panel Forces

	LC	Wall Label	Elevation [ft]	Axial [k]	x Shear [k]	z Shear [k]	x-x Moment [k-ft]	z-z Moment [k-ft]
1	2	SW1-0-1	-14.67	0	-5.958	.001	0	279.784
2	2	SW1-1-2	-4.67	0	-10.182	-.011	-.011	220.266
3	2	SW1-2-3	5.25	0	-6.382	.076	.097	119.381
4	2	SW1-3-R	6.75	0	-7.211	-.001	-.017	109.829
5	2	SW2-4-R	9.92	0	-6.218	.008	.059	75.032
6	2	SW2-G-4	0	0	-6.796	-.003	0	142.372
7	2	SW3-0-1	-14.67	0	-.269	0	0	10.068
8	2	SW3-1-3	-4.67	0	-.842	0	.002	7.427
9	2	SW3-3-4	6.75	0	.663	-.001	-.004	-2.133
10	2	SW4-0-1	-14.67	.909	-5.334	0	0	78.094
11	2	SW4-1-G	-4.67	.979	-2.662	.009	.013	24.097
12	2	SW4-3-4	6.75	.517	-1.653	.007	-.001	1.358
13	2	SW4-G-3	0	1.157	-2.632	0	0	14.502
14	2	SW5-0-1	-14.67	-3.282	-3.892	-.001	0	20.159
15	2	SW6-0-1	-14.67	3.282	-2.607	-.019	0	19.174
16	2	SW6-1-G	-4.67	0	-3.611	-.056	-.054	53.566
17	2	SW6-2-3	5.25	0	-2.512	.006	.041	17.582
18	2	SW6-3-4	6.75	0	-1.271	.052	-.009	13.608
19	2	SW6-4-R	9.92	0	-.805	-.027	-.183	9.768
20	2	SW6-G-2	0	0	-3.612	-.053	-.009	36.626
21	2	SW7-0-1	-14.67	-.91	.139	-.028	0	2.164
22	2	SW7-1-2	-4.67	-.979	.561	-.071	-.016	2.084
23	2	SW7-2-3	5.25	-.679	-.416	-.026	-.006	19.76
24	2	SW7-3-4	6.75	-.517	-1.676	-.02	-.005	21.152
25	2	SW7-4-R	9.92	0	-1.844	-.03	-.016	22.269

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job name: CCE pg
job number: 15105 of

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Shear Wall Forces

Shear Wall Segment	Roof/Floor Levels	Gross Shears							
		EqX (N-S)	EqZ (E-W)	Eq-X (S-N)	Eq-Z (W-E)	EqX + .3EqZ	EqX + .3Eq-Z	EqZ + .3EqX	EqZ + .3Eq-X
		(kips)	(kips)	(kips)	(kips)	(kips)	(kips)	(kips)	(kips)
SW1	0-1	-5.958	0.312	5.958	-0.312	-5.864	-6.052	-1.475	2.099
	1-2	-10.182	0.279	10.182	-0.279	-10.098	-10.266	-2.776	3.334
	2-3	-6.382	-0.399	6.382	0.399	-6.502	-6.262	-2.314	1.516
	3-RF	-7.211	1.179	7.211	-1.179	-6.857	-7.565	-0.984	3.342
SW2	G-4	-6.796	3.475	6.796	-3.475	-5.754	-7.839	1.436	5.514
	4-RF	-6.218	1.493	6.218	-1.493	-5.770	-6.666	-0.372	3.358
SW3	0-1	-0.269	0.127	0.269	-0.127	-0.231	-0.307	0.046	0.208
	1-3	-0.842	0.295	0.842	-0.295	-0.754	-0.931	0.042	0.548
	3-4	0.663	-1.085	-0.663	1.085	0.338	0.989	-0.886	-1.284
SW4	0-1	-5.334	3.320	5.334	-3.32	-4.338	-6.330	1.720	4.920
	1-3	-2.662	1.078	2.662	-1.078	-2.339	-2.985	0.279	1.877
	3-4	-1.653	0.267	1.653	-0.267	-1.573	-1.733	-0.229	0.763
SW5	0-1	-3.892	-0.960	3.892	0.96	-4.180	-3.604	-2.128	0.208
SW6	0-1	-2.607	18.039	2.607	-18.039	2.805	-8.019	17.257	18.821
	1-2	-3.611	14.582	3.611	-14.582	0.764	-7.986	13.499	15.665
	2-3	-2.512	12.485	2.512	-12.485	1.234	-6.258	11.731	13.239
	3-4	-1.271	10.478	1.271	-10.478	1.872	-4.414	10.097	10.859
	4-RF	-0.805	7.705	0.805	-7.705	1.507	-3.117	7.464	7.947
SW7	0-1	0.139	5.290	-0.139	-5.29	1.726	-1.448	5.332	5.248
	1-2	0.561	6.432	-0.561	-6.432	2.491	-1.369	6.600	6.264
	2-3	-0.417	4.762	0.417	-4.762	1.012	-1.846	4.637	4.887
	3-4	-1.676	5.451	1.676	-5.451	-0.041	-3.311	4.948	5.954
	4-RF	-1.844	5.982	1.844	-5.982	-0.049	-3.639	5.429	6.535

Level 0 = -14'-8"
Level 1 = -4'-8"
Level 2 = +5'-3"
Level 3 = +6'-9"
Level 4 = +9'-11"
Level G = 0'-0"

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designed by: MAR date: 3/2017
checked by: date:

Shear Wall Forces			
Shear Wall Segment	Roof/Floor Levels	Gross Shear (kips)	Net Shear (kips)
SW1	0-1	-6.052	-4.214
	1-2	-10.266	4.003
	2-3	-6.262	-1.302
	3-RF	-7.565	6.834
SW2	G-4	-7.839	1.173
	4-RF	-6.666	6.666
SW3	0-1	-0.307	0.623
	1-3	-0.931	-1.919
	3-4	0.989	0.989
SW4	0-1	-6.330	3.345
	1-3	-2.985	1.252
	3-4	-1.733	1.733
SW5	0-1	-4.180	4.180
SW6	0-1	18.821	3.156
	1-2	15.665	2.427
	2-3	13.239	2.379
	3-4	10.859	2.913
	4-RF	7.947	7.947
SW7	0-1	5.248	-1.015
	1-2	6.264	1.377
	2-3	4.887	-1.067
	3-4	5.954	-0.581
	4-RF	6.535	6.535

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SHEAR WALL DESIGN

SW1 - WALLS ON LINE C

ALL WALLS ARE TIED TOGETHER AT ROOF & LO2.
UNITS ARE INDEPENDENT AT LO1

AT ROOF

$V_{TOT} = 4(4101) + 6631 = 25459^{\#}$ EQ
$= 4095 + 3021(.38) = 5243^{\#}$ WIND

$L_{WALL} = 4(4.5) + 6.33 = 24.33'$

$V_{GRP\ UNIT} = \frac{26393}{5243}\left(\frac{4.5}{24.33}\right) = 4881^{\#}$ EQ., $970^{\#}$ WIND

$V_{GLR} = 11\left(\frac{6.33}{24.33}\right) = 6614^{\#}$ EQ., $1364^{\#}$ WIND

7565 | 26393 | 6834 | 4881

AT LO2

$V_{TOT} = 4(2053) + 4054 = 12266^{\#}$ EQ.
$= 6712 + 5881(.38) = 8947^{\#}$ WIND

$L_{WALL} = 4(10.92) + 10.5 = 54.18$ FT

$V_{GRP\ UNIT} = \frac{10913}{8947}\left(\frac{10.92}{54.18}\right) = 2412^{\#}$ EQ, $1803^{\#}$ WL.

$V_{GLR} = 11\left(\frac{10.5}{54.18}\right) = 2377^{\#}$ EQ, $1734^{\#}$ WL.

2701 | 10913 | 10913 | 8947 | 2200 | 2113

AT LO1

$V_{GRP\ UNIT} = 558^{\#}$ EQ, $3055^{\#}$ WIND (N. UNIT ONLY)

$V_{GLR} = 637^{\#}$ EQ, $1027^{\#}$ WIND

-4214

STRAP FORCE BTWN UNITS:

$T_{ROOF} =$ 7565-6834 = 731#

$T_{LO2} =$ 2701-2113 = 588#

USE SIMPSON MSTA30 STRAP

$T_{ALLOW} = 2050^{\#}$

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SHEAR WALL DESIGN

SW2

FOR WALLS ON LINE 4, ALL WALLS ARE TIED TOGETHER AT LOZ & ROOF.
ASSUME SHEAR DISTR. IS PROPORTIONAL TO WALL LENGTH.

$L_{WALL} = 4(41.67') + 5.5 = 24.18$ FT

AT ROOF: $V_{TOT} = 4(4107) + 6025 = 25494$ (6666) $= 24853^{\#}$ EQ.

$= 3759(0.38) + 4229 = 5657^{\#}$ WIND

(0.38 = N. UNIT, 4229 = GLR; FOR SUCTION)

$V_{THP\ UNIT} = \frac{25494}{24853\ OR\ 5657}\left(\frac{41.67}{24.18}\right) = 4924$ (4800)$^{\#}$ EQ, $1093^{\#}$ WI

$V_{GLR} = \quad " \quad \left(\frac{5.5}{24.18}\right) = 5657^{\#}$ EQ, $1287^{\#}$ WI

AT LOZ $V_{TOT} = 4(492) + 410 = 3141$ (1173) $= 2378^{\#}$ EQ. (5798)

$= 1734(.38) + 2192 = 2851^{\#}$ WIND

$L_{WALL} = 4(41.67) + 5.5 = 24.18$

$V_{THP\ UNIT} = \frac{3141}{2378\ OR\ 2851}\left(\frac{41.67}{24.18}\right) = 607$ (459)$^{\#}$ EQ, $551^{\#}$ WIND

$V_{GLR} = \quad " \quad \left(\frac{5.5}{24.18}\right) = 541^{\#}$ EQ, $648^{\#}$ WIND (714)

MAX STRAP FORCES BTWN UNITS:

$T_{ROOF} =$ 6666-5798 = 868#

$T_{LOZ} =$ 1173-714 = 459#

USE SIMPSON LSTA24 MIN ($T_{ALLOW} = 1235^{\#}$ OK)

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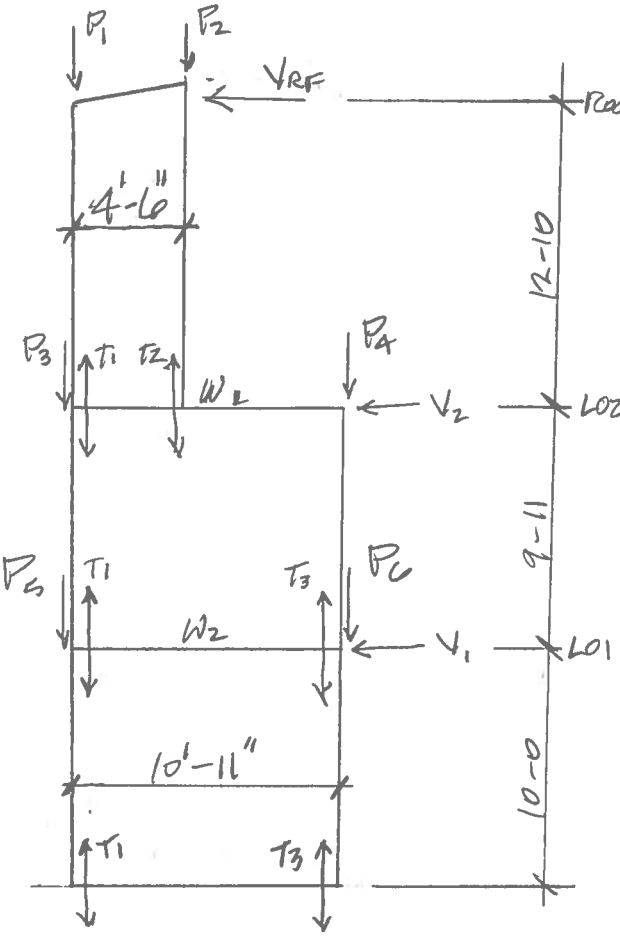
TYPICAL UNIT SHEAR WALLS

WALL SU1



HEAD CONN'S

$N_{LPT} = \frac{4709}{670} = 7.0$ USE (8) SIMPSON LPT4

$N_{LAG} = \frac{4709}{1.4(440)} = 4.6$ USE (5) 5/8" ϕ LAGS

ROOF V_{RF} = 4881 4709# EQ
970# WIND

$P_1 = 330^{\#}_D + 8300^{\#}_{S368} + 3098^{\#}_{S240}$

$P_2 = 2526^{\#}_D + 19890^{\#}_{S368} + 23697^{\#}_{S240}$

V_2 = 2200 2473# EQ.
1803# WIND

$P_3 = P_4 = 764^{\#}_D + 5892^{\#}_S$

$W_1 = 390_D + 3000_S$, PLF

V_1 = 558# EQ.
3055# WIND (H. UNIT ONLY)

$P_5 = P_6 = 636^{\#}_D + 882^{\#}_L$

$W_2 = 305_D + 423_L$, PLF

WALL BTWN LO2 & ROOF

1085

v = 1046 PLF EQ, 216 PLF WIND

$H/b = 2.85 > 2$

USE 5/8" SHT'G B.S. OF WALL W/ 10d @ 2" O.C.

$v_{allow(EQ)} = 2(1140)\frac{1}{2.0}\left(\frac{2}{2.85}\right) = 1221$ ok.

$v_{allow(WIND)} = 2(2435)\frac{1}{2.0}\left(\frac{2}{2.85}\right) = 1709$ ok

SILL CONN.

5/8" ϕ BOLTS @ 16" O.C.; $v_{allow} = 1.6(1001)\frac{12}{16} = 1208$ PLF ok (3 BOLTS MIN.)

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OTM @ L02 = 4.881 / 4.709(12.83) = ±62.62 k EQ. , $F_T = \pm \frac{62.62}{3.83'} = \pm 16.35^k$

TIE AT T1:

P = 3300 + 8300 MAX / 3098 MIN (S) ± 15760 EQ.

= 18818 / 18315 # MAX (SEE NEXT SHT FOR POST DESIGN)

= .6(330 + 12.83(5)(10)/2) − 15760 = −15576 # (MAX TENSION)

SIMPSON MST37 EA SIDE OF EA POST

TALLOW = 4(5080) = 20320 # ok

TIE AT T2:

P = 25260 + 23691 MAX / 19890 MIN (S) ± 15760 EQ.

= 32,561 / 32118# MAX (SEE NEXT SHTS FOR POST DESIGN)

= .6(2526+321) − 15760 = −14057 # (MAX TENSION)

16,350 16,350 14,642

SIMPSON HDU14-SDS2.5

TALLOW = 14445 #

4881+2200=7081 < 7181
prior design is
OK by Inspection

Delta = 1.4%
OK

WALL BRACE L01 & L02

V = 7181 # EQ, 2773 # WIND

U = 658 EQ, 254 WIND, PLF

H/b_s = 0.91 < 2 ok

USE 5/8" SHTG BOTH SIDES OF WALL W/ 10d @ 6" O.C. (MIN)

$U_{ALLOW} = \frac{2(680)}{2.0}$ = 680 EQ.

$= \frac{2(950)}{2.0}$ = 950 WIND

ok

SILL CONN.

