MILAN STREET AND ATHENS AVENUE

APN 179-04-503-001

MILAN LOT 2

HENDERSON, NEVADA

LRP PROJECT NO: 20045

STRUCTURAL DESIGN CALCULATIONS

PREPARED FOR ASSURED REAL ESTATE

BY

L.R. POPE ENGINEERING, INC.
1240 E 100 S #15B
ST. GEORGE, UTAH 84790
435-628-1676

#6008
LLOYD RIED
POPE
CIVIL

3-29-21

Project Information

Project Number:	20045
Project Name:	Milan Lot 2
Project Location:	Henderson, Nevada

Project Design Criteria per IBC 2018
Gravity Loads

Roof:

Live Load	20 psf
Dead Load	25 psf

Floor:

Live Load	Living Space	40 psf
Dead Load	Wood Framed floor	15 psf

Lateral Loads:

Seismic:

Latitude: 36.0706° N Longitude: -114.9947° W

Seismic Design Category :	D
Site Class:	D
Occupancy Category:	II
Importance Factor:	1.00
Seismic force resisting system:	Light framed wall sheathed w/ wood structural panels
Response Modification Factor:	6.5
Overstrength Factor:	3.0
Deflection Amplification factor:	4.0
Design Base shear, V = CsW:	0.0648W
Analysis procedure:	Equivalent lateral force procedure
S_{DS}:	0.421

Wind:

Design Wind Speed:	115 mph
Exposure:	C
Occupancy category:	II
Importance Factor:	1.00
Height and exposure coefficient:	1.32

Deflection criteria

	Live load	Total load
Roof members:	L/360	L/240
Floor members:	L/360	L/240
Walls:	L/240	

Project Specifications

Allowable soil bearing capacity = 2000 psf
Soils Report = DuPont Engineering Inc dated June 30, 2019 project no. 19-0437
Concrete f'c = 4,500 psi
Masonry f'm = 2000 psi
Reinforcing steel: ASTM A615 Grade 60 steel
Solid sawn lumber = DF #2 (min)
Laminated veneer lumber = Fb = 2,600 psi (min), Fv = 285 psi (min)
Rectangular HSS sections – ASTM A500 GR. B, Fy = 46 ksi
Plates, bars, and other shapes – ASTM A36, Fy = 36 ksi
High strength bolts = ASTM A325
Anchor bolts and rods = ASTM F1554, GR. 36
Common Bolts = ASTM A307 GR. A
Post installed anchor bolts = As specified on construction drawings

** Materials provided to construct this project shall conform to the specifications listed above. No material specifications are to be changed without the consent of the engineer of record. Some aspects of the structural design may require different material specifications than what is listed above. In that case, those requirements will be noted in the construction drawings.**

General Notes and Requirements

Install Simpson straps, tie downs, and other hardware and meet all nailing, reinforcement and other structural requirements as noted on the construction drawings and within the pages of this document. The structural calculations are based on the structural criteria listed above. If the conditions listed herein are not met or are different from what was assumed, it shall be brought to the attention of the engineer. Roof truss system is to be engineered by the supplier and reviewed and approved by the engineer of record. All structural engineering has been performed according to the project soils report provided to this firm. In the event that a project soils report is not provided to this firm or does not exist, this engineering assumes that the building site is dry and stable with no adverse conditions or soils such as: a high water table, expansive clays, plastic clays, collapsible soils, fills etc. that could cause future flooding, settlement, site instability, or other adverse conditions. Any site engineering including grading, drainage, and site retaining walls is the responsibility of others. These calculations and engineering are for the building structure only and do not provide any engineering analysis of or liability/warranty for the non-structural portions of the building, or the site itself. The purpose of these calculations and engineering is to help reduce structural damage and loss of life due to seismic activity and/or high wind conditions. The contractor shall verify all conditions, dimensions and structural details of the drawing. Multiple uses of structural design calculations are not permitted.

1. Contractor to verify all dimensions, spans, and conditions and notify engineer of any errors, omissions, or discrepancies prior to construction.
2. If discrepancies are found in the project specifications, the more stringent specification shall be followed.