$N = \frac{7181}{1001(6.6)} = 4.45$

USE (5) 5/8" ϕ BOLTS

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Engineer: MAR
Project Descr:

Project ID: 15105
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ENERCALC, INC. 1983-2017, Build:6.17.1.16, Ver:6.17.1.16

Wood Column

Lic. # : KW-06002357
Licensee : RUDOW & BERRY

Description : Typ Unit - SW1 - T1 Post from L02 to Roof - 4x8 D.Fir#1

Code References

Calculations per 2012 NDS, IBC 2012, CBC 2013, ASCE 7-10
Load Combinations Used : ASCE 7-10 w/ ASD Wind & EQ

General Information

Analysis Method :	Allowable Stress Design		
End Fixities	Top & Bottom Pinned		
Overall Column Height	12.50 ft		
(Used for non-slender calculations)			
Wood Species	Douglas Fir - Larch		
Wood Grade	No.1		
Fb - Tension	1,200.0 psi	Fv	170.0 psi
Fb - Compr	1,200.0 psi	Ft	825.0 psi
Fc - Prll	1,000.0 psi	Density	31.20 pcf
Fc - Perp	625.0 psi		

E : Modulus of Elasticity . . .	x-x Bending	y-y Bending	Axial
Basic	1,600.0	1,600.0	1,600.0 ksi
Minimum	580.0	580.0	

Wood Section Name	4x8		
Wood Grading/Manuf.	Graded Lumber		
Wood Member Type	Sawn		
Exact Width	3.50 in	Allow Stress Modification Factors	
Exact Depth	7.250 in	Cf or Cv for Bending	1.30
Area	25.375 in^2	Cf or Cv for Compression	1.050
Ix	111.148 in^4	Cf or Cv for Tension	1.20
Iy	25.904 in^4	Cm : Wet Use Factor	1.0
		Ct : Temperature Factor	1.0
		Cfu : Flat Use Factor	1.0
		Kf : Built-up columns	1.0 NDS 15.3.2
		Use Cr : Repetitive ?	No

Brace condition for deflection (buckling) along columns :
X-X (width) axis : Fully braced against buckling along X-X Axis
Y-Y (depth) axis : Unbraced Length for X-X Axis buckling = 12.50 ft, K = 1.0

Applied Loads

Service loads entered. Load Factors will be applied for calculations.

AXIAL LOADS . . .
Axial Load at 12.50 ft, D = 0.330, S = 8.30, E = 15.760 k

OK by Inspection

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio = 0.8015 : 1
Load Combination +D+0.750L+0.750S+0.750E+H
Governing NDS Forumla Comp Only, fc/Fc'
Location of max.above base 0.0 ft
At maximum location values are . . . 18818
Applied Axial 18.375 k
Applied Mx 0.0 k-ft
Applied My 0.0 k-ft
Fc : Allowable 903.51 psi

PASS Maximum Shear Stress Ratio = 0.0 : 1
Load Combination +0.60D+E+0.60H
Location of max.above base 12.50 ft
Applied Design Shear 0.0 psi
Allowable Shear 272.0 psi

Maximum SERVICE Lateral Load Reactions . .

Top along Y-Y	0.0 k	Bottom along Y-Y	0.0 k
Top along X-X	0.0 k	Bottom along X-X	0.0 k

Maximum SERVICE Load Lateral Deflections . . .

Along Y-Y 0.0 in at ft above base
for load combination :
Along X-X in at ft above base
for load combination :

Other Factors used to calculate allowable stresses . . .

	Bending	Compression	Tension

Load Combination Results

Load Combination	C_D	C_P	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Maximum Shear Ratios: Stress Ratio	Status	Location

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction @ Base	X-X Axis Reaction @ Top	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Axial Reaction @ Base

Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance

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Engineer: MAR
Project Descr:

Project ID: 15105

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ENERCALC, INC. 1983-2017, Build:6.17.1.16, Ver:6.17.1.16

Wood Column

Lic. # : KW-06002357
Licensee : RUDOW & BERRY

Description : Typ Unit - SW1 - T2 Post from L02 to Roof - 6x8 D.Fir#1

Code References

Calculations per 2012 NDS, IBC 2012, CBC 2013, ASCE 7-10
Load Combinations Used : ASCE 7-10 w/ ASD Wind & EQ

General Information

Analysis Method :	Allowable Stress Design		
End Fixities	Top & Bottom Pinned		
Overall Column Height	13.170 ft		
(Used for non-slender calculations)			
Wood Species	Douglas Fir - Larch		
Wood Grade	No.1		
Fb - Tension	1,200.0 psi	Fv	170.0 psi
Fb - Compr	1,200.0 psi	Ft	825.0 psi
Fc - Prll	1,000.0 psi	Density	31.20 pcf
Fc - Perp	625.0 psi		

E : Modulus of Elasticity . . .	x-x Bending	y-y Bending	Axial
Basic	1,600.0	1,600.0	1,600.0 ksi
Minimum	580.0	580.0	

Wood Section Name	6x8		
Wood Grading/Manuf.	Graded Lumber		
Wood Member Type	Sawn		
Exact Width	5.50 in	Allow Stress Modification Factors	
Exact Depth	7.250 in	Cf or Cv for Bending	1.0
Area	39.875 in^2	Cf or Cv for Compression	1.0
Ix	174.661 in^4	Cf or Cv for Tension	1.0
Iy	100.518 in^4	Cm : Wet Use Factor	1.0
		Ct : Temperature Factor	1.0
		Cfu : Flat Use Factor	1.0
		Kf : Built-up columns	1.0 NDS 15.3.2
		Use Cr : Repetitive ?	No

Brace condition for deflection (buckling) along columns :
X-X (width) axis : Fully braced against buckling along X-X Axis
Y-Y (depth) axis : Unbraced Length for X-X Axis buckling = 13.170 ft, K = 1.0

Applied Loads

Service loads entered. Load Factors will be applied for calculations.

AXIAL LOADS . . .
Axial Load at 13.170 ft, D = 2.526, S = 23.697, E = 15.760 k

(32,561/32,119)*.9744
= .988
OK

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio = 0.9744 : 1
Load Combination +D+0.750L+0.750S+0.750E+H
Governing NDS Forumla Comp Only, fc/Fc'
Location of max.above base 0.0 ft
At maximum location values are . . . 32,561
Applied Axial 32.119 k
Applied Mx 0.0 k-ft
Applied My 0.0 k-ft
Fc : Allowable 826.62 psi

PASS Maximum Shear Stress Ratio = 0.0 : 1
Load Combination +0.60D+E+0.60H
Location of max.above base 13.170 ft
Applied Design Shear 0.0 psi
Allowable Shear 272.0 psi

Maximum SERVICE Lateral Load Reactions . .

Top along Y-Y	0.0 k	Bottom along Y-Y	0.0 k
Top along X-X	0.0 k	Bottom along X-X	0.0 k

Maximum SERVICE Load Lateral Deflections . . .

Along Y-Y 0.0 in at ft above base
for load combination :
Along X-X in at ft above base
for load combination :

Other Factors used to calculate allowable stresses . . .

	Bending	Compression	Tension

Load Combination Results

Load Combination	C_D	C_P	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Maximum Shear Ratios: Stress Ratio	Status	Location

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction @ Base	X-X Axis Reaction @ Top	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Axial Reaction @ Base

Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance

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OHM @ LO1 = 7.181(9.92) = 71.23 k EQ $F_T = \pm 7.12^k$ EQ.
= 2.773(9.92) = 27.51 k W $F_T = \pm 2.75^k$ W.

CUMULATIVE T @ T1 = 15.76 + 7.12 = 22.88 k EQ.
= 3.25 + 2.75 = 6.00 k WIND

TIE AT T1

P = 1094 D + 14192 MAX / 9990 MIN S ± 22880 EQ. / 6000 WIND

= 28898# MAX (SEE NEXT SHT FOR POST)

= .6(1094 + 142 + $\frac{10.92(9.92)}{2}$(10)) − 22880 = −21513 # MAX. TENS.

TIE AT T3

P = 164 D + 5892 S ± 7120 EQ / 2750 WIND

= 11042.5# MAX (SEE NEXT SHT FOR POST)

= .6(164 + $\frac{10.92(9.92)}{2}$(10)) − 7120 = −6337# MAX TENSION

SIMPSON MST37 EA SIDE OF EACH POST
TALLOW = 20320# OK

4881+2200+1085=8166

WALL BELOW GRD @ LO1

V = 7139# EQ, 5828# WIND

U = 709 EQ, 534 WIND, PLF

748

USE 5/8" SHTG BOTH SIDES OF WALL W 10d @ 4" O.C. (MIN)

U ALLOW = 1010 EQ, 1430 WIND OK

SILL CONN

8166

R = $\frac{7139}{1.6(1540)}$ = 3.14

3.31

USE (4) 3/4"ϕ ABS TO STEM

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7.081*9.92 + 8.166*10 = 151.90

15.19

OHM @ GRD = 11.23 + 7.139(10) = 148.62 k EQ. $F_T = \pm 14.86$ k EQ

= 21.51 + 5.828(10) = 80.79 k W. $F_T = \pm 8.58$ k W.

16.35 + 15.19 = 31.54

CUMULATIVE AT T1 = 15.72 + 14.86 = 30.62 k EQ

= 15.72 + 8.58 = 24.34 k WIND

TIE AT T1:

31540

$P = 1730^{\#}_{D} + 14192^{\max}_{8990\ \text{MIN}}{}_{S} + 882_{L} \pm \begin{matrix} 30620\ \text{EQ} \\ 24340\ \text{WIND} \end{matrix}$

36690

$P_{MAX} = 36690^{\#}$ MAX COMPR. (SEE NEXT SHT FOR POST DESIGN)

31540

$P_{MIN} = .6\left[1730 + 642 + \frac{10.92(19.92)}{2}(10)\right] - 30620$

$= -28544^{\#}$ MAX TENSION

29464

TIE AT T3

15190

$P = 1400_{D} + 5892_{S} + 882_{L} \pm \begin{matrix} 14860\ \text{EQ.} \\ 8580\ \text{WIND} \end{matrix}$

17873

$P_{MAX} = 17626^{\#}$ MAX COMPR. (SEE NEXT SHT FOR POST DES.)

15190

$P_{MIN} = .6\left(1400 + \frac{10.92(19.92)}{2}(10)\right) - 14860$

$= -13367^{\#}$ MAX TENSION

13697

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Engineer: MAR
Project Descr:
Project ID: 15105

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Wood Column

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Lic. # : KW-06002357
Licensee : RUDOW & BERRY

Description : Typ Unit - SW1 - T1 Post from GND to L01 - (2)4x8 D.Fir#1

Code References

Calculations per 2012 NDS, IBC 2012, CBC 2013, ASCE 7-10
Load Combinations Used : ASCE 7-10 w/ ASD Wind & EQ

General Information

Analysis Method :	Allowable Stress Design		
End Fixities	Top & Bottom Pinned		
Overall Column Height	10.0 ft		
(Used for non-slender calculations)			
Wood Species	Douglas Fir - Larch		
Wood Grade	No.1		
Fb - Tension	1,200.0 psi	Fv	170.0 psi
Fb - Compr	1,200.0 psi	Ft	825.0 psi
Fc - Prll	1,000.0 psi	Density	31.20 pcf
Fc - Perp	625.0 psi		

E : Modulus of Elasticity . . .	x-x Bending	y-y Bending	Axial
Basic	1,600.0	1,600.0	1,600.0 ksi
Minimum	580.0	580.0	

Wood Section Name	4x8		
Wood Grading/Manuf.	Graded Lumber		
Wood Member Type	Sawn		
Exact Width	3.50 in	Allow Stress Modification Factors	
Exact Depth	7.250 in	Cf or Cv for Bending	1.30
Area	25.375 in^2	Cf or Cv for Compression	1.050
Ix	111.148 in^4	Cf or Cv for Tension	1.20
Iy	25.904 in^4	Cm : Wet Use Factor	1.0
		Ct : Temperature Factor	1.0
		Cfu : Flat Use Factor	1.0
		Kf : Built-up columns	1.0 NDS 15.3.2
		Use Cr : Repetitive ?	No

Brace condition for deflection (buckling) along columns :
X-X (width) axis : Fully braced against buckling along X-X Axis
Y-Y (depth) axis : Unbraced Length for X-X Axis buckling = 10.0 ft, K = 1.0

Applied Loads

Service loads entered. Load Factors will be applied for calculations.

AXIAL LOADS . . .
Axial Load at 10.0 ft, D = 1.730, L = 0.8820, S = 14.192, W = 24.340, E = 30.620 k

DESIGN SUMMARY

36690/36001 * 1.201/2 = 0.612 - OK

Bending & Shear Check Results

FAIL Max. Axial+Bending Stress Ratio = **1.201 : 1**

Load Combination	+D+0.750L+0.750S+0.750E+H
Governing NDS Forumla	Comp Only, fc/Fc'
Location of max.above base	0.0 ft
At maximum location values are . . .	36.690
Applied Axial	36.001 k
Applied Mx	0.0 k-ft
Applied My	0.0 k-ft
Fc : Allowable	1,181.08 psi

PASS Maximum Shear Stress Ratio = **0.0 : 1**

Load Combination	+0.60D+E+0.60H
Location of max.above base	10.0 ft
Applied Design Shear	0.0 psi
Allowable Shear	272.0 psi

Maximum SERVICE Lateral Load Reactions . .

Top along Y-Y	0.0 k	Bottom along Y-Y	0.0 k
Top along X-X	0.0 k	Bottom along X-X	0.0 k

Maximum SERVICE Load Lateral Deflections . . .

Along Y-Y 0.0 in at ft above base
for load combination :
Along X-X in at ft above base
for load combination :

Other Factors used to calculate allowable stresses . . .

	Bending	Compression	Tension

Load Combination Results

Load Combination	C_D	C_P	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Maximum Shear Ratios: Stress Ratio	Status	Location

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction @ Base	X-X Axis Reaction @ Top	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Axial Reaction @ Base

Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance

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Project Title: Copper Crest East
Engineer: MAR
Project Descr:

Project ID: 15105

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Wood Column

File = C:_jobs\15105C~1\ENG\cce-2017.ec6
ENERCALC, INC. 1983-2017, Build:6.17.1.16, Ver:6.17.1.16

Lic. # : KW-06002357 — Licensee : RUDOW & BERRY

Description : Typ Unit - SW1 - T3 Post from GND to L01 - (2)4x8 D.Fir#1

Code References

Calculations per 2012 NDS, IBC 2012, CBC 2013, ASCE 7-10
Load Combinations Used : ASCE 7-10 w/ ASD Wind & EQ

General Information

Analysis Method :	Allowable Stress Design		
End Fixities	Top & Bottom Pinned		
Overall Column Height	10.0 ft		
(Used for non-slender calculations)			
Wood Species	Douglas Fir - Larch		
Wood Grade	No.1		
Fb - Tension	1,200.0 psi	Fv	170.0 psi
Fb - Compr	1,200.0 psi	Ft	825.0 psi
Fc - Prll	1,000.0 psi	Density	31.20 pcf
Fc - Perp	625.0 psi		

E : Modulus of Elasticity . . .	x-x Bending	y-y Bending	Axial
Basic	1,600.0	1,600.0	1,600.0 ksi
Minimum	580.0	580.0	

Wood Section Name	4x8		
Wood Grading/Manuf.	Graded Lumber		
Wood Member Type	Sawn		
Exact Width	3.50 in	Allow Stress Modification Factors	
Exact Depth	7.250 in	Cf or Cv for Bending	1.30
Area	25.375 in^2	Cf or Cv for Compression	1.050
Ix	111.148 in^4	Cf or Cv for Tension	1.20
Iy	25.904 in^4	Cm : Wet Use Factor	1.0
		Ct : Temperature Factor	1.0
		Cfu : Flat Use Factor	1.0
		Kf : Built-up columns	1.0 *NDS 15.3.2*
		Use Cr : Repetitive ?	No

Brace condition for deflection (buckling) along columns :
X-X (width) axis : Fully braced against buckling along X-X Axis
Y-Y (depth) axis : Unbraced Length for X-X Axis buckling = 10.0 ft, K = 1.0

Applied Loads

Service loads entered. Load Factors will be applied for calculations.

AXIAL LOADS . . .
Axial Load at 10.0 ft, D = 1.40, L = 0.8820, S = 5.892, W = 8.580, E = 14.860 k

DESIGN SUMMARY

OK by inspection

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio = **0.5881 : 1**

Load Combination	+D+0.750L+0.750S+0.750E+H
Governing NDS Forumla	Comp Only, fc/Fc'
Location of max.above base	0.0 ft
At maximum location values are . . .	17.873
Applied Axial	17.626 k
Applied Mx	0.0 k-ft
Applied My	0.0 k-ft
Fc : Allowable	1,181.08 psi

PASS Maximum Shear Stress Ratio = **0.0 : 1**

Load Combination	+0.60D+E+0.60H
Location of max.above base	10.0 ft
Applied Design Shear	0.0 psi
Allowable Shear	272.0 psi

Maximum SERVICE Lateral Load Reactions . .

Top along Y-Y	0.0 k	Bottom along Y-Y	0.0 k
Top along X-X	0.0 k	Bottom along X-X	0.0 k

Maximum SERVICE Load Lateral Deflections . . .

Along Y-Y 0.0 in at ft above base
for load combination :
Along X-X in at ft above base
for load combination :

Other Factors used to calculate allowable stresses . . .

	Bending	Compression	Tension

Load Combination Results

Load Combination	C_D	C_P	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Maximum Shear Ratios: Stress Ratio	Status	Location

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction: @ Base	@ Top	Y-Y Axis Reaction: @ Base	@ Top	Axial Reaction: @ Base

Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance

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job name: Copper Crest East
job number: 15105

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designed by: MAR date: 1/17
checked by: date:

FOOTING

151.90 8.166 170.27

Otm @ B.O. FTG = 148.62 + 7.1391(2.75) = 166.03 k EQ.

P_{DL} = 330 + 2526 + 764 + 764 + 636 + 636 + 4.5(1280) + 10.92(199) + 10.92(390 + 305) = 15994 #
(1280: 526; 199: 2173; 390 + 305: 1589)

W_{DL} = 15994 / 10.92 = 1465 PLF

M_{DL} = (330 + 764 + 636)(5.46) + 526(3.21) + 2526(.96) − (764 + 636)(5.46)
= 6.08 ft-k CCW

P_L = 882 + 882 + 10.92(493) = 6602 #, W_L = 605 PLF

M_L = 0

P_{SL} = 8700 + 19890 + 5892 + 5892 + 10.92(3006) = 77,800 #
(3006: 2782)

W_{SL} = 6607 PLF

M_{SL} = (8.3 + 5.892)(5.46) + 19.99(.96) − 5.892(5.46)
= 64.41 ft-k CCW

– SEE NEXT SHT FOR FTG –

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(602) 946-8171

project name: CCE

project no. 15105

designed by: MAR date: Jun-15
checked by: date:

SHEAR WALL FOOTING DESIGN

INPUT DATA : Typical Unit Wall SW1

Allow. Soil Pr. =	2.400	ksf	DL OTM =	6.08	f t - kips	
Fy =	60	ksi	FLR LL OTM =	0	f t - kips	
f 'c =	3000	psi	RF LL OTM =	64.41	f t - kips	
Wall DL =	1.47	klf	SEISMIC OTM =	166.03 ~~f t - kips~~		170.27 See next sheet
Roof LL =	6.67	klf	Footing Length :	16.67	feet	
Floor LL =	0.61	klf	Footing Width :	4.33	feet	
Wall Length =	10.92	feet	Footing Thkness:	18	inches	
Wall Thickness =	8	inches	Footing DL :	1.649	klf	

OUTPUT DATA :

EQ'N 16-11 : DL + .75(FL + RL):

P =	103.0 kips	P_{ult} =	147.5 kips		
OTM =	54.39 ft-kips	OTM_{ult} =	84.59 ft-kips		
e =	0.53 feet	X bar =	N/A feet		
Soil Pr. =	1.70 ksf, max.,	2.43 ksf, ult.		Required Width =	3.06 feet

EQ'N 16-14: DL + .75(FL + RL + .7E)

P =	103.0 kips	P_{ULT} =	147.5 kips		
OTM =	178.9 ft-kips	OTM_{ULT} =	283.82 ft-kips		
e =	1.74 feet	X bar =	N/A feet		
Soil Pr. =	2.32 ksf, max.,	3.32 ksf, ult.		Required Width =	4.19 feet

EQ'N 16-16: 0.6DL + 0.7E

P =	26.1 kips	P (ult) =	31.3 kips		
OTM =	169.7 ft-kips	OTM_{ULT} =	190.33 ft-kips		
e =	6.50 feet	X bar =	1.83 feet		
Soil Pr. =	2.19 ksf, max.,	2.63 ksf, ult.		Required Width =	3.96 feet

Resisting Moment = 362.43 ft-kips Factor of Safety = 3.02

FOOTING REINFORCING:

LONGITUDINAL DIRECTION:

Req'd Unreinf Thickness =	32	inches			
Moment =	16.40 ft-kips/ft	Fb(allow)=	178 psi	fb(act.)=	109 psi
Shear =	0.69 kips/ft	Fv(allow)=	71 psi	fv(act.)=	3 psi

TRANSVERSE DIRECTION:

Req'd Unreinf Thickness =	20	inches			
Moment =	6.84 ft-kips/ft	Fb(allow)=	178 psi	fb(act.)=	127 psi
Shear =	0.54 kips/ft	Fv(allow)=	71 psi	fv(act.)=	3 psi

Reinf. Thickness (if used) =	18	inches		
Longitudinal Steel Required =	0.26	sq.in./ft.	v(longit.) =	31 psi
Transverse Steel Required =	0.11	sq.in./ft.	v(transv.)=	11 psi
			V(allow) =	93.1 psi

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project name: CCE

project no. 15105

designed by: MAR date: 3/6/17
checked by: date:

SHEAR WALL FOOTING DESIGN

INPUT DATA :

Typical Unit Wall SW1

Allow. Soil Pr. =	2.400	ksf	DL OTM =	6.08	f t - kips
Fy =	60	ksi	FLR LL OTM =	0	f t - kips
f 'c =	3000	psi	RF LL OTM =	64.41	f t - kips
Wall DL =	1.47	klf	SEISMIC OTM =	170.27	f t - kips
Roof LL =	6.67	klf	Footing Length :	16.67	feet
Floor LL =	0.61	klf	Footing Width :	4.33	feet
Wall Length =	10.92	feet	Footing Thkness:	18	inches
Wall Thickness =	8	inches	Footing DL :	1.649	klf

OUTPUT DATA :

EQ'N 16-11 : DL + .75(FL + RL):

P =	103.0 kips	P_{ult} =	147.5 kips		
OTM =	54.39 ft-kips	OTM_{ult} =	84.59 ft-kips		
e =	0.53 feet	X bar =	N/A feet		
Soil Pr. =	1.70 ksf, max.,	2.43 ksf, ult.		Required Width =	3.06 feet

EQ'N 16-14: DL + .75(FL + RL + .7E)

P =	103.0 kips	P_{ULT} =	147.5 kips		
OTM =	182.1 ft-kips	OTM_{ULT} =	288.91 ft-kips		
e =	1.77 feet	X bar =	N/A feet		
Soil Pr. =	2.34 ksf, max.,	3.34 ksf, ult.		Required Width =	4.21 feet

EQ'N 16-16: 0.6DL + 0.7E

P =	26.1 kips	P (ult) =	31.3 kips		
OTM =	173.9 ft-kips	OTM_{ULT} =	195.08 ft-kips		
e =	6.67 feet	X bar =	1.67 feet		
Soil Pr. =	2.41 ksf, max.,	2.89 ksf, ult.		Required Width =	4.34 feet

Resisting Moment : 362.43 ft-kips Factor of Safety = 2.95

FOOTING REINFORCING:

LONGITUDINAL DIRECTION:

Req'd Unreinf Thickness =	32	inches			
Moment =	16.51 ft-kips/ft	Fb(allow)=	178 psi	fb(act.)=	110 psi
Shear =	0.69 kips/ft	Fv(allow)=	71 psi	fv(act.)=	3 psi

TRANSVERSE DIRECTION:

Req'd Unreinf Thickness =	20	inches			
Moment =	6.89 ft-kips/ft	Fb(allow)=	178 psi	fb(act.)=	128 psi
Shear =	0.55 kips/ft	Fv(allow)=	71 psi	fv(act.)=	3 psi

Reinf. Thickness (if used) =	18	inches		
Longitudinal Steel Required =	0.26	sq.in./ft.	v(longit.) =	31 psi
Transverse Steel Required =	0.11	sq.in./ft.	v(transv.)=	11 psi
			V(allow) =	93.1 psi

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job name: Copper Crest East
job number: 15105

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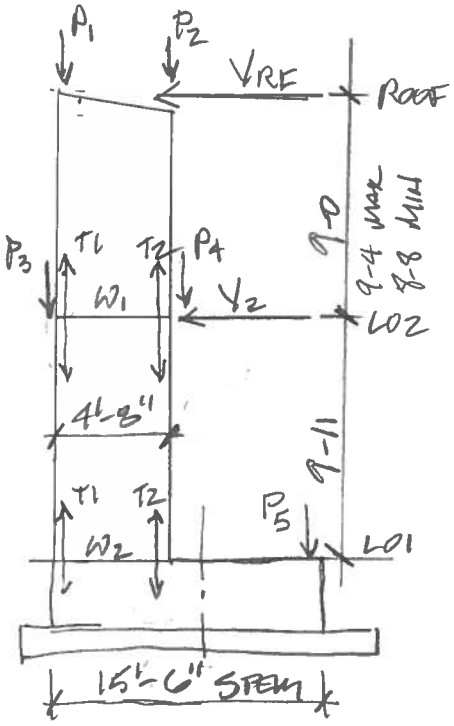
designed by: MAR date: 1/17
checked by: date:

WALL SW-2



V_{RF} = 4924# EQ., 1093# WIND

$P_1 = 117\#_D + 1271\#_{S328} + 1205\#_{S240}$

$P_2 = 2591\#_D + 2141\#_{S328} + 2403\#_{S240}$

V_2 = 607# EQ., 551# WIND

$P_3 = \frac{1}{2}(4.67)(1.17)(25+40) = 68\#_D + 109\#_L$

$P_4 = \frac{1}{2}(8.5)(1.17)(25+40) = 139\#_D + 222\#_L$

$W_1 = 9(10) = 90$ PLF $_{DL}$, PLF

$W_2 = 9.92(10) + 1.17(25+40) = 128_D + 47_L$, PLF

$P_5 = 1669_D + {}^{1610_G}_{1547_S}$

WALL BTWN LO2 & ROOF

$V = 1028$ PLF EQ, 234 PLF WIND

$H/b = 1.93 < 2$

1054

USE 5/8" SHTG BOTH SIDES w/ 10d @ 2" O.C.

$V_{ALLOW} = \frac{2(1740)}{2.0} = 1740$ EQ.

$= \frac{2(2435)}{2.0} = 2435$ W.

OK

SILL CONN.

$M = \frac{4924}{1.6(1001)} = 3.05$

USE MIN (3) 5/8"ϕ BOLTS @ LO2

HEAD CONN

$N_{LAT} = \frac{4924}{670} = 7.35$

$N_{LAT} = \frac{4800}{1.6(640)} = 4.69$

4.81

USE (8) SIMPSON LPT4

USE MIN (5) 5/8"ϕ LAGS

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job number: 15105

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designed by: MAR date: 1/17
checked by: date:

DSM @ LO2 = 4.924 4960 (9) = 44.3 k EQ. $F_T = \frac{44.3}{3.92} = 11.31$ k EQ
= 1,093 (9) = 9.8 k WIND $F_T = 2.51$ k WIND

TIE AT T1:

P = 111 D + 11205 MAX / 1271 MIN ± 11310 EQ / 2510 W.

P_{MAX} = 14663 14446# - SEE NEXT SHT FOR POST DESIGN

$P_{MIN} = .6\left(111 + \frac{4.16}{2}(90)\right) - 11310 = -10718$# MAX TENSION

USE SIMPSON MST37 EA SIDE OF EA POST

$T_{allow} = 4(5080) = 20160$# ok

TIE AT T2

P = 2591 D + 27442 MAX / 24037 MIN ± 11310 EQ. / 2510 W.

P_{MAX} = 30063 30460# - SEE NEXT SHTS FOR POST DESIGN

$P_{MIN} = .6\left(2591 + \frac{4.16}{2}(90)\right) - 11310 = -9629$# MAX TENS.

USE SAME AS T1

Controls due to lower duration factor

WALL BTWN L01 & L02

V = 5531 5259# EQ, 1644# WIND

v = 1126 PLF EQ, 352 PLF WIND

h/b_s = 1184 2.12

USE 5/8" SHT'G B.S W/ 8d @ 2" O.C.

v_{allow} = 1740 EQ / 2435 WIND ok

SILL CONN'T

N = $\frac{5259}{1.6(1540)}$ = 2.13 2.24

USE (3) 3/4" ϕ AB's TO STEM

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Project Title: Copper Crest East
Engineer: MAR
Project Descr:
Project ID: 15105

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Wood Column

File = C:_jobs\15105C~1\ENG\cce-2017.ec6
ENERCALC, INC. 1983-2017, Build:6.17.1.16, Ver:6.17.1.16

Lic. # : KW-06002357 — Licensee : RUDOW & BERRY

Description : Typ Unit - SW2 - T1 Post from L02 to Roof - (2)3x8 D.Fir#1

Code References

Calculations per 2012 NDS, IBC 2012, CBC 2013, ASCE 7-10
Load Combinations Used : ASCE 7-10 w/ ASD Wind & EQ

General Information

Analysis Method :	Allowable Stress Design			Wood Section Name	3x8
End Fixities	Top & Bottom Pinned			Wood Grading/Manuf.	Graded Lumber
Overall Column Height	9.330 ft			Wood Member Type	Sawn
(Used for non-slender calculations)					
Wood Species	Douglas Fir - Larch			Exact Width	5.0 in
Wood Grade	No.1			Exact Depth	7.250 in
Fb - Tension	1,200.0 psi	Fv	170.0 psi	Area	36.250 in^2
Fb - Compr	1,200.0 psi	Ft	825.0 psi	Ix	158.783 in^4
Fc - Prll	1,000.0 psi	Density	31.20 pcf	Iy	75.521 in^4
Fc - Perp	625.0 psi				

E : Modulus of Elasticity . . .	x-x Bending	y-y Bending	Axial
Basic	1,600.0	1,600.0	1,600.0 ksi
Minimum	580.0	580.0	

Allow Stress Modification Factors

Cf or Cv for Bending	1.20
Cf or Cv for Compression	1.050
Cf or Cv for Tension	1.20
Cm : Wet Use Factor	1.0
Ct : Temperature Factor	1.0
Cfu : Flat Use Factor	1.0
Kf : Built-up columns	1.0 NDS 15.3.2
Use Cr : Repetitive ?	No

Brace condition for deflection (buckling) along columns :
X-X (width) axis : Fully braced against buckling along X-X Axis
Y-Y (depth) axis : Unbraced Length for X-X Axis buckling = 9.330 ft, K = 1.0

Applied Loads

Service loads entered. Load Factors will be applied for calculations.