3. Contractor shall assure that all materials are used per manufactures recommendations.
4. Site engineering and liability shall be provided by the owner/builder as required.
5. Contractor shall assure that soil footings bear on is properly drained and dry prior to pouring foundation. Footings shall bear on undisturbed native soil or soil approved by the project geotechnical engineer a minimum of 14 inches below finished grade. Foundation shall have a minimum horizontal clearance from ascending slopes shall be a minimum of 25 feet unless approved by the project geotechnical engineer.
6. The contractor shall conform to all building codes and practices as per the IBC 2018 edition and its referenced standards.
7. Builder shall follow all recommendations found in the project soils report and all referenced documents, letters, and addendums.
8. Contractor to verify all dimensions, spans, and conditions with architectural drawings. If any omissions, mistakes, or discrepancies exist within the construction drawings, the engineer shall be promptly notified so that he may have the opportunity to take whatever steps necessary to resolve them. Failure to promptly notify the engineer of such conditions shall absolve the engineer from any responsibility for the consequences of such a failure.
9. If discrepancies are found, the more stringent specification shall be followed. Contractor is responsible for adequate bracing of structural members, walls, and non-structural items during construction.
10. The engineer and his consultants do not warrant or guarantee the accuracy and completeness of the work herein beyond a reasonable diligence. If any omissions, mistakes, or discrepancies are found to exist within the work product, the engineer shall be promptly notified so that he may have the opportunity to take whatever steps necessary to resolve them. Failure to promptly notify the engineer of such conditions shall absolve the engineer from any responsibility for the consequences of such a failure.
11. Many portions of the construction documents, notes, and specifications are the result of demands by various approving agencies that must be performed as part of this work product. Any actions taken without the knowledge and consent of the engineer shall become the responsibility not of the engineer, but of the parties responsible for making the change and taking action to do so. Action taken without the knowledge and consent of the engineer or the contradiction of the engineer's work product, the intent, and/or recommendations, shall become the responsibility not of the engineer, but of the parties responsible for taking such action. The engineer should be contacted in matters of any and all changes to the drawings and specifications herein without exception.
12. Non structural framing requirements are not specified on the structural drawings. See architectural drawings for any additional framing required.
13. Contractor shall assure that all products and hardware are used and installed per manufacturer's recommendations and requirements.

****Refer to Sheet S0.10 of the construction documents for additional project specifications and requirements****

Required Project Special Inspections

Soils inspections per IBC 1705.6 & Table 1705.6
Post installed anchor bolts per manufacturer and ICC report requirements

(2) 2X8 DF #2 Span Table

27-Jun-12

	Pitch 0.5:12		2:12		3:12		4:12		5:12		6:12	
Beam Clear Span	Trib. Width (ft)	Support Reaction (lbs)	Trib. Width (ft)	Support Reaction (lbs)	Trib. Width (ft)	Support Reaction (lbs)	Trib. Width (ft)	Support Reaction (lbs)	Trib. Width (ft)	Support Reaction (lbs)	Trib. Width (ft)	Support Reaction (lbs)
2.0	**70.83**	2796	**54.67**	2795	**54.17**	2795	**53.50**	2796	**52.67**	2795	**51.75**	2797
2.5	**57.92**	2795	**44.75**	2797	**44.33**	2797	**43.75**	2795	**43.08**	2796	**42.33**	2798
3.0	**49.00**	2796	**37.83**	2796	**37.50**	2797	**37.00**	2795	**36.42**	2795	**35.75**	2794
3.5	**42.42**	2794	**32.75**	2794	**32.42**	2792	**32.08**	2798	**31.58**	2798	**31.00**	2797
4.0	**32.67**	2585	**25.25**	2588	**25.00**	2586	**24.67**	2584	**24.33**	2589	**23.83**	2583
4.5	**26.42**	2325	**20.42**	2327	**20.25**	2330	**19.92**	2321	**19.67**	2328	**19.25**	2321
5.0	**21.75**	2108	**16.83**	2113	**16.67**	2112	**16.42**	2107	**16.17**	2107	**15.92**	2113
5.5	**18.25**	1932	**14.08**	1931	**13.92**	1926	**13.75**	1928	**13.58**	1934	**13.33**	1934
6.0	**15.50**	1781	**12.00**	1785	**11.83**	1777	**11.67**	1774	**11.50**	1777	**11.33**	1783
6.5	**13.33**	1652	**10.25**	1645	**10.17**	1647	**10.08**	1654	**9.92**	1652	**9.75**	1654
7.0	**11.58**	1540	**8.92**	1536	**8.83**	1535	**8.75**	1540	**8.58**	1535	**8.42**	1533
7.5	**10.17**	1445	**7.83**	1441	**7.75**	1440	**7.67**	1442	**7.50**	1433	**7.42**	1443
8.0	**8.92**	1349	**6.92**	1355	**6.83**	1351	**6.75**	1352	**6.67**	1356	**6.50**	1347
8.5	**7.92**	1271	**6.08**	1265	**6.08**	1276	**6.00**	1275	**5.92**	1277	**5.75**	1264
9.0	**7.08**	1203	**5.50**	1209	**5.42**	1202	**5.33**	1199	**5.25**	1199	**5.17**	1201
9.5	**6.42**	1149	**4.92**	1141	**4.83**	1132	**4.83**	1146	**4.75**	1144	**4.67**	1145
10.0	**5.75**	1084	**4.42**	1079	**4.42**	1088	**4.33**	1082	**4.25**	1078	**4.17**	1076