AXIAL LOADS . . .
Axial Load at 9.330 ft, D = 0.7770, S = 7.205, E = 11.020 k

DESIGN SUMMARY

OK by inspection

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio = **0.3173 : 1**

Load Combination	+D+0.750L+0.750S+0.750E+H
Governing NDS Forumla	Comp Only, fc/Fc'
Location of max.above base	0.0 ft
At maximum location values are . . .	14.663
Applied Axial	14.446 k
Applied Mx	0.0 k-ft
Applied My	0.0 k-ft
Fc : Allowable	1,255.76 psi

PASS Maximum Shear Stress Ratio = **0.0 : 1**

Load Combination	+0.60D+E+0.60H
Location of max.above base	9.330 ft
Applied Design Shear	0.0 psi
Allowable Shear	272.0 psi

Maximum SERVICE Lateral Load Reactions . .

Top along Y-Y	0.0 k	Bottom along Y-Y	0.0 k
Top along X-X	0.0 k	Bottom along X-X	0.0 k

Maximum SERVICE Load Lateral Deflections . . .

Along Y-Y 0.0 in at ft above base
for load combination :
Along X-X in at ft above base
for load combination :

Other Factors used to calculate allowable stresses . . .

	Bending	Compression	Tension

Load Combination Results

Load Combination	C_D	C_P	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Maximum Shear Ratios: Stress Ratio	Status	Location

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction @ Base	X-X Axis Reaction @ Top	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Axial Reaction @ Base

Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance

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Project Title: Copper Crest East
Engineer: MAR
Project Descr:

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Wood Column

File = C:_jobs\15105C~1\ENG\cce-2017.ec6
ENERCALC, INC. 1983-2017, Build:6.17.1.16, Ver:6.17.1.16
Lic. # : KW-06002357 Licensee : RUDOW & BERRY

Description : Typ Unit - SW2 - T2 Post from L02 to Roof - (2)3x8 D.Fir#1

Code References

Calculations per 2012 NDS, IBC 2012, CBC 2013, ASCE 7-10
Load Combinations Used : ASCE 7-10 w/ ASD Wind & EQ

General Information

Analysis Method : Allowable Stress Design
End Fixities : Top & Bottom Pinned
Overall Column Height : 8.670 ft
(Used for non-slender calculations)

Wood Species : Douglas Fir - Larch
Wood Grade : No.1

Fb - Tension	1,200.0 psi	Fv	170.0 psi
Fb - Compr	1,200.0 psi	Ft	825.0 psi
Fc - Prll	1,000.0 psi	Density	31.20 pcf
Fc - Perp	625.0 psi		

E : Modulus of Elasticity . . .	x-x Bending	y-y Bending	Axial
Basic	1,600.0	1,600.0	1,600.0 ksi
Minimum	580.0	580.0	

Wood Section Name : 3x8
Wood Grading/Manuf. : Graded Lumber
Wood Member Type : Sawn

Exact Width	5.0 in	Area	36.250 in^2
Exact Depth	7.250 in	Ix	158.783 in^4
		Iy	75.521 in^4

Allow Stress Modification Factors

Cf or Cv for Bending	1.20
Cf or Cv for Compression	1.050
Cf or Cv for Tension	1.20
Cm : Wet Use Factor	1.0
Ct : Temperature Factor	1.0
Cfu : Flat Use Factor	1.0
Kf : Built-up columns	1.0 NDS 15.3.2
Use Cr : Repetitive ?	No

Brace condition for deflection (buckling) along columns :
X-X (width) axis : Fully braced against buckling along X-X Axis
Y-Y (depth) axis : Unbraced Length for X-X Axis buckling = 8.670 ft, K = 1.0

Applied Loads

Service loads entered. Load Factors will be applied for calculations.

AXIAL LOADS . . .
Axial Load at 8.670 ft, D = 2.591, S = 27.472, E = 11.020 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio = 0.7986 : 1

Load Combination	+D+S+H
Governing NDS Forumla	Comp Only, fc/Fc'
Location of max.above base	8.670 ft
At maximum location values are . . .	
Applied Axial	30.063 k
Applied Mx	0.0 k-ft
Applied My	0.0 k-ft
Fc : Allowable	1,038.53 psi

Still controls due to lower Cd

PASS Maximum Shear Stress Ratio = 0.0 : 1

Load Combination	+0.60D+E+0.60H
Location of max.above base	8.670 ft
Applied Design Shear	0.0 psi
Allowable Shear	272.0 psi

Maximum SERVICE Lateral Load Reactions . .

Top along Y-Y	0.0 k	Bottom along Y-Y	0.0 k
Top along X-X	0.0 k	Bottom along X-X	0.0 k

Maximum SERVICE Load Lateral Deflections . . .

Along Y-Y 0.0 in at ft above base
for load combination :
Along X-X in at ft above base
for load combination :

Other Factors used to calculate allowable stresses . . .

	Bending	Compression	Tension

Load Combination Results

Load Combination	C_D	C_P	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Maximum Shear Ratios: Stress Ratio	Status	Location

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction @ Base	X-X Axis Reaction @ Top	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Axial Reaction @ Base

Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance

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job name: Copper Crest East
job number: 15105

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designed by: MAR date: 1/17
checked by: date:

44.3 5.531 99.17

OTM @ LOI/CMU = 47.12 + 5.259(9.92) = 95.177 lb EQ
= 9.18 + 1.644(11) = 26.11 lb WIND

99.17

$F_T = \frac{\pm 95.177}{3.92} = \pm 24.32^{k}$

25.30

TIE AT T1

$P = 845D + 109L + \frac{1205\ MAX}{1271\ MIN} \leq \pm 24320$ EQ. 25300

$P_{MAX} = 26145^{\#}$ MAX - SEE NEXT SHT FOR POST DESIGN

$P_{MIN} = .6\left(845 + \frac{4.167}{2}(90+128)\right) - 24320 = -23502^{\#}$ MAX T.

25300 -24487

TIE AT T2

$P = 2730D + 222L + \frac{27472\ MAX}{24637\ MIN} \leq \pm 24320$ EQ 25300

$P_{MAX} = 42475^{\#}$ - SEE NEXT SHTS FOR POST DESIGN

$P_{MIN} = .6\left(2730 + \frac{4.167}{2}(218)\right) - 24320 = -22376^{\#}$ MAX T.

25300 -23356

USE CARBON-BAR SYSTEM @ T1 & T2

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Project Title: Copper Crest East
Engineer: MAR
Project Descr:
Project ID: 15105

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Wood Column

File = C:_jobs\15105C~1\ENG\ccs-2017.ec6
ENERCALC, INC. 1983-2017, Build:6.17.1.16, Ver:6.17.1.16

Lic. # : KW-06002357 — Licensee : RUDOW & BERRY

Description : Typ Unit - SW2 - T1 Post from L01 to L02 - (2)3x8 D.Fir#1

Code References

Calculations per 2012 NDS, IBC 2012, CBC 2013, ASCE 7-10
Load Combinations Used : ASCE 7-10 w/ ASD Wind & EQ

General Information

Analysis Method :	Allowable Stress Design		
End Fixities	Top & Bottom Pinned		
Overall Column Height	9.920 ft		
(Used for non-slender calculations)			
Wood Species	Douglas Fir - Larch		
Wood Grade	No.1		
Fb - Tension	1,200.0 psi	Fv	170.0 psi
Fb - Compr	1,200.0 psi	Ft	825.0 psi
Fc - Prll	1,000.0 psi	Density	31.20 pcf
Fc - Perp	625.0 psi		

E : Modulus of Elasticity . . .	x-x Bending	y-y Bending	Axial
Basic	1,600.0	1,600.0	1,600.0 ksi
Minimum	580.0	580.0	

Wood Section Name	3x8		
Wood Grading/Manuf.	Graded Lumber		
Wood Member Type	Sawn		
Exact Width	5.0 in	Allow Stress Modification Factors	
Exact Depth	7.250 in	Cf or Cv for Bending	1.20
Area	36.25 in^2	Cf or Cv for Compression	1.050
Ix	158.783 in^4	Cf or Cv for Tension	1.20
Iy	75.521 in^4	Cm : Wet Use Factor	1.0
		Ct : Temperature Factor	1.0
		Cfu : Flat Use Factor	1.0
		Kf : Built-up columns	1.0 NDS 15.3.2
		Use Cr : Repetitive ?	No

Brace condition for deflection (buckling) along columns :
X-X (width) axis : Fully braced against buckling along X-X Axis
Y-Y (depth) axis : Unbraced Length for X-X Axis buckling = 9.920 ft, K = 1.0

Applied Loads

Service loads entered. Load Factors will be applied for calculations.

AXIAL LOADS . . .
Axial Load at 9.920 ft, D = 0.8450, L = 0.1090, S = 7.205, E = 24.320 k

DESIGN SUMMARY

OK by inspection

Bending & Shear Check Results

PASS	Max. Axial+Bending Stress Ratio =	0.5833 : 1
	Load Combination	+D+E+H
	Governing NDS Forumla	Comp Only, fc/Fc'
	Location of max.above base	0.0 ft
	At maximum location values are . . .	26.145
	Applied Axial	25.165 k
	Applied Mx	0.0 k-ft
	Applied My	0.0 k-ft
	Fc : Allowable	1,190.14 psi
PASS	Maximum Shear Stress Ratio =	0.0 : 1
	Load Combination	+0.60D+E+0.60H
	Location of max.above base	9.920 ft
	Applied Design Shear	0.0 psi
	Allowable Shear	272.0 psi

Maximum SERVICE Lateral Load Reactions . .

Top along Y-Y	0.0 k	Bottom along Y-Y	0.0 k
Top along X-X	0.0 k	Bottom along X-X	0.0 k

Maximum SERVICE Load Lateral Deflections . . .

Along Y-Y 0.0 in at ft above base
for load combination :
Along X-X in at ft above base
for load combination :

Other Factors used to calculate allowable stresses . . .

	Bending	Compression	Tension

Load Combination Results

Load Combination	C_D	C_P	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Maximum Shear Ratios: Stress Ratio	Status	Location

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction @ Base	X-X Axis Reaction @ Top	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Axial Reaction @ Base

Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance

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Project Title: Copper Crest East
Engineer: MAR
Project Descr:

 Project ID: 15105 324

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Wood Column

File = C:_jobs\15105C~1\ENG\cce-2017.ec6
ENERCALC, INC. 1983-2017, Build:6.17.1.16, Ver:6.17.1.16

Lic. # : KW-06002357 Licensee : RUDOW & BERRY

Description : Typ Unit - SW2 - T2 Post from L01 to L02 - (2)3x8 D.Fir#1

Code References

Calculations per 2012 NDS, IBC 2012, CBC 2013, ASCE 7-10
Load Combinations Used : ASCE 7-10 w/ ASD Wind & EQ

General Information

Analysis Method :	Allowable Stress Design		
End Fixities	Top & Bottom Pinned		
Overall Column Height	9.920 ft		
(Used for non-slender calculations)			
Wood Species	Douglas Fir - Larch		
Wood Grade	No.1		
Fb - Tension	1,200.0 psi	Fv	170.0 psi
Fb - Compr	1,200.0 psi	Ft	825.0 psi
Fc - Prll	1,000.0 psi	Density	31.20 pcf
Fc - Perp	625.0 psi		

E : Modulus of Elasticity . . .	x-x Bending	y-y Bending	Axial
Basic	1,600.0	1,600.0	1,600.0 ksi
Minimum	580.0	580.0	

Wood Section Name	3x8		
Wood Grading/Manuf.	Graded Lumber		
Wood Member Type	Sawn		
Exact Width	5.0 in	Allow Stress Modification Factors	
Exact Depth	7.250 in	Cf or Cv for Bending	1.20
Area	36.25 in^2	Cf or Cv for Compression	1.050
Ix	158.783 in^4	Cf or Cv for Tension	1.20
Iy	75.521 in^4	Cm : Wet Use Factor	1.0
		Ct : Temperature Factor	1.0
		Cfu : Flat Use Factor	1.0
		Kf : Built-up columns	1.0 *NDS 15.3.2*
		Use Cr : Repetitive ?	No

Brace condition for deflection (buckling) along columns :
X-X (width) axis : Fully braced against buckling along X-X Axis
Y-Y (depth) axis : Unbraced Length for X-X Axis buckling = 9.920 ft, K = 1.0

Applied Loads

Service loads entered. Load Factors will be applied for calculations.

AXIAL LOADS . . .
Axial Load at 9.920 ft, D = 2.730, L = 0.2220, S = 27.472, E = 24.320 k

DESIGN SUMMARY

Bending & Shear Check Results

42475/41741 * .9675 = .9845 OK

PASS Max. Axial+Bending Stress Ratio = **0.9675 : 1**

Load Combination	+D+0.750L+0.750S+0.750E+H
Governing NDS Forumla	Comp Only, fc/Fc'
Location of max.above base	0.0 ft
At maximum location values are . . .	42.475
Applied Axial	41.741 k
Applied Mx	0.0 k-ft
Applied My	0.0 k-ft
Fc : Allowable	1,190.14 psi

PASS Maximum Shear Stress Ratio = **0.0 : 1**

Load Combination	+0.60D+E+0.60H
Location of max.above base	9.920 ft
Applied Design Shear	0.0 psi
Allowable Shear	272.0 psi

Maximum SERVICE Lateral Load Reactions . .

Top along Y-Y	0.0 k	Bottom along Y-Y	0.0 k
Top along X-X	0.0 k	Bottom along X-X	0.0 k

Maximum SERVICE Load Lateral Deflections . . .

Along Y-Y 0.0 in at ft above base
for load combination :
Along X-X in at ft above base
for load combination :

Other Factors used to calculate allowable stresses . . .