Continued

	7:12		8:12		9:12		10:12		11:12		12:12	
Beam Clear Span	Trib. Width (ft)	Support Reaction (lbs)	Trib. Width (ft)	Support Reaction (lbs)	Trib. Width (ft)	Support Reaction (lbs)	Trib. Width (ft)	Support Reaction (lbs)	Trib. Width (ft)	Support Reaction (lbs)	Trib. Width (ft)	Support Reaction (lbs)
2.0	**50.67**	2796	**49.58**	2797	**48.42**	2797	**47.17**	2794	**46.00**	2796	**44.83**	2798
2.5	**41.42**	2794	**40.50**	2794	**39.58**	2796	**38.58**	2795	**37.58**	2793	**36.67**	2798
3.0	**35.00**	2792	**34.25**	2794	**33.50**	2798	**32.67**	2798	**31.83**	2797	**31.00**	2797
3.5	**30.33**	2793	**29.67**	2794	**29.00**	2796	**28.25**	2793	**27.58**	2798	**26.83**	2795
4.0	**23.42**	2590	**22.83**	2583	**22.33**	2587	**21.75**	2583	**21.25**	2589	**20.67**	2586
4.5	**18.92**	2327	**18.50**	2328	**18.08**	2330	**17.58**	2323	**17.17**	2327	**16.67**	2319
5.0	**15.58**	2112	**15.25**	2113	**14.83**	2105	**14.50**	2109	**14.17**	2115	**13.75**	2107
5.5	**13.08**	1936	**12.75**	1930	**12.42**	1925	**12.17**	1933	**11.83**	1929	**11.50**	1925
6.0	**11.08**	1780	**10.83**	1779	**10.58**	1780	**10.33**	1781	**10.08**	1784	**9.83**	1786
6.5	**9.50**	1645	**9.33**	1653	**9.08**	1647	**8.83**	1643	**8.67**	1653	**8.42**	1649
7.0	**8.25**	1534	**8.08**	1536	**7.92**	1541	**7.67**	1530	**7.50**	1536	**7.33**	1542
7.5	**7.25**	1440	**7.08**	1439	**6.92**	1439	**6.75**	1439	**6.58**	1440	**6.42**	1441
8.0	**6.42**	1357	**6.25**	1351	**6.08**	1347	**5.92**	1343	**5.83**	1359	**5.67**	1355
8.5	**5.67**	1271	**5.58**	1281	**5.42**	1272	**5.25**	1265	**5.17**	1277	**5.00**	1269
9.0	**5.08**	1206	**4.92**	1193	**4.83**	1201	**4.75**	1210	**4.58**	1198	**4.50**	1208
9.5	**4.58**	1147	**4.42**	1131	**4.33**	1136	**4.25**	1142	**4.17**	1149	**4.00**	1133
10.0	**4.08**	1076	**4.00**	1078	**3.92**	1081	**3.83**	1085	**3.75**	1089	**3.67**	1093

(1) Trimmer provided for spans less than 4'. (2) Trimmers for spans equal to and greater than 4'.
Uniform Loading is as follows: Pitched Roof, DL 25 psf, LL 20 psf and Flat Roof, DL 15 psf, LL 20 psf.
Unbraced Length is equal to beam's clear span
Deflection Criteria is as follows, LL L/360, TL L/240
Created with LR Pope Engr. - ASD Wood Member Design v7.0.5 (10-20-11)

(2) 2X10 DF #2 Span Table

27-Jun-12

	Pitch 0.5:12		2:12		3:12		4:12		5:12		6:12	
Beam Clear Span	Trib. Width (ft)	Support Reaction (lbs)	Trib. Width (ft)	Support Reaction (lbs)	Trib. Width (ft)	Support Reaction (lbs)	Trib. Width (ft)	Support Reaction (lbs)	Trib. Width (ft)	Support Reaction (lbs)	Trib. Width (ft)	Support Reaction (lbs)
2.0	**70.83**	2798	**54.67**	2796	**54.17**	2796	**53.50**	2797	**52.67**	2797	**51.67**	2795
2.5	**57.92**	2797	**44.67**	2794	**44.25**	2794	**43.75**	2797	**43.08**	2798	**42.25**	2795
3.0	**48.92**	2794	**37.75**	2792	**37.42**	2794	**37.00**	2798	**36.42**	2797	**35.75**	2796
3.5	**42.42**	2797	**32.75**	2797	**32.42**	2794	**32.00**	2793	**31.50**	2793	**30.92**	2792
4.0	**48.75**	3855	**37.67**	3858	**37.33**	3859	**36.83**	3856	**36.25**	3855	**35.58**	3854
4.5	**39.42**	3467	**30.42**	3465	**30.17**	3468	**29.75**	3464	**29.25**	3459	**28.75**	3463
5.0	**32.42**	3139	**25.08**	3146	**24.83**	3144	**24.50**	3141	**24.17**	3147	**23.67**	3139
5.5	**27.17**	2873	**21.00**	2876	**20.75**	2869	**20.50**	2870	**20.25**	2880	**19.83**	2873
6.0	**23.08**	2648	**17.83**	2650	**17.67**	2649	**17.42**	2645	**17.17**	2648	**16.83**	2645
6.5	**19.83**	2454	**15.33**	2457	**15.17**	2453	**15.00**	2457	**14.75**	2454	**14.50**	2457
7.0	**17.25**	2290	**13.33**	2292	**13.17**	2285	**13.00**	2284	**12.83**	2291	**12.58**	2287
7.5	**15.08**	2139	**11.67**	2142	**11.50**	2132	**11.42**	2143	**11.25**	2145	**11.00**	2136
8.0	**13.33**	2012	**10.25**	2003	**10.17**	2006	**10.08**	2014	**9.92**	2012	**9.75**	2015
8.5	**11.83**	1894	**9.17**	1900	**9.08**	1900	**8.92**	1890	**8.75**	1884	**8.67**	1900
9.0	**10.58**	1791	**8.17**	1790	**8.08**	1789	**8.00**	1793	**7.83**	1783	**7.75**	1796
9.5	**9.50**	1696	**7.33**	1696	**7.25**	1692	**7.17**	1694	**7.08**	1700	**6.92**	1691
10.0	**8.58**	1612	**6.58**	1602	**6.58**	1617	**6.50**	1616	**6.33**	1600	**6.25**	1608