	Bending	Compression	Tension

Load Combination Results

Load Combination	C_D	C_P	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Maximum Shear Ratios: Stress Ratio	Status	Location

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction @ Base	X-X Axis Reaction @ Top	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Axial Reaction @ Base

Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance

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job name: Copper Crest East
job number: 15105

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designed by: MAR date: 1/17
checked by: date:

OTM @ B.O.FTG = 99.17 + 5.531 (4) = 121.29 k
STEM L = 15'-6"

$P_{DL} = 177 + 2591 + 68 + 139 + 4.67(218) + 1669 = 6262^{\#}$
(1018)

$W_{DL} = 404$ PLF

$M_{DL} = (177+68)(1.67) + (2591+139)(2.92) + 1.018(5.3) + 1.669(-0.92)$
$= 8.30^{k}$

$P_L = 109 + 222 + 4.67(47) = 550^{\#}$
(219)

$W_L = 35$ PLF

$M_L = 109(1.67) + 222(2.92) + 219(5.3) = 2.105^{k}$

$P_S = 1.205 + 24.037 + 16.702^{\#} = 41.948^{\#}$

$W_S = 3093$ PLF

$M_S = 1.205(1.67) + 24.037(2.92) + 16.702(-0.92)$
$= 9.84^{k}$

OR

$P_S = 1271 + 27.472 + 15.478 = 44.221^{\#}$

$W_S = 2853$ PLF

$M_S = 1.271(1.67) + 27.472(2.92) + 15.478(-0.92)$
$= -17.14^{k}$

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project name: CCE
designed by: MAR date: Jun-15
checked by: date:

project no. 15105

SHEAR WALL FOOTING DESIGN

INPUT DATA : Typical Unit Wall SW2

Allow. Soil Pr. =	2.400	ksf	DL OTM =	8.30	f t - kips	
Fy =	60	ksi	FLR LL OTM =	2.65	f t - kips	
f 'c =	3000	psi	RF LL OTM =	9.84	f t - kips	
Wall DL =	0.40	klf	SEISMIC OTM =	116.41	f t - kips	121.29 See next sheet
Roof LL =	3.09	klf	Footing Length :	17.50	feet	
Floor LL =	0.04	klf	Footing Width :	2.67	feet	
Wall Length =	15.50	feet	Footing Thkness:	12	inches	
Wall Thickness =	8	inches	Footing DL :	1.422	klf	

OUTPUT DATA :

EQ'N 16-11 : DL + .75(FL + RL):

P =	67.5 kips	P_{ult} =	95.6 kips		
OTM =	17.67 ft-kips	OTM_{ult} =	24.95 ft-kips		
e =	0.26 feet	X bar =	N/A feet		
Soil Pr. =	1.57 ksf, max.,	2.23 ksf, ult.		Required Width =	1.75 feet

EQ'N 16-14: DL + .75(FL + RL + .7E)

P =	67.5 kips	P_{ULT} =	95.6 kips		
OTM =	105.0 ft-kips	OTM_{ULT} =	164.64 ft-kips		
e =	1.56 feet	X bar =	N/A feet		
Soil Pr. =	2.22 ksf, max.,	3.14 ksf, ult.		Required Width =	2.46 feet

EQ'N 16-16: 0.6DL + 0.7E

P =	18.7 kips	P (ult) =	22.4 kips		
OTM =	121.4 ft-kips	OTM_{ULT} =	136.36 ft-kips		
e =	6.50 feet	X bar =	2.25 feet		
Soil Pr. =	2.07 ksf, max.,	2.48 ksf, ult.		Required Width =	2.30 feet

Resisting Moment = 272.49 ft-kips Factor of Safety = 3.15

FOOTING REINFORCING:

LONGITUDINAL DIRECTION:

Req'd Unreinf Thickness =	32	inches			
Moment =	2.74 ft-kips/ft	Fb(allow)=	178 psi	fb(act.)=	18 psi
Shear =	0.00 kips/ft	Fv(allow)=	71 psi	fv(act.)=	0 psi

TRANSVERSE DIRECTION:

Req'd Unreinf Thickness =	20	inches			
Moment =	2.66 ft-kips/ft	Fb(allow)=	178 psi	fb(act.)=	49 psi
Shear =	0.00 kips/ft	Fv(allow)=	71 psi	fv(act.)=	0 psi

Reinf. Thickness (if used) =	12	inches		
Longitudinal Steel Required =	0.07	sq.in./ft.	v(longit.) =	9 psi
Transverse Steel Required =	0.07	sq.in./ft.	v(transv.)=	9 psi
			V(allow) =	93.1 psi

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project name: CCE

project no. 15105

designed by: MAR date: Jun-15

checked by: date:

SHEAR WALL FOOTING DESIGN

INPUT DATA :

Typical Unit Wall SW2

Allow. Soil Pr. =	2.400	ksf	DL OTM =	8.30	f t - kips
Fy =	60	ksi	FLR LL OTM =	2.65	f t - kips
f 'c =	3000	psi	RF LL OTM =	9.84	f t - kips
Wall DL =	0.40	klf	SEISMIC OTM =	121.29	f t - kips
Roof LL =	3.09	klf	Footing Length :	17.50	feet
Floor LL =	0.04	klf	Footing Width :	2.67	feet
Wall Length =	15.50	feet	Footing Thkness:	12	inches
Wall Thickness =	8	inches	Footing DL :	1.422	klf

OUTPUT DATA :

EQ'N 16-11 : DL + .75(FL + RL):

P =	67.5 kips	P_{ult} =	95.6 kips		
OTM =	17.67 ft-kips	OTM_{ult} =	24.95 ft-kips		
e =	0.26 feet	X bar =	N/A feet		
Soil Pr. =	1.57 ksf, max.,	2.23 ksf, ult.		Required Width =	1.75 feet

EQ'N 16-14: DL + .75(FL + RL + .7E)

P =	67.5 kips	P_{ULT} =	95.6 kips		
OTM =	108.6 ft-kips	OTM_{ULT} =	170.5 ft-kips		
e =	1.61 feet	X bar =	N/A feet		
Soil Pr. =	2.24 ksf, max.,	3.17 ksf, ult.		Required Width =	2.49 feet

EQ'N 16-16: 0.6DL + 0.7E

P =	18.7 kips	P (ult) =	22.4 kips		
OTM =	126.3 ft-kips	OTM_{ULT} =	141.82 ft-kips		
e =	6.76 feet	X bar =	1.99 feet		
Soil Pr. =	2.34 ksf, max.,	2.81 ksf, ult.		Required Width =	2.61 feet

Resisting Moment = 272.49 ft-kips Factor of Safety = 3.03

FOOTING REINFORCING:

LONGITUDINAL DIRECTION:

Req'd Unreinf Thickness =	32 inches				
Moment =	2.78 ft-kips/ft	Fb(allow)=	178 psi	fb(act.)=	19 psi
Shear =	0.00 kips/ft	Fv(allow)=	71 psi	fv(act.)=	0 psi

TRANSVERSE DIRECTION:

Req'd Unreinf Thickness =	20 inches				
Moment =	2.69 ft-kips/ft	Fb(allow)=	178 psi	fb(act.)=	50 psi
Shear =	0.00 kips/ft	Fv(allow)=	71 psi	fv(act.)=	0 psi

Reinf. Thickness (if used) =	12 inches		
Longitudinal Steel Required =	0.07 sq.in./ft.	v(longit.) =	9 psi
Transverse Steel Required =	0.07 sq.in./ft.	v(transv.)=	9 psi
		V(allow) =	93.1 psi

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checked by: date:

WALLS AT UNIT 62R

WALL SW1

6834
2113
-4214
4733

V_{ROOF} = 6624# EQ, 1364# WIND
V_{LO2} = 2377# EQ, 1734# WIND
V_{LO1} = 637# EQ, 1027# WIND
V_{TOT} = 9638# EQ, 4125# WIND — EQ CONTROLS DESIGN BY INSPECTION

ROOF TO LO2

H = 11'-11"
L = 6'-4"

1080
V_{MAR} = 1046 EQ, 215 W
H/L = 1.88 < 2 ok

USE 5/8" SHT'G B/S
NAILED 2" O.C.

V_{ALLOW} = 1740 EQ
= 2435 W
ok

HEAD COLL'S:

6834
13.3
$N_{A34} = \frac{6624}{515} = 12.9$

USE MIN 14 SIMPSON A34
(OR LPT4 W V_{ALLOW} = 670# EQ)

6834
6.57
$N_{1/2"\phi} = \frac{6624}{1.6(650)} = 6.38$

USE MIN (8) 1/2"ϕ BOLTS TO W10 BM

6834
6.67
$N_{LAG} = \frac{6624}{1.6(640)} = 6.47$

USE MIN (7) 5/8"ϕ LAGS TO BLK'G

SILL COLL

6834
4.24
$N_{5/8"\phi} = \frac{6624}{1.6(1007)} = 4.11$

USE (4) 5/8"ϕ BOLTS

Δ = 2.8% ok

Delta = 6% - OK due to additional HSS post conn's

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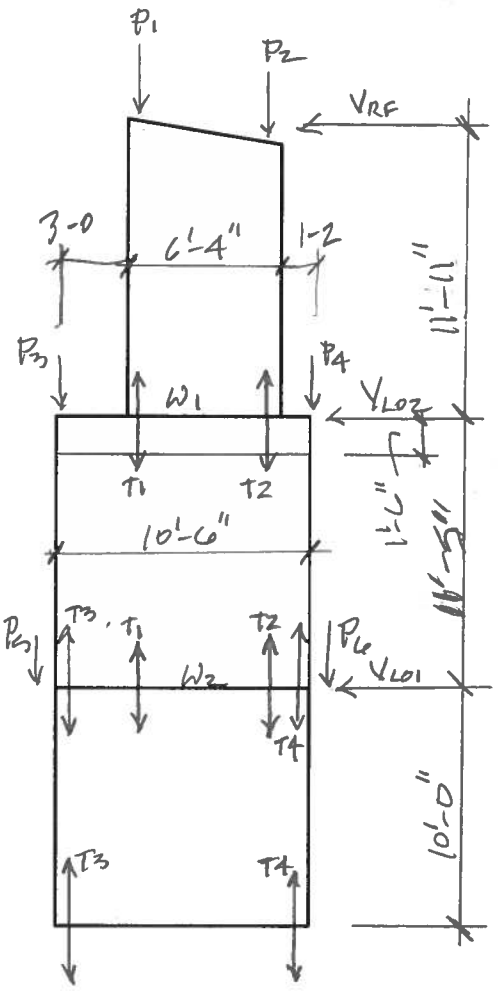
designed by: MAR
checked by:

date: 1/17
date:

WALL SW1 (CONT'D)



$P_1 = -497_D - 825_{S\,SLOPE} - 308_{S\,240}$

$P_2 = 9398_D + 8742_{L\,MAX\ OR\ S\ MIN}$

$P_3 = 1451_D + 912_L + 783_{S\,SLOPE}$

$P_4 = 4559_D + 2873_L + 2456_{S\,SLOPE}$

$W_1 = 515_D + 343_L + 289_{S\,SLOPE}$, PLF

$P_5 = 704_D + 112_L$

$P_6 = 3589_D + 543_L$

$W_2 = 443_D + 70_L$

ROOF TO LOZ (CONT'D)

$M_{@\ LOZ} = \frac{6.624}{6.834}(11.92) = \frac{78,976}{81.46}$ FT-K

$F_T = \frac{78.96^K}{5.71'} = \pm 13.83^K$ EQ.
$\frac{81.46}{14.27}$

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ROOF TO LOZ (CONT'D)

TIE AT T1:

$P = -497\text{D} - \frac{8215 \text{ MAX}}{308 \text{ MIN}} \leq \pm 13830$ EQ (14270)

$P_{MAX} = 13773^{\#}$ (13773) - SEE HEELT SHT FOR POST DESIGN

$P_{MIN} = \left(-497 + \frac{6333(12)(10)}{2}\right) - .75(-8215 - 13830)$ (14270)

$= -16650^{\#}$ MAX TENSION (-16981)

USE HSS POST FULL HT

TIE AT T2:

$P = 9398\text{D} + \frac{8412 \text{ MAX}}{6245 \text{ MIN}} \leq \pm 13830$ EQ. (14270)

$P_{MAX} = 9681\text{D}^{\#}$ - SEE HEELT SHT FOR POST DESIGN

$P_{MIN} = .6\left(9398 + \frac{6333(12)(10)}{2}\right) - 13830 = -8403^{\#}$ MAX. T. (-8403)

USE HSS POST FULL HT

WALL FROM LOZ TO LO1 (8947 - prev. design OK)

$V = 9001^{\#}$ EQ
$L_{WALL} = 10'-6"$
} $v = 857$ PLF

5/8" SHT'G B.S. w/ 10d @ 2" O.C.

$v_{ALLOW} = 1140$ EQ, OK

SILL CONN'S

$N_{5/8"\phi} = \frac{9001}{1.6(1001)} = 5.59$

USE MIN (6) 5/8"ϕ BOLTS

OTM @ TENSION $= 9.001(1.5) = 13.50^{k}$

$F_T(MAX) = 13.5\left(\frac{1}{101}\right) = 1.35^{k}$

SIMPSON LSTA24 B.S.

$T_{ALLOW} = 2(1.235) = 2.47^{k}$ OK

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8.947

OHM @ LOI = 9.001(11.42) = 102.79 lb 102.17

previous design OK

$F_T = \pm \frac{102.79}{10.124'} = \pm 10.07^{lb}$

TIE AT T3:

P = 1451 lb + 912 L + 7836 s ± 10007 EQ

P_{MAX} = 15517# — SEE T4 FOR POST DESIGN

$P_{MIN} = .6\left(1451 + \frac{10.5}{2}(515)\right) - 10007$

= −7514# MAX TENSION

SIMPSON MGT37 B.S. OF POST

$T_{ALLOW} = (2)(5080) = 10160^{\#}$ OK

TIE AT T4

P = 4539 lb + 2073 L + 2456 s ± 10007 EQ

P_{MAX} = 32473# — SEE HEADER FOR POST DESIGN

$P_{MIN} = .6\left(4539 + \frac{10.5}{2}(515)\right) - 10007$

= −5661# MAX TENSION

SIMPSON MGT37 B.S. OF POST

T_{ALLOW} = 10160# OK

4733# - prev. design OK

WALL FROM LOI TO CMU

V = 9638# EQ
L = 10'-6"
} v = 918 PLF

5/8" SHT'G B.S. W/ 10d @ 2" O.C.

v_{ALLOW} = 1740 EQ. OK

SILL CONN

$N_{3/4"\phi} = \frac{9638}{1.6(1540)} = 3.91$

USE MIN (4) 3/4"φ BOLTS TO STEM

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STRAPS AT HSS/PL'S

$T_{MAX} = 2371\left(\frac{12}{32}\right) =$ ...

SIMPSON LSTA 18
EA. SIDE OF WALL

$T_{ALLOW} = 2(1235)\left(\frac{2}{3}\right) = 1647^{\#}$ ok

DRAG AT BMS @ E.O. WALL

$T_{MAX} = 2371\left(\frac{15.5}{32}\right)\left(\frac{4}{1.3}\right) = 3543^{\#}$

SIMPSON MST 37

$T_{ALLOW} = 5080^{\#}$

OTM @ GRID = 102.79 + 9.638(10) = 199.17 ^{k}

102.17 | 4.733 | 149.50 — previous design OK

$F_T = \pm \frac{199.17}{10.08'} = \pm 19.76^{k}$

TIE AT T3

$P = 2155_D + 2098_L + 7836_S \pm 19760$

$P_{MAX} = 24425^{\#}$ – SEE T4 FOR POST DESIGN

$P_{MIN} = .6\left(2155 + \frac{10.5}{2}(515 + 443 + 21.42(10))\right) - 19760$

$= -14775^{\#}$ MAX T.

SIMPSON HDU14-SDS2.5

$T_{ALLOW} = 14445^{\#}$

$\Delta = 2.3\%$ ok

TIE AT T4

$P = 8128_D + 8109_L + 24565_S \pm 19760$

$P_{MAX} = 41453^{\#}$ – SEE NEXT SHT FOR POST DESIGN

$P_{MIN} = .6\left(9128 + \frac{10.6}{2}(515 + 443 + 214)\right) - 19760$

$= 11191^{\#}$ MAX T.