Continued

	7:12		8:12		9:12		10:12		11:12		12:12	
Beam Clear Span	Trib. Width (ft)	Support Reaction (lbs)	Trib. Width (ft)	Support Reaction (lbs)	Trib. Width (ft)	Support Reaction (lbs)	Trib. Width (ft)	Support Reaction (lbs)	Trib. Width (ft)	Support Reaction (lbs)	Trib. Width (ft)	Support Reaction (lbs)
2.0	**50.67**	2797	**49.50**	2794	**48.33**	2794	**47.17**	2795	**46.00**	2797	**44.75**	2794
2.5	**41.42**	2796	**40.50**	2796	**39.50**	2793	**38.58**	2797	**37.58**	2795	**36.58**	2794
3.0	**35.00**	2794	**34.25**	2796	**33.42**	2794	**32.58**	2793	**31.75**	2792	**30.92**	2792
3.5	**30.33**	2796	**29.67**	2796	**28.92**	2791	**28.25**	2795	**27.50**	2792	**26.83**	2797
4.0	**34.92**	3860	**34.08**	3853	**33.33**	3859	**32.50**	3857	**31.67**	3856	**30.83**	3855
4.5	**28.17**	3463	**27.58**	3468	**26.92**	3465	**26.25**	3465	**25.58**	3465	**24.92**	3465
5.0	**23.17**	3136	**22.67**	3138	**22.17**	3142	**21.58**	3137	**21.08**	3144	**20.50**	3139
5.5	**19.42**	2871	**19.00**	2872	**18.58**	2877	**18.08**	2870	**17.67**	2877	**17.17**	2871
6.0	**16.50**	2646	**16.17**	2651	**15.75**	2645	**15.42**	2654	**15.00**	2650	**14.58**	2645
6.5	**14.17**	2450	**13.92**	2461	**13.58**	2460	**13.25**	2460	**12.92**	2460	**12.50**	2445
7.0	**12.33**	2288	**12.08**	2292	**11.75**	2283	**11.50**	2291	**11.17**	2282	**10.92**	2291
7.5	**10.75**	2131	**10.58**	2145	**10.33**	2145	**10.08**	2146	**9.75**	2129	**9.50**	2130
8.0	**9.50**	2004	**9.33**	2013	**9.08**	2006	**8.83**	2001	**8.67**	2014	**8.42**	2008
8.5	**8.42**	1883	**8.25**	1888	**8.08**	1894	**7.83**	1882	**7.67**	1890	**7.50**	1898
9.0	**7.58**	1794	**7.42**	1794	**7.25**	1796	**7.00**	1778	**6.83**	1781	**6.67**	1784
9.5	**6.75**	1685	**6.67**	1701	**6.50**	1699	**6.33**	1697	**6.17**	1695	**6.00**	1694
10.0	**6.17**	1619	**6.00**	1611	**5.83**	1604	**5.67**	1598	**5.58**	1615	**5.42**	1609

(1) Trimmer provided for spans less than 4'. (2) Trimmers for spans equal to and greater than 4'.

Uniform Loading is as follows: Pitched Roof, DL 25 psf, LL 20 psf and Flat Roof, DL 15 psf, LL 20 psf.

Unbraced Length is equal to beam's clear span

Deflection Criteria is as follows, LL L/360, TL L240

Created with LR Pope Engr. - ASD Wood Member Design v7.0.5 (10-20-11)

(2) 9-1/2" LVL Span Table

27-Jun-12

	Pitch 0.5:12		2:12		3:12		4:12		5:12		6:12		7:12		8:12		9:12		10:12		11:12		12:12	
Beam Clear Span	Trib. Width (ft)	Support Reaction (lbs)	Trib. Width (ft)	Sprt. Rea. (Lbs)	Trib. Width (ft)	Sprt. Rea. (Lbs)	Trib. Width (ft)	Sprt. Rea. (Lbs)	Trib. Width (ft)	Sprt. Rea. (Lbs)	Trib. Width (ft)	Sprt. Rea. (Lbs)	Trib. Width (ft)	Sprt. Rea. (Lbs)	Trib. Width (ft)	Sprt. Rea. (Lbs)	Trib. Width (ft)	Sprt. Rea. (Lbs)	Trib. Width (ft)	Sprt. Rea. (Lbs)	Trib. Width (ft)	Sprt. Rea. (Lbs)	Trib. Width (ft)	Sprt. Rea. (Lbs)
2.0	82.58	3264	63.75	3263	63.17	3263	62.33	3261	61.42	3264	60.25	3261	59.08	3264	57.75	3262	56.42	3264	55.00	3262	53.58	3261	52.25	3265
2.5	67.50	3263	52.08	3261	51.58	3260	51.00	3264	50.17	3261	49.25	3261	48.25	3260	47.25	3265	46.08	3261	45.00	3264	43.83	3263	42.67	3261
3.0	57.08	3264	44.08	3264	43.67	3263	43.08	3261	42.42	3261	41.67	3262	40.83	3263	39.92	3262	39.00	3264	38.00	3260	37.08	3265	36.08	3262
3.5	52.50	3465	38.17	3263	37.75	3258	37.33	3263	36.75	3263	36.08	3262	35.33	3261	34.58	3263	33.75	3261	32.92	3261	32.08	3261	31.25	3262
4.0	82.58	6528	63.75	6526	63.17	6527	62.33	6523	61.42	6528	60.25	6522	59.08	6528	57.75	6525	56.42	6527	55.00	6524	53.58	6522	52.25	6530
4.5	74.25	6524	57.33	6524	56.83	6527	56.08	6523	55.25	6528	54.25	6528	53.17	6530	51.92	6520	50.75	6527	49.50	6526	48.25	6528	47.00	6528
5.0	67.50	6526	52.08	6521	51.58	6519	51.00	6528	50.17	6522	49.25	6521	48.25	6521	47.25	6530	46.08	6522	45.00	6529	43.83	6526	42.67	6522
5.5	61.83	6524	47.75	6525	47.33	6528	46.67	6518	46.00	6527	45.17	6526	44.25	6526	43.25	6523	42.25	6525	41.17	6518	40.17	6526	39.08	6520
6.0	57.08	6527	44.08	6528	43.67	6527	43.08	6522	42.42	6522	41.67	6525	40.83	6527	39.92	6524	39.00	6527	38.00	6521	37.08	6529	36.08	6523
6.5	53.00	6529	40.92	6528	40.50	6522	40.00	6523	39.42	6529	38.67	6523	37.92	6529	37.08	6530	36.17	6521	35.25	6516	34.42	6528	33.50	6524
7.0	49.42	6525	38.17	6526	37.75	6516	37.33	6526	36.75	6525	36.08	6525	35.33	6521	34.58	6527	33.75	6523	32.92	6522	32.08	6523	31.25	6523
7.5	46.33	6528	35.75	6523	35.42	6523	35.00	6528	34.42	6521	33.83	6528	33.08	6516	32.42	6528	31.58	6513	30.83	6519	30.08	6526	29.25	6515
8.0	43.58	6527	33.67	6529	33.33	6525	32.92	6526	32.42	6528	31.83	6529	31.17	6524	30.50	6528	29.75	6521	29.00	6517	28.25	6514	27.50	6511
8.5	39.58	6280	30.58	6284	30.25	6274	29.92	6284	29.42	6276	28.92	6283	28.33	6284	27.67	6274	27.08	6290	26.42	6290	25.67	6271	25.00	6271
9.0	34.67	5812	26.75	5808	26.50	5807	26.17	5807	25.75	5805	25.33	5816	24.83	5819	24.25	5811	23.67	5807	23.08	5807	22.50	5808	21.92	5809
9.5	29.67	5242	22.92	5244	22.67	5236	22.42	5244	22.08	5247	21.67	5243	21.25	5249	20.75	5241	20.25	5238	19.75	5237	19.25	5238	18.75	5238
10.0	25.58	4754	19.75	4753	19.58	4757	19.33	4756	19.00	4747	18.67	4750	18.33	4762	17.92	4758	17.50	4760	17.08	4763	16.58	4745	16.17	4749
10.5	22.25	4338	17.17	4335	17.00	4333	16.75	4324	16.50	4326	16.25	4339	15.92	4338	15.50	4320	15.17	4328	14.83	4340	14.42	4328	14.08	4341
11.0	19.42	3965	15.00	3967	14.83	3960	14.67	3965	14.42	3959	14.17	3962	13.83	3949	13.58	3965	13.25	3960	12.92	3958	12.58	3957	12.25	3955
11.5	17.08	3647	13.17	3640	13.00	3628	12.83	3627	12.67	3637	12.42	3631	12.17	3631	11.92	3636	11.67	3646	11.33	3631	11.08	3643	10.75	3629
12.0	15.08	3361	11.58	3343	11.50	3350	11.33	3344	11.17	3347	11.00	3357	10.75	3349	10.50	3345	10.25	3344	10.00	3345	9.75	3346	9.50	3347
12.5	13.33	3097	10.25	3084	10.17	3088	10.08	3101	9.92	3098	9.75	3102	9.50	3085	9.33	3099	9.08	3089	8.83	3080	8.67	3100	8.42	3091
13.0	11.92	2882	9.17	2871	9.08	2872	9.00	2881	8.83	2873	8.67	2871	8.50	2874	8.33	2881	8.08	2862	7.92	2873	7.67	2856	7.50	2868
13.5	10.58	2662	8.17	2660	8.08	2658	8.00	2664	7.92	2677	7.75	2669	7.58	2666	7.42	2666	7.25	2669	7.08	2673	6.92	2678	6.67	2651
14.0	9.50	2482	7.33	2481	7.25	2476	7.17	2479	7.08	2488	6.92	2475	6.83	2495	6.67	2489	6.50	2485	6.33	2483	6.17	2481	6.00	2478
14.5	8.58	2327	6.58	2312	6.58	2333	6.50	2332	6.33	2309	6.25	2320	6.08	2306	6.00	2325	5.83	2315	5.67	2306	5.58	2330	5.42	2322
15.0	7.75	2178	6.00	2184	5.92	2174	5.83	2171	5.75	2173	5.67	2181	5.50	2161	5.42	2176	5.25	2160	5.17	2179	5.00	2164	4.92	2184
15.5	7.00	2038	5.42	2043	5.33	2030	5.25	2024	5.17	2024	6.08	2028	5.00	2035	4.92	2046	4.75	2025	4.67	2039	4.50	2018	4.42	2033
16.0	6.33	1909	4.92	1919	4.83	1905	4.83	1928	4.75	1925	4.67	1926	4.50	1897	4.42	1904	4.33	1912	4.25	1922	4.08	1896	4.00	1907
16.5	5.75	1794	4.50	1817	4.42	1801	4.33	1790	4.33	1817	4.25	1815	4.17	1816	4.00	1784	3.92	1789	3.83	1794	3.75	1801	3.67	1808
17.0	5.25	1693	4.08	1705	4.00	1687	4.00	1707	3.92	1698	3.83	1693	3.75	1691	3.67	1691	3.58	1692	3.50	1694	3.42	1697	3.33	1699
17.5	4.83	1610	3.75	1618	3.67	1598	3.67	1617	3.58	1606	3.50	1598	3.42	1592	3.33	1589	3.25	1586	3.17	1585	3.08	1583	3.00	1582
18.0	4.42	1520	3.42	1523	3.33	1501	3.33	1519	3.25	1505	3.25	1531	3.17	1523	3.08	1517	3.00	1512	2.92	1507	2.83	1503	2.75	1498