SIMPSON HDU14-SDS2.5

$T_{ALLOW} = 14445^{\#}$ ok

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81.46 8.947 4.733

OTM @ B.O.F.G. = 78.96 + 9.001(11.42) + 9.639(10 + 2.125)

= 299.82 ^k

241.61 ← previous design OK

L_{WALL} = 10'-6", CTR @ 5'-3"

P_{DL} = -497 + 9398 + 1451 + 45391 + 6773 (760) + 204 + 3589 + 10.5(515 + 443 + 215) (12317)

= 32260 #

$W_{DL} = \frac{32260}{10.92}$ = 2954 PLF

M_{DL} = -.497(-2.125) + 9.398(4.08) + 1.451(-5.125) + 45.39(5.125) + 0.76(.92) + 0.704(-5.125) + 3.589(5.125) = 71.52 ^k

P_L = 912 + 2073 + 1126 + 5436 + 10.5(343 + 708) = 21243 #

$A_{LL} = \frac{21243}{40}$ = 531 FT² LL = 28 PSF

$W_L = \frac{21243}{10.5}\left(\frac{28}{40}\right)$ = 1416 PLF

$M_L = \left[(912 + 1126)(-5.125) + (2073 + 5436)(5.125)\right]\frac{28}{40}$ = 22.09 ^k

P_S = -8215 + 87412 + 7836 + 24565 + 10.5(2897) = 111.63 ^k

W_S = 10631 PLF

M_S = -8.215(-2.125) + 87.412(4.08) + 7.836(-5.125) + 24.565(5.125)

= 462.95 ^k

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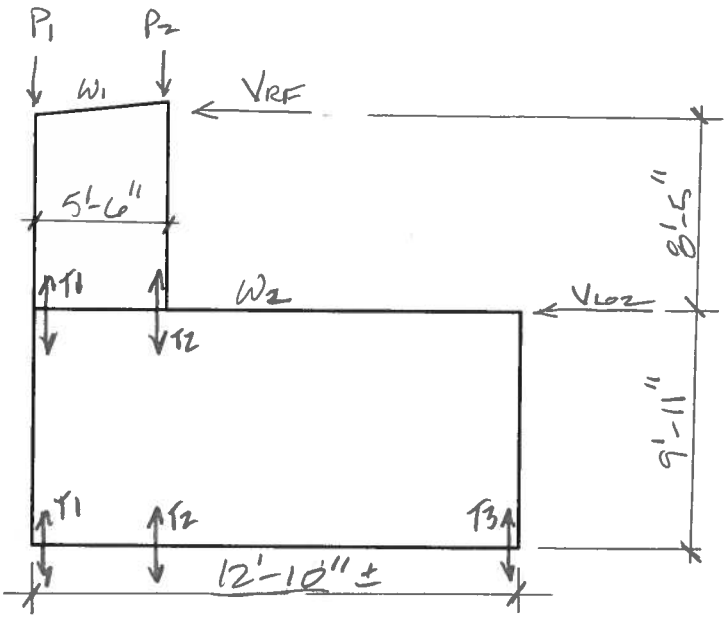
UNIT G2R - WALL SW2



$V_{RF} = 5798^{\#}$ EQ
$= 5653^{\#}$ EQ
$= 1287^{\#}$ WIND

$P_1 = 204^{\#}_D + \frac{17868 \text{ MIN}}{15542 \text{ MIN}}$ S

$P_2 = 39639^{\#}_D + \frac{35640 \text{ MIN}}{34213 \text{ MIN}}$ S

$W_1 = 2100_D + 2508_S$

$V_{LO2} = 714^{\#}$ EQ
$= 541^{\#}$ EQ
$= 648^{\#}$ WIND

$W_2 = 9.5(25+40)$
$= 2375_D + 3230_L$

WALL BTWN LO2 & ROOF

$V = 5798^{\#}$ EQ
$L = 5'-6"$
$\} v = 1028$ PLF
1054

5/8" SHT'G B.S. w/ 10d @ 2" O.C.
$V_{ALLOW} = 1140$ ok

HEAD CONN'S

$N_{A34} = \frac{5798}{(645+670)} = 4.9$ / 4.8

$N_{\frac{1}{2}"\phi} = \frac{5798}{1.6(650)} = 5.58$ / 5.44

$N_{LAG} = \frac{5798}{1.6(640)} = 5.66$ / 5.52

USE MIN (5) SIMPSON A34 + CAP4

USE MIN (6) 1/2"ϕ BOLTS TO W10 BM

USE MIN (6) 5/8"ϕ LAGS TO BCK'G

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SILL CONN

$N_{5/8"\phi} = \frac{5798}{16(1000)} = 3.6 \approx 3.51$

USE (4) 5/8" ϕ THRU-BOLTS

OTM @ LOZ = $5.798(8.42) = 48.82^{in}$

$F_T = \frac{48.82^{in}}{5.21'} = \pm 9.14^{k}$ → 9.37

TIE AT T1

$P = 204\# + \frac{17868}{1554Z} \leq \pm 9140$ → 9370

$P_{MAX} = 20.63^{k}$ — SEE NEXT SHT FOR POST DES.

$P_{MIN} = .6\left(204 + \frac{5.5}{2}(270) + \frac{5.5}{2}(8.5)(10)\right) - 9140$ → 9370

$= -8432\#$ MAX T.

-8662 — STEEL POST OR BG INSP

TIE AT T2

$P = 3963\# + 33660\# \leq \pm 9140$ → 9370

$P_{MAX} = 31623\#$ — SEE NEXT SHTS FOR POST DESIGN

$P_{MIN} = .6(3963 + 143 + 234) - 9140$ → 9370

$= 6176\#$ MAX T,

-6406

STL POST OR BG INSP.

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WALL BRACE L01 & L02

V = 6512 / 6194# EQ
L = 12'-10"
} V = 483 PLF EQ. (508)

5/8" SHTG B.S. W/ 10d @ 2" O.C.
V_{ALLOW} = 1740, EQ

STRAP @ POST @ TOP OF PT:

$T_{MAX} = \frac{6512}{6194}\left(\frac{1.5\,B}{12.83}\right) = 3847^{\#}$

SIMPSON MST27 EA. SIDE OF PT

$T_{ALLOW} = 2(3700) = 7400^{\#}$

STRAP @ WALL END:

$T_{MAX} = \frac{714}{541}\left(\frac{7.0}{19.92}\right)\left(\frac{4}{1.3}\right) = 772^{\#}$

SIMPSON CS22 STRAP W/ (12) 10d @ END

$T_{ALLOW} = 845^{\#}$

SILL CONNECT:

$S = \frac{1.6(1540)}{483}(12) = 61"\ O.C.$ (508) (58")

USE 3/4" ϕ J-BOLTS @ 32" OC MAX

(3 MIN REQ)

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OHM @ GHB $= 6.194(9.42) = 64.60^{lk}$ (6.512)

$T_T = \pm \frac{64.60}{12.25'} = \pm 5016^{\#}$ (64.60, 5273)

TIE AT T1

$P = 204 + \frac{17868}{15542}S \pm (5016 + 9140)$ (5273 + 9370)

14156 14643

$P_{MAX} = 24722^{\#}$ (24587) — HSSCXT2x1/4 OK BY INSP

$P_{MIN} = 0.6\left(204 + 210(5.5) + 5.5(8.5)(10) + \frac{12.83}{2}(238+100)\right)$ (14643)

$- 14156$

$= 11561^{\#}$ MAX T (-12048)

STL POST OK BY INSP

TIE AT T3

$P = \pm 5016^{\#}$ (5273)

$P_{MAX} = 5016^{\#}$ (5273) 3x8 STUD OK BY INSP.

$P_{MIN} = .6\left(\frac{12.83}{2}(238+100)\right) - 5016 = 3716^{\#}$ (5273, -3972)

SIMPSON HDU4-SDS25 TO 4x8 POST

$T_{ALLOW} = 4565^{\#}$ ok

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48.82 64.60 6.512 139.47

Otm @ B.O. FDC = 41.60 + 61.44 + 6.194(4) = 133.82 ^k

h_{WALL} = 12'-10", t @ 0'-5"

P_{DL} = 204 + 3693 + 5.5(220 + 85) + 12.83(238 + 100) = 10186 #
(1952) (4334)

W_{DL} = 794 PLF

M_{DL} = 204(6.42) + 3693(.92) + 1952(3.61) = 11187 ^k

W_{LL} = 380 PCF

M_{LL} = 0

P_S = 17868 + 33660 = 51528 #

W_{SL} = 4016 PCF

M_{SL} = 17.868(6.42) + 33.66(.92) = 145.68 ^k

- SEE NEXT SHT FOR FDC -

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project name: CCE

project no. 15105

designed by: MAR date: Jun-15
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SHEAR WALL FOOTING DESIGN

INPUT DATA : Unit 62R Wall SW2

Allow. Soil Pr. =	2.400	ksf	DL OTM =	0.00	f t - kips	
Fy =	60	ksi	FLR LL OTM =	0	f t - kips	
f 'c =	3000	psi	RF LL OTM =	0	f t - kips	
Wall DL =	0.79	klf	SEISMIC OTM =	133.82	~~f t - kips~~	139.47 See next sheet
Roof LL =	4.02	klf	Footing Length :	17.17	feet	
Floor LL =	0.38	klf	Footing Width :	3.00	feet	
Wall Length =	12.83	feet	Footing Thkness:	12	inches	
Wall Thickness =	8	inches	Footing DL :	1.590	klf	

OUTPUT DATA :

EQ'N 16-11 : DL + .75(FL + RL):

P =	79.8 kips	P_{ult} =	112.7 kips		
OTM =	0.00 ft-kips	OTM_{ult} =	0.00 ft-kips		
e =	0.00 feet	X bar =	N/A feet		
Soil Pr. =	1.55 ksf, max.,	2.19 ksf, ult.		Required Width =	1.94 feet

EQ'N 16-14: DL + .75(FL + RL + .7E)

P =	79.8 kips	P_{ULT} =	112.7 kips		
OTM =	100.4 ft-kips	OTM_{ULT} =	160.58 ft-kips		
e =	1.26 feet	X bar =	N/A feet		
Soil Pr. =	2.23 ksf, max.,	3.15 ksf, ult.		Required Width =	2.79 feet

EQ'N 16-16: 0.6DL + 0.7E

P =	22.5 kips	P (ult) =	27.0 kips		
OTM =	133.8 ft-kips	OTM_{ULT} =	149.88 ft-kips		
e =	5.95 feet	X bar =	2.64 feet		
Soil Pr. =	1.90 ksf, max.,	2.28 ksf, ult.		Required Width =	2.37 feet

Resisting Moment = 321.83 ft-kips Factor of Safety = 3.44

FOOTING REINFORCING:

LONGITUDINAL DIRECTION:

Req'd Unreinf Thickness =	32	inches			
Moment =	9.60 ft-kips/ft	Fb(allow)=	178 psi	fb(act.)=	64 psi
Shear =	0.00 kips/ft	Fv(allow)=	71 psi	fv(act.)=	0 psi

TRANSVERSE DIRECTION:

Req'd Unreinf Thickness =	20	inches			
Moment =	3.25 ft-kips/ft	Fb(allow)=	178 psi	fb(act.)=	60 psi
Shear =	0.00 kips/ft	Fv(allow)=	71 psi	fv(act.)=	0 psi

Reinf. Thickness (if used) =	12	inches		
Longitudinal Steel Required =	0.26	sq.in./ft.	v(longit.) =	44 psi
Transverse Steel Required =	0.09	sq.in./ft.	v(transv.)=	13 psi
			V(allow) =	93.1 psi

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project no. 15105

designed by: MAR date: Jun-15

checked by: date:

SHEAR WALL FOOTING DESIGN

INPUT DATA :

Unit 62R Wall SW2

Allow. Soil Pr. =	2.400	ksf		DL OTM =	0.00	f t - kips
Fy =	60	ksi		FLR LL OTM =	0	f t - kips
f 'c =	3000	psi		RF LL OTM =	0	f t - kips
Wall DL =	0.79	klf		SEISMIC OTM =	139.47	f t - kips
Roof LL =	4.02	klf		Footing Length :	17.17	feet
Floor LL =	0.38	klf		Footing Width :	3.00	feet
Wall Length =	12.83	feet		Footing Thkness:	12	inches
Wall Thickness =	8	inches		Footing DL :	1.590	klf

OUTPUT DATA :

EQ'N 16-11 : DL + .75(FL + RL):

P =	79.8	kips	P_{ult} =	112.7	kips		
OTM =	0.00	ft-kips	OTM_{ult} =	0.00	ft-kips		
e =	0.00	feet	X bar =	N/A	feet		
Soil Pr. =	1.55	ksf, max.,	2.19	ksf, ult.		Required Width =	1.94 feet

EQ'N 16-14: DL + .75(FL + RL + .7E)

P =	79.8	kips	P_{ULT} =	112.7	kips		
OTM =	104.6	ft-kips	OTM_{ULT} =	167.36	ft-kips		
e =	1.31	feet	X bar =	N/A	feet		
Soil Pr. =	2.26	ksf, max.,	3.19	ksf, ult.		Required Width =	2.82 feet

EQ'N 16-16: 0.6DL + 0.7E

P =	22.5	kips	P (ult) =	27.0	kips		
OTM =	139.5	ft-kips	OTM_{ULT} =	156.21	ft-kips		
e =	6.20	feet	X bar =	2.38	feet		
Soil Pr. =	2.10	ksf, max.,	2.52	ksf, ult.		Required Width =	2.62 feet

Resisting Moment = 321.83 ft-kips Factor of Safety = 3.30

FOOTING REINFORCING:

LONGITUDINAL DIRECTION:

Req'd Unreinf Thickness =	32	inches					
Moment =	9.72	ft-kips/ft	Fb(allow)=	178	psi	fb(act.)=	65 psi
Shear =	0.00	kips/ft	Fv(allow)=	71	psi	fv(act.)=	0 psi

TRANSVERSE DIRECTION:

Req'd Unreinf Thickness =	20	inches					
Moment =	3.29	ft-kips/ft	Fb(allow)=	178	psi	fb(act.)=	61 psi
Shear =	0.00	kips/ft	Fv(allow)=	71	psi	fv(act.)=	0 psi

Reinf. Thickness (if used) =	12	inches			
Longitudinal Steel Required =	0.26	sq.in./ft.	v(longit.) =	45	psi
Transverse Steel Required =	0.09	sq.in./ft.	v(transv.)=	14	psi
			V(allow) =	93.1	psi

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WALL SW3

989

V_{LO2} = 761# EQ., 2438# WIND (1689 @ +9-11, 749 @ +6-9)

V_{LO1} = 366# EQ., 1127# WIND (@ -4'-8")

-1919 → 1127#

-931 → 3165#

WIND CONTROLS previous design OK

L_{WALL} = 7'-4"

FROM LO1 TO LO2

V = 332 PLF WIND

½"SHTG w/ 8d @ 6" O.C.

V_{ALLOW} = 365 PLF WIND

OTM @ +6'-9" FLR = 1.089(3.17') = 5.35^{1k}

WALL DL = 6.17(10) = 62 PLF

$F_T = \frac{1}{2}(6.75)(62)(0.6) - \frac{5350}{7'} = 634^{\#}$ TENS

SIMPSON MST37

$T_{ALLOW} = 2405^{\#}$

OTM @ -4'-8" = 5.35 + 2.438(11.42) = 33.19^{1k}

WALL DL = 8.33(24) + 11.75(10) = 318 PLF

$F_T = \frac{8.67(318)(.6) - 33190}{7} = 2833^{\#}$ TENS

SIMPSON MST48

$T_{ALLOW} = 3960^{\#}$

FROM GND TO LO1

V = 432 PLF

½"SHTG w/ 8d @ 4" O.C.

V_{ALLOW} = 533 PLF OK

SILL CONN'S

$s = \frac{1.6(141)}{432}(12) = 6.3"$

USE 16d @ 6" O.C.

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WALL SW4

1733

$V_{LO2} = 522^{\#}$ EQ, $1685^{\#}$ WIND

$V_{LO1} = 97^{\#}$ EQ, $221^{\#}$ WIND

1252

2985

$619^{\#}$ $1906^{\#}$ WIND CONTROLS

$L_{WALL} = 14\frac{1}{2}'2''$

2985

$v = \frac{1685}{14.17} = 119$ PLF

211

$\frac{1}{2}$"SHTG w/ 8d@6"

$v_{ALLOW} = 365$

1.733 17.19

OTM @ LO1 $= 1.685(9.92) = 16.72^{k}$

$WALL\ DL = 12(.25)(24) + 9.92(10) = 399$ PLF

17.19

$F_T = .6\left(\frac{14.17}{2}(.399)\right) - \frac{16.72}{13.67} = 0$ NO UPLIFT

OTM @ B.O.FTG = 1.733(9.92) + 2.985(16.83) = 67.43 ft-kips
Wall DL = 12(25) + 14.92(10) + 15.83(150) = 2824 plf
Floor LL = 12(40) = 480 plf
See next sheet for footing design.

Wall SW-5:
V_{EQ} = 4.18 kips
OTM @ B.O.FTG = 4.18(15) = 62.7 ft-kips
Wall DL = 14(100) + 9(25 + 28) = 1877 plf
Floor LL = 9(40) = 360 plf
Snow LL = 9(264) = 2376 plf
See second sheet following for footing design.