(1) Trimmer provided for spans less than 4'. (2) Trimmers for spans equal to and greater than 4'.

Uniform Loading is as follows: Pitched Roof, DL 25 psf, LL 20 psf and Flat Roof, DL 15 psf, LL 20 psf.

Unbraced Length is equal to beam's clear span

Deflection Criteria is as follows, LL L/360, TL L240

Created with LR Pope Engr. - ASD Wood Member Design v7.0.5 (10-20-11)

Floor Span Table

27-Jun-12

	(2) 2X10DF#2		(2) 9-1/2"LVL		(2) 11-78" LVL	
Beam Clear Span	Trib. Width (ft)	Support Reaction (lbs)	Trib. Width (ft)	Support Reaction (lbs)	Trib. Width (ft)	Support Reaction (lbs)
2.0	**45.08**	2797	**52.58**	3265	**52.50**	3262
2.5	**36.83**	2795	**42.92**	3259	**42.92**	3262
3.0	**31.17**	2796	**36.33**	3263	**36.25**	3259
3.5	**27.00**	2797	**31.42**	3258	**31.42**	3263
4.0	**24.83**	3088	**50.58**	6282	**52.50**	6524
4.5	**20.08**	2778	**45.50**	6280	**47.25**	6527
5.0	**16.50**	2514	**41.33**	6278	**42.92**	6524
5.5	**13.83**	2302	**37.92**	6285	**39.33**	6526
6.0	**11.75**	2122	**34.92**	6273	**36.25**	6519
6.5	**10.08**	1964	**32.42**	6274	**33.67**	6523
7.0	**8.75**	1829	**29.50**	6121	**31.42**	6525
7.5	**7.67**	1713	**25.83**	5722	**29.42**	6520
8.0	**6.75**	1606	**22.83**	5379	**27.67**	6519
8.5	**6.00**	1515	**20.33**	5076	**26.08**	6510
9.0	**5.42**	1446	**18.17**	4792	**24.75**	6524
9.5	**4.83**	1362	**16.33**	4540	**23.50**	6523
10.0	**4.33**	1286	**14.75**	4310	**22.17**	6464
10.5	x	x	**13.08**	4011	**20.08**	6142
11.0	x	x	**11.50**	3693	**18.25**	5841
11.5	x	x	**10.08**	3386	**16.67**	5573
12.0	x	x	**8.92**	3126	**15.33**	5347
12.5	x	x	**7.92**	2893	**14.08**	5114
13.0	x	x	**7.08**	2695	**13.00**	4908
13.5	x	x	**6.33**	2506	**12.00**	4705
14.0	x	x	**5.67**	2330	**11.08**	4507
14.5	x	x	**5.17**	2204	**10.08**	4250
15.0	x	x	**4.67**	2064	**9.17**	4001
15.5	x	x	**4.25**	1948	**8.33**	3764
16.0	x	x	**3.83**	1819	**7.58**	3541
16.5	x	x	**3.50**	1719	**6.92**	3337
17.0	x	x	**3.25**	1649	**6.33**	3154
17.5	x	x	**3.00**	1572	**5.83**	2997
18.0	x	x	**2.75**	1489	**5.33**	2825

(1) Trimmer provided for spans less than 4', (2) Trimmers all else.
Uniform Loading is as follows: DL 15 psf, LL 40 psf.
Unbraced Length is equal to beam's clear span
Deflection Criteria is as follows, LL L/360, TL L240
Created with LR Pope Engr. - ASD Wood Member Design v7.0.5 (10-20-11)

Wood Framed Shear Wall Schedule

SW-1 7/16" APA rated sheathing, exp. 1 with 8d common nails at 6" o.c. along panel edges and 12" o.c. at intermediate supports. Bolt 2x sill plate to foundation with 1/2" dia. x 10" anchor bolts & 0.229" x 3" x 3" steel plate washers at 48" o.c. Nail 2x sill plate to wood floor with 16d common nails at 12" o.c. Allowable shear = 140 plf.

SW-2 7/16" APA rated sheathing, exp. 1 with 8d common nails at 6" o.c. along panel edges and 12" o.c. at intermediate supports. Bolt 2x sill plate to foundation with 1/2" dia. x 10" anchor bolts & 0.229" x 3" x 3" steel plate washers at 32" o.c. Nail 2x sill plate to wood floor with 16d common nails at 6" o.c. Allowable shear = 260 plf.

SW-3 7/16" APA rated sheathing, exp. 1 with 8d common nails at 4" o.c. along panel edges and 12" o.c. at intermediate supports. Bolt 2x sill plate to foundation with 1/2" dia. x 10" anchor bolts & 0.229" x 3" x 3" steel plate washers at 23" o.c. Nail 2x sill plate to wood floor with 16d common nails at 4-3/4" o.c. Allowable shear = 350 plf.

SW-4 7/16" APA rated sheathing, exp. 1 with 8d common nails at 3" o.c. along panel edges and 12" o.c. at intermediate supports. Framing at adjoining panel edges shall be 3x or double 2x. Nails shall be staggered at adjoining panel edges. Bolt 2x sill plate to foundation w/ 1/2" dia. x 10" anchor bolts & 0.229" x 3" x 3" steel plate washers at 17" o.c. Nail 2x sill plate to wood floor with 16d common nails at 3-1/2" o.c. Allowable shear = 490 plf.

SW-5 7/16" APA rated sheathing, exp. 1 with 10d common nails at 2" o.c. along panel edges and 12" o.c. at intermediate supports. Framing at adjoining panel edges shall be 3x or double 2x. Nails shall be staggered at all panel edges. Bolt 2x sill plate to foundation w/ 5/8" dia. x 10" anchor bolts & 0.229" x 3" x 3" steel plate washers at 24" o.c. Nail 2x sill plate to wood floor with (2) 16d common nails at 5" o.c. Allowable shear = 640 plf

SW-6 15/32" APA rated sheathing, exp. 1 with 10d common nails at 2" o.c. along panel edges and 12" o.c. at intermediate supports. Framing at adjoining panel edges shall be 3x or double 2x. Nails shall be staggered at all panel edges. Bolt 2x sill plate to foundation w/ 5/8" dia. x 10" anchor bolts & 0.229" x 3" x 3" steel plate washers at 18" o.c. Nail 2x sill plate to wood floor with (2) 16d common nails at 4" o.c. Allowable shear = 770 plf

SW-7 19/32" APA rated sheathing, exp. 1 with 10d common nails at 2" o.c. along panel edges and 12" o.c. at intermediate supports. Framing at adjoining panel edges shall be 3x or double 2x. Nails shall be staggered at all panel edges. Bolt 2x sill plate to foundation w/ 5/8" dia. x 10" anchor bolts & 0.229" x 3" x 3" steel plate washers at 14" o.c. Nail 2x sill plate to wood floor with (2) 16d common nails at 3" o.c. Allowable shear = 870 plf

Cast in anchor bolts for interior shear walls may be replaced with Simpson Strong bolts, Titen HD, or Hilti Kwik Bolt TZ anchors of the same diameter and 4-1/2" minimum embedment. Interior shear wall anchor bolts may also be epoxied into concrete with Hilti HIT-RE 500-SD epoxy and a minimum 4-1/2" embedment. Interior shear walls shall extend to bottom of floor sheathing or roof sheathing. 0.229"x3"x3" steel plate washers shall extend to within 1/2" from edge of sill plate on the sheathed side of the wall

Wood Framed Roof Diaphragm Schedule

RD-1 7/16" APA rated sheathing, exp. 1, unblocked with 8d common nails at 6" o.c. along diaphragm perimeter, shear wall lines, and supported panel edges and 8d common nails at 12" o.c. in the field.
Allowable Shear = 230 plf (Minimum required roof diaphragm nailing)

RD-2 7/16" APA rated sheathing, exp. 1, blocked with 8d common nails at 4" o.c. along diaphragm perimeter and shear wall lines. 8d common nails at 6" o.c. at all other supported panel edges and 8d common nails at 12" o.c. in the field.
Allowable Shear = 340 plf

RD-3 19/32" APA rated sheathing, exp. 1, blocked with 10d common nails at 4" o.c. along diaphragm perimeter and shear wall lines. 10d common nails at 6" o.c. at all other supported panel edges and 10d common nails at 12" o.c. in the field. Allowable Shear = 425 plf