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SHEAR WALL FOOTING DESIGN

INPUT DATA :

Unit 62R Wall SW4

Allow. Soil Pr. =	2.400	ksf	DL OTM =	0.00	f t - kips
Fy =	60	ksi	FLR LL OTM =	0	f t - kips
f 'c =	3000	psi	RF LL OTM =	0	f t - kips
Wall DL =	2.82	klf	SEISMIC OTM =	67.43	f t - kips
Roof LL =	0.00	klf	Footing Length :	23.00	feet
Floor LL =	0.48	klf	Footing Width :	2.00	feet
Wall Length =	23.00	feet	Footing Thkness:	12	inches
Wall Thickness =	12	inches	Footing DL :	0.615	klf

OUTPUT DATA :

EQ'N 16-11 : DL + .75(FL + RL):

P =	87.4	kips	P_{ult} =	108.2	kips		
OTM =	0.00	ft-kips	OTM_{ult} =	0.00	ft-kips		
e =	0.00	feet	X bar =	N/A	feet		
Soil Pr. =	1.90	ksf, max.,	2.35	ksf, ult.		Required Width =	1.58 feet

EQ'N 16-14: DL + .75(FL + RL + .7E)

P =	87.4	kips	P_{ULT} =	108.2	kips		
OTM =	50.6	ft-kips	OTM_{ULT} =	80.916	ft-kips		
e =	0.58	feet	X bar =	N/A	feet		
Soil Pr. =	2.19	ksf, max.,	2.71	ksf, ult.		Required Width =	1.82 feet

EQ'N 16-16: 0.6DL + 0.7E

P =	47.5	kips	P (ult) =	57.0	kips		
OTM =	67.4	ft-kips	OTM_{ULT} =	75.522	ft-kips		
e =	1.42	feet	X bar =	N/A	feet		
Soil Pr. =	1.41	ksf, max.,	1.70	ksf, ult.		Required Width =	1.18 feet

Resisting Moment = 909.72 ft-kips Factor of Safety = 19.27

FOOTING REINFORCING:

LONGITUDINAL DIRECTION:

Req'd Unreinf Thickness =	12	inches						
Moment =	0.15	ft-kips/ft	Fb(allow)=	178	psi	fb(act.)=	9	psi
Shear =	0.00	kips/ft	Fv(allow)=	71	psi	fv(act.)=	0	psi

TRANSVERSE DIRECTION:

Req'd Unreinf Thickness =	12	inches						
Moment =	0.93	ft-kips/ft	Fb(allow)=	178	psi	fb(act.)=	56	psi
Shear =	0.00	kips/ft	Fv(allow)=	71	psi	fv(act.)=	0	psi

Reinf. Thickness (if used) =	12	inches			
Longitudinal Steel Required =	0.00	sq.in./ft.	v(longit.) =	0	psi
Transverse Steel Required =	0.02	sq.in./ft.	v(transv.)=	0	psi
			V(allow) =	93.1	psi

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project no. 15105

designed by: MAR date: 3/6/2017

checked by: date:

SHEAR WALL FOOTING DESIGN

INPUT DATA :

Unit 62R Wall SW5

Allow. Soil Pr. =	2.400	ksf	DL OTM =	0.00	f t - kips
Fy =	60	ksi	FLR LL OTM =	0	f t - kips
f 'c =	3000	psi	RF LL OTM =	0	f t - kips
Wall DL =	1.88	klf	SEISMIC OTM =	62.7	f t - kips
Roof LL =	2.38	klf	Footing Length :	15.00	feet
Floor LL =	0.36	klf	Footing Width :	2.00	feet
Wall Length =	11.50	feet	Footing Thkness:	12	inches
Wall Thickness =	8	inches	Footing DL :	0.604	klf

OUTPUT DATA :

EQ'N 16-11 : DL + .75(FL + RL):

P =	54.2	kips	P_{ult} =	74.5	kips		
OTM =	0.00	ft-kips	OTM_{ult} =	0.00	ft-kips		
e =	0.00	feet	X bar =	N/A	feet		
Soil Pr. =	1.81	ksf, max.,	2.48	ksf, ult.		Required Width =	1.51 feet

EQ'N 16-14: DL + .75(FL + RL + .7E)

P =	54.2	kips	P_{ULT} =	74.5	kips		
OTM =	47.0	ft-kips	OTM_{ULT} =	75.24	ft-kips		
e =	0.87	feet	X bar =	N/A	feet		
Soil Pr. =	2.44	ksf, max.,	3.35	ksf, ult.		Required Width =	2.03 feet

EQ'N 16-16: 0.6DL + 0.7E

P =	18.4	kips	P (ult) =	22.1	kips		
OTM =	62.7	ft-kips	OTM_{ULT} =	70.224	ft-kips		
e =	3.41	feet	X bar =	4.09	feet		
Soil Pr. =	1.50	ksf, max.,	1.80	ksf, ult.		Required Width =	1.25 feet

Resisting Moment = 229.83 ft-kips Factor of Safety = 5.24

FOOTING REINFORCING:

LONGITUDINAL DIRECTION:

Req'd Unreinf Thickness =	18	inches						
Moment =	7.09	ft-kips/ft	Fb(allow)=	178	psi	fb(act.)=	166	psi
Shear =	0.83	kips/ft	Fv(allow)=	71	psi	fv(act.)=	6	psi

TRANSVERSE DIRECTION:

Req'd Unreinf Thickness =	12	inches						
Moment =	1.55	ft-kips/ft	Fb(allow)=	178	psi	fb(act.)=	93	psi
Shear =	0.00	kips/ft	Fv(allow)=	71	psi	fv(act.)=	0	psi

Reinf. Thickness (if used) =	12	inches			
Longitudinal Steel Required =	0.19	sq.in./ft.	v(longit.) =	34	psi
Transverse Steel Required =	0.04	sq.in./ft.	v(transv.)=	0	psi
			V(allow) =	93.1	psi

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job number: 15105

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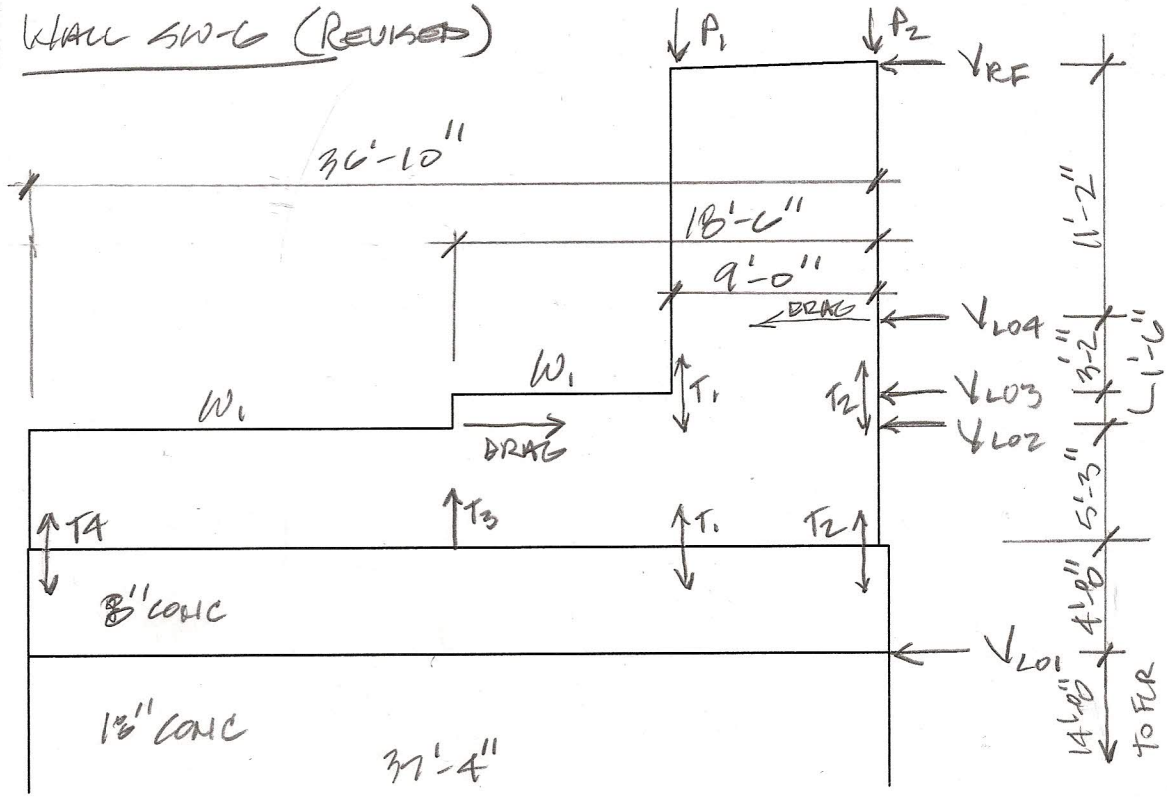
checked by:

date: 3/2017
date:

WALL SW-6 (REVISED)



- SEISMIC CONTROLS WALL DESIGN

$V_{RF} = 1947\#$
$V_4 = 10859\#$ GR, $2913\#$ NET
$V_3 = 13229\#$ GR, $2379\#$ NET
$V_2 = 15665\#$ GR, $2427\#$ NET
$V_1 = 18821\#$ GR, $3156\#$ NET

$P_1 = 768\# + \frac{1170}{7384} \leq$

$P_2 = 1575\# + \frac{1085}{11149} \leq$

$W_1 = 2(25+40) = 50\# + 80\#$

WALL BTWN LO3 & RF:

$v = \frac{13229}{9} = 1471$ PLF $\Rightarrow$ 5/8" STRUC B.S w/ 10d@2"
$v_{ALLOW} = 1740$ ok

ROOF CONN'S:

$N_{LPT4} = \frac{1947}{670} = 11.9$ USE (14) LPT4 CLIPS

$N_{RBC\ \&\ A34} = \frac{1947}{(515+435)} = 8.3$ USE (8) A34 + RBC CLIPS
$\Delta = 4.5\%$ ok

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SILL CONN'S: $N_{3/4"\phi} = \frac{10859}{1.6(960)} = 7.06$ USE (7) 3/4"ϕ LAGS

LOA DRAG TO WALL: $V = 2913^{\#}\left(\frac{4}{1.3}\right) = 8963^{\#}$

USE SIMPSON CMST12 STRAP

$T_{ALLOW} = 9215^{\#}$

$N = \frac{8963}{9215.}(86) = 84$

USE (84) 10d x 1½" NAILS

OTM @ LOS = $7.947(11.17) + 10.859(3.17) = 123.19^{lk}$

$F_T = \pm \frac{123.19}{8.71'} = \pm 14.14^{k}$

TIE AT T1:

$P = 868 + \frac{1770}{1384}S \pm 14140$ EQ

$P_{MAX} = 17.30^{k}$ - SEE NEXT SHT FOR POST DESIGN

$P_{MIN} = .6\left(868 + 4.5(14.83)(15)\right) - 14140$

$= -13038^{\#}$

SIMPSON MST60 EA. SIDE

$T_{ALLOW} = 2(6730) = 13460^{\#}$ ok

TIE AT T2:

$P = 1575 + \frac{10859}{11149}S \pm 14140$

$P_{MAX} = 20541^{\#}$ - SEE NEXT SHT FOR POST DESIGN

$P_{MIN} = .6\left(1575 + 4.5(14.83)(15)\right) - 14140$

$= -12615^{\#}$

SAME AS T1

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Project Title: Copper Crest East
Engineer: MAR
Project Descr:

Project ID: 15105

Printed: 7 MAR 2017, 10:35AM

Wood Column

File = C:_jobs\15105C~1\ENG\CCE-20~1.EC6
ENERCALC, INC. 1983-2017, Build:6.17.2.28, Ver:6.17.2.28

Lic. # : KW-06002357 **Licensee : RUDOW & BERRY**

Description : 62R - SW6 - T2 Post from L02 to Roof - 4x8 D.Fir#1

Code References

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combinations Used : ASCE 7-10 w/ ASD Wind & EQ

General Information

Analysis Method : Allowable Stress Design
End Fixities : Top & Bottom Pinned
Overall Column Height : 14.250 ft
(Used for non-slender calculations)

Wood Species : Douglas Fir - Larch
Wood Grade : No.1

Fb - Tension	1,000.0 psi	Fv	180.0 psi
Fb - Compr	1,000.0 psi	Ft	675.0 psi
Fc - Prll	1,500.0 psi	Density	31.20 pcf
Fc - Perp	625.0 psi		

E : Modulus of Elasticity . . .	x-x Bending	y-y Bending	Axial
Basic	1,700.0	1,700.0	1,700.0 ksi
Minimum	620.0	620.0	

Wood Section Name : **4x8**
Wood Grading/Manuf. : Graded Lumber
Wood Member Type : Sawn

Exact Width	**3.50**	in
Exact Depth	**7.250**	in
Area	25.375	in^2
Ix	111.148	in^4
Iy	25.904	in^4

Allow Stress Modification Factors

Cf or Cv for Bending	1.30
Cf or Cv for Compression	1.050
Cf or Cv for Tension	1.20
Cm : Wet Use Factor	1.0
Ct : Temperature Factor	1.0
Cfu : Flat Use Factor	1.0
Kf : Built-up columns	1.0 *NDS 15.3.2*
Use Cr : Repetitive ?	No

Brace condition for deflection (buckling) along columns :
X-X (width) axis : Fully braced against buckling along X-X Axis
Y-Y (depth) axis : Unbraced Length for X-X Axis buckling = 14.250 ft, K = 1.0

Applied Loads

Service loads entered. Load Factors will be applied for calculations.

AXIAL LOADS . . .
Axial Load at 14.250 ft, D = 1.575, S = 11.149, E = 14.140 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio = **0.9710** : 1
Load Combination : +D+0.750L+0.750S+0.750E+H
Governing NDS Forumla : Comp Only, fc/Fc'
Location of max.above base : 0.0 ft
At maximum location values are . . .
Applied Axial : 20.542 k
Applied Mx : 0.0 k-ft
Applied My : 0.0 k-ft
Fc : Allowable : 833.68 psi

PASS Maximum Shear Stress Ratio = **0.0** : 1
Load Combination : +0.60D+E+0.60H
Location of max.above base : 14.250 ft
Applied Design Shear : 0.0 psi
Allowable Shear : 288.0 psi

Maximum SERVICE Lateral Load Reactions . .

Top along Y-Y	0.0 k	Bottom along Y-Y	0.0 k
Top along X-X	0.0 k	Bottom along X-X	0.0 k

Maximum SERVICE Load Lateral Deflections . . .
Along Y-Y : 0.0 in at 0.0 ft above base
for load combination : n/a
Along X-X : 0.0 in at 0.0 ft above base
for load combination : n/a

Other Factors used to calculate allowable stresses . . .

	Bending	Compression	Tension

Load Combination Results

Load Combination	C_D	C_P	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Maximum Shear Ratios: Stress Ratio	Status	Location
+D+H	0.900	0.528	0.08292	PASS	0.0 ft	0.0	PASS	14.250 ft
+D+L+H	1.000	0.488	0.08069	PASS	0.0 ft	0.0	PASS	14.250 ft
+D+Lr+H	1.250	0.409	0.07712	PASS	0.0 ft	0.0	PASS	14.250 ft
+D+S+H	1.150	0.438	0.6326	PASS	0.0 ft	0.0	PASS	14.250 ft
+D+0.750Lr+0.750L+H	1.250	0.409	0.07712	PASS	0.0 ft	0.0	PASS	14.250 ft
+D+0.750L+0.750S+H	1.150	0.438	0.4940	PASS	0.0 ft	0.0	PASS	14.250 ft
+D+W+H	1.600	0.331	0.07445	PASS	0.0 ft	0.0	PASS	14.250 ft
+D+E+H	1.600	0.331	0.7429	PASS	0.0 ft	0.0	PASS	14.250 ft
+D+0.750Lr+0.750L+0.750W+H	1.600	0.331	0.07445	PASS	0.0 ft	0.0	PASS	14.250 ft
+D+0.750L+0.750S+0.750W+H	1.600	0.331	0.4697	PASS	0.0 ft	0.0	PASS	14.250 ft
+D+0.750L+0.750S+0.750E+H	1.600	0.331	0.9710	PASS	0.0 ft	0.0	PASS	14.250 ft
+0.60D+W+0.60H	1.600	0.331	0.04467	PASS	0.0 ft	0.0	PASS	14.250 ft
+0.60D+E+0.60H	1.600	0.331	0.7131	PASS	0.0 ft	0.0	PASS	14.250 ft

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Project ID: 15105

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Wood Column

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ENERCALC, INC. 1983-2017, Build:6.17.2.28, Ver:6.17.2.28

Lic. # : KW-06002357 **Licensee : RUDOW & BERRY**

Description : 62R - SW6 - T2 Post from L02 to Roof - 4x8 D.Fir#1

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction @ Base	X-X Axis Reaction @ Top	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Axial Reaction @ Base
+D+H		k		k	1.575 k
+D+L+H		k		k	1.575 k
+D+Lr+H		k		k	1.575 k
+D+S+H		k		k	12.724 k
+D+0.750Lr+0.750L+H		k		k	1.575 k
+D+0.750L+0.750S+H		k		k	9.937 k
+D+W+H		k		k	1.575 k
+D+E+H		k		k	15.715 k
+D+0.750Lr+0.750L+0.750W+H		k		k	1.575 k
+D+0.750L+0.750S+0.750W+H		k		k	9.937 k
+D+0.750L+0.750S+0.750E+H		k		k	20.542 k
+0.60D+W+0.60H		k		k	0.945 k
+0.60D+E+0.60H		k		k	15.085 k
D Only		k		k	1.575 k
Lr Only		k		k	k
L Only		k		k	k
S Only		k		k	11.149 k
W Only		k		k	k
E Only		k		k	14.140 k
H Only		k		k	k

Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance
+D+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+L+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+Lr+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+S+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750Lr+0.750L+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750L+0.750S+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+W+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+E+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750Lr+0.750L+0.750W+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750L+0.750S+0.750W+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+D+0.750L+0.750S+0.750E+H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+0.60D+W+0.60H	0.0000 in	0.000 ft	0.000 in	0.000 ft
+0.60D+E+0.60H	0.0000 in	0.000 ft	0.000 in	0.000 ft
D Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
Lr Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
L Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
S Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
W Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
E Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
H Only	0.0000 in	0.000 ft	0.000 in	0.000 ft

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Wood Column

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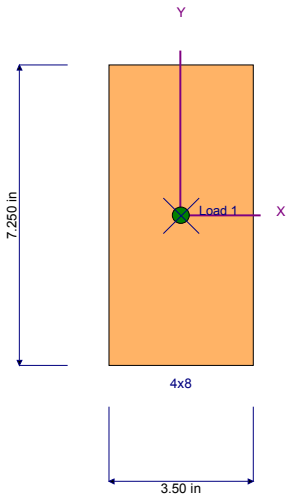
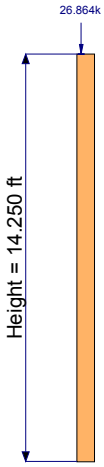
ENERCALC, INC. 1983-2017, Build:6.17.2.28, Ver:6.17.2.28

Lic. # : KW-06002357 **Licensee : RUDOW & BERRY**

Description : 62R - SW6 - T2 Post from L02 to Roof - 4x8 D.Fir#1

Sketches





Loads are total entered value. Arrows do not reflect absolute direction.

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WALL BTWN L02 & L03

$V = 13239^{\#}$, $L = 18'-6$ $\Big\}$ $v = 716$ PLF

5/16" STRUC B.S. w/ 10d @ 6"
$v_{ALLOW} = 680$ PLF
$\Delta = 5\%$ ok

SILL CHECK:

v @ 9' WIDTH $= \frac{10859}{9} + \frac{2379}{18.5} = 1335$ PLF

V @ 9' WIDTH $= 10859 + 2379\left(\frac{9}{18.5}\right) = 12016^{\#}$

$N_{3/4"\phi} = \frac{12016}{1.1(900)} = 7.82$ USE MIN (8) 3/4"ϕ LAGS

DRAG TO REM. OF WALL:

$V_{DRAG} = 10859\left(\frac{9.5}{18.5}\right) = 5576^{\#}$

USE SIMPSON MST37 EA SIDE OF WALL

TIE AT T2:

$F_T = -12016 + \left[\frac{13239(1.5)}{18}\right] = -13218^{\#}$

TIE AT T3:

$F_T = \frac{13239(1.5)}{18} = 1103^{\#} \Rightarrow$ STEEL COL CONN OK BY INSP.

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WALL BELOW LOZ & T.O. 8" CONC WALL

$V = 15665^{\#}$
$L = 36'-10"$
$\} \; v = 425 \text{ PLF}$

5/8" B.S. w/ 10d @ 6"
$v_{allow} = 680$ ok

SILL CONN'T:

$s_{REQ'D} = \frac{1.6(710)}{425}(12) = 34.8"$

USE ½"ϕ AB's @ 16" OC MAX

DRAG CONN'T

$V_{DRAG} = 2471\left(\frac{165}{36.83}\right)\left(\frac{4}{1.3}\right) = 3751^{\#}$

SIMPSON CMSTC STRAP

$T_{allow} = 4585^{\#}$

$N = \frac{3751}{4585}(50) = 41$

USE (42) 10d x 1½" NAILS

OTM @ T.O. CONC WALL $= 15.665(5.25) = 82.24^{k\text{-}ft}$

$F_T = \frac{82.24}{36} = \pm 2.28^{k}$

TIE AT T4:

$F_T = 0.6\left(18.42(680 + 9.25(15))\right) - 2280 = 194^{\#} \geq 0$

NO UPLIFT DUE TO WALL AROUND CORNER

TIE AT T2:

$F_T = -13718 + \left[.6(50 + 9.25(15)) - 2280\right] = -13912^{\#}$

SIMPSON HDU14-SDS2.5

$T_{allow} = 14445^{\#}$ ok
(TO 5½" POST)

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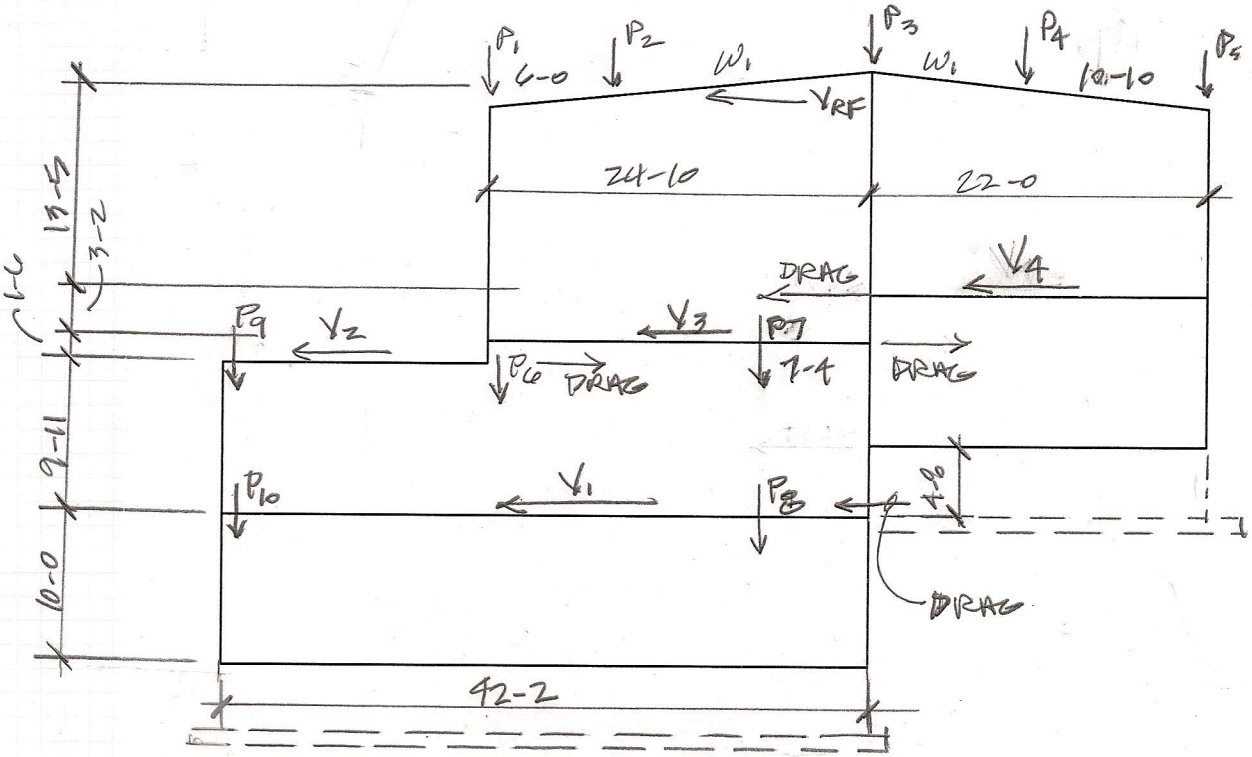
WALL SW1 @ CCR



V_{RF} = 6535# GR

V_4 = 5954# GR, -581# NET

V_3 = 4887# GR, -1067# NET

V_2 = 6264# GR, 1377# NET

V_1 = 5248# GR, -1015# NET

$P_1 = R_{RB8L} = 1205D\,L + \frac{6448}{9661}S$

$P_2 = R_{RB5L} = 511D\,L + \frac{4559}{39123}S$

$P_3 = R_{RB4L} = 2280D + \frac{14619}{10231}S$

$P_4 = R_{RB9L} = 4110D\,L + \frac{-2639}{+2511}S$

$P_5 = R_{RB12L} = 1693D + \frac{9319}{14453}S$

$P_6 = R_{FB3L} = 1451D + 912L + 783S$

$P_7 = R_{FB7L} = 1038D + 1383L$

$P_8 = R_{FB15L} = 1513D + 2345L$

$P_9 = R_{THSL} = 2063D + 1386L S$

$P_{10} = R_{THSL} = 515D + 710L - 2373S$

$W_1 = 9(24 + 203) = 216D + 1827S$, PLF

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WALL BEHIND ROOF @ LOZ

V = 6535 MAX, 4887 MIN
L_{MIN} = 24-10

V = 263 PLF — ½" SHEATHING w 8d @ 6"
V_{ALLOW} = 260 ok

OTM @ LOZ = 6.535(13.42) + 5.954(3.16) = 106.51 ft-lb

$F_T = \frac{106.51}{46'} = \pm 2316^{\#}$

TIE @ EAST: $F_T = 0.6(1683 + 0.2(9319) + 16.58(23)(16)) - 2316$
= 0 NO UPLIFT

TIE @ WEST: $F_T = 0.6(1205 + 0.2(6948) + 16.58(23)(16)) - 2316$
= 0 NO UPLIFT

DRAG @ LOA: $F_{DRAG} = 581\left(\frac{24.83}{46.83}\right)\left(\frac{4}{1.3}\right) = 948^{\#}$

SIMPSON CMSTC16 STRAP

$N = \frac{948}{458.5}(50) = 11$

USE (20) 10d x 1½ NAILS

$l_{DRAG} = \frac{948}{260} = 3.64$ FT MIN

DRAG @ LOZ: $F_{DRAG} = 1061\left(\frac{22}{46.83}\right)\left(\frac{4}{1.3}\right) = 1542^{\#}$

SIMPSON CMSTC16 STRAP

$N = \frac{1542}{458.5}(50) = 16.8$

USE (20) 10d x 1½ NAILS

$l_{DRAG} = \frac{1542}{260} = 5.93'$ MIN

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DRAG @ LO2: $F_{DRAG} = 1377\left(\frac{24.83}{42.16}\right)\left(\frac{4}{1.3}\right) = 2495^{\#}$

SIMPSON CMSTC16 STRAP

$N = \frac{2495}{4585}(50) = 27.2$

USE (28) 10d x 1½ NAILS

$L_{DRAG} = \frac{2495}{260} = 9.6$ FT MIN

DRAG @ LO1 $F_{DRAG} = 4887\left(\frac{22}{46.83}\right)\left(\frac{4}{1.3}\right) = 1064^{\#}$

(2) SIMPSON HDU4-SDS25

$T_{ALLOW} = 2(4565) = 9130^{\#}$ ok

OTM @ T.O. STEM =

$= 6535\left(\frac{24.83}{46.83}\right)(38') - 581\left(\frac{24.83}{46.83}\right)(24.58) - 1067\left(\frac{24.83}{46.83}\right)(21.42)$

$+ 1377(19.92) + 4887\left(\frac{22}{46.83}\right)(14.67) - 1015(10) = 162.94^{1k}$

PREVIOUS OTM FOR SEGMENT (B) = $175.36^{1k} > 162.94^{1k}$

PREVIOUS FOOTING DESIGN ok

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job name: Copper Crest East
job number: 15105

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designed by: MAR date: 1/17
checked by: date:

HOLDDOWN BOLT EMBED. REQUIREMENTS

UNIT	WALL	TIE #	T (ASD)	T (ULT)	INTO
TYP.	SW1	T1	29464 28544#	47973 46501#	FTG. (18")
	SW1	T2	14642 14052#	24110 23106#	FTG. (18")
	SW1	T3	13697 13367#	22513 21984#	FTG. (18")
	SW2	T1	24487 23508#	39505 37938#	8" WALL
	SW2	T2	23356 22316#	38148 36579#	8" WALL
62R	SW1	T1	16981 16650#	26105 25695#	REBAR TO FTG
		T2	8403 7963#	15792 15088#	REBAR TO FTG
		T3	14775#	25634#	FTG (18")
		T4	11191#	21333#	FTG (18")
	SW2	T1	12048 11561#	20315 19556#	REBAR TO STEEL
		T2	6406 6176#	11435 11067#	REBAR TO STEEL
		T3	3972 3715#	6876 6464#	8" WALL
	SW6	T1	13038 12262#	21278 15078#	8" WALL
		T2	13912 9858#	26070 16597#	8" WALL

T3	1103#	1765#	STEEL POST
T4	0#	0#	N/A

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designed by: MAR date: 1/17
checked by: date:

TYPE 1: BOLT EMBED IN 12" THK FTG

$P_u(max) = 46.50^k$ 47.973

MAX EMBED = 15"

$A_{Nco} = 9(15)^2 = 2025\ in^2$

$A_{Nc} = A_{Nco}$

$N_b = 24(3000)^{\frac{1}{2}}(15)^{1.5} = 76.37^k$

$N_{cb} = N_b = 76.37^k$

$\phi N_{cb} = 0.75(76.37) = 57.28^k > 46.50^k$ 47.973 ok

USE 15" MIN EMBED

TYPE 2: BOLT EMBED IN 8" WALL (12" WALL SIMILAR)

$P_u(max) = 37938\#$ 39505

$A_s(req'd) = \frac{37.938}{0.9(60)} = 0.70\ in^2$ 39.505 0.73

USE (4) #5 VERTS

TENS. DEV. LENGTH = 20.7" } ok
HOOK DEV. LENGTH = 8"

TYPE 3: STEEL POST TO EMBED

$P_u(max) = 20595\#$
$A_s(req'd) = 0.49\ in^2$

USE (4) #6 TO EMBED

TENS. DEV. LENGTH = 12" MIN
HOOK DEV. LENGTH = 6" MIN