RD-4 19/32" APA rated sheathing, exp. 1, blocked with 10d common nails at 2 1/2" o.c. along diaphragm perimeter and shear wall lines. 10d common nails at 4" o.c. at all other supported panel edges and 10d common nails at 12" o.c. in the field. Panel edges to be supported/blocked with 3" nominal framing or wider.
Allowable Shear = 640 plf

Wood Framed Floor Diaphragm Schedule

FD-1 3/4" tongue and groove APA rated sheathing exp. 1, Case 1, unblocked with 10d common nails at 6" o.c. along diaphragm perimeter, shear wall lines, and supported panel edges and 10d common nails at 12" o.c. in the field. Floor sheathing shall be glued to all supports in addition to required diaphragm nailing.
Allowable Shear = 285 plf (Minimum required floor diaphragm nailing)

FD-2 3/4" tongue and groove APA rated sheathing, exp. 1, Case 1, blocked with 10d common nails at 4" o.c. along diaphragm perimeter and shear wall lines. 10d common nails at 12" o.c. at all other supported panel edges and 10d common nails at 12" o.c. in the field. Floor sheathing shall be glued to all supports in addition to required diaphragm nailing.
Allowable Shear = 425 plf

Top Plate Splice Schedule

Bearing, shear and exterior wall studs shall be capped with double top plates installed to provide overlapping at corners and at intersections with other partitions. Double top plates shall be nailed with 16d nails at 16" o.c. end joints in double top plates shall be offset at least 48" unless noted otherwise 8-16d nails shall be placed each side of top chord splice.

TC-1 8-16d nails = 8 x 93 x 1.6 = 1,190 lbs (Minimum)
TC-2 10-16d nails = 1,480 lbs
TC-3 15-16d nails = 2,230 lbs
TC-4 20-16d nails = 2,970 lbs
TC-5 24-16d nails = 3,570 lbs
TC-6 Simpson MST48 strap = 4,840 lbs
TC-7 Simpson MST60 strap = 6,420 lbs

LATERAL LOAD ANALYSIS FOR MILAN LOT 2
BY L.R. POPE ENGINEERING, INC.

Version 9.3.2 (9/14/15)

CONSTRUCTION TYPE = RESIDENTIAL

WIND LOADS
SIMPLIFIED WIND LOAD METHOD (ASCE 7-10 28.6.3)

Risk Category = II

BASIC WIND SPEED = 115 MPH

EXPOSURE = C

	Main	**Alternate**	
Roof Height (r) =	5.33	4.67	ft
Wall Height 2nd Level (hs) =	9.08	16.08	ft
Width of Floor (f) =	2.00		ft
Wall Height First Level (hf) =	9.08		ft
ROOF PITCH =	4	4	:12
ROOF TYPE =	HIP	HIP	
Topographical factor, K_{zt} =	1.00		

Htotal = hs+f+hf = 25.49

mean roof ht, h = 22.825

Building ht & exposure, λ = 1.32 ASCE 7-10 Figure 28.6-1

Wind pressure zone A, p_{s30} = 28.15 psf

Wind pressure zone B, p_{s30} = -8.01 psf

Wind pressure zone C, p_{s30} = 18.81 psf

Wind pressure zone D, p_{s30} = -4.45 psf

Wind pressure zone E_{OH}, p_{s30} = -35.30 psf

Wind pressure zone G_{OH}, p_{s30} = -27.60 psf

ASCE 7-10 Figure 28.6-1

EDGE STRIPS (a)		END ZONES 2(a)	
LONG. (a) =	3.78	2(a)=	7.57
TRANS. (a) =	3.00	2(a)=	6.00

Formula

WIND LOAD, $p_s = \lambda K_{zt} I p_{S30}$

	Results		
	TRANS	**LONG**	
WALL END ZONE A, $\lambda \mathbf{K_{zt}} I_W p_{S30}$ =	37.3	37.3	psf
ROOF END ZONE B, $\lambda \mathbf{K_{zt}} I_W p_{S30}$ =	10.6	10.6	psf
WALL INTERIOR ZONE C, $\lambda \mathbf{K_{zt}} I_W p_{S30}$ =	24.9	24.9	psf
ROOF INTERIOR ZONE D, $\lambda \mathbf{K_{zt}} I_W p_{S30}$ =	5.9	5.9	psf
OVERHANG INTERIOR ZONE E_{OH}, $\lambda \mathbf{K_{zt}} I_W p_{S30}$ =	-46.7	-46.7	psf
OVERHANG INTERIOR ZONE G_{OH}, $\lambda \mathbf{K_{zt}} I_W p_{S30}$ =	-36.5	-36.5	psf

****Min allowable pressure = ±16 psf (Zones A & C) and ±8 psf (Zones B & D) per ASCE 7-10 28.6.4****

Total Wind Load at Roof

w=0.6*WL*(r+hs/2)

		Main	**Alternate**	
TRANSVERSE	**END ZONE w=**	**135**	**209.5357**	**plf**
	INTERIOR w=	**87**	**136.6089**	**plf**
LONGITUDINAL	**END ZONE w=**	**135**	**209.5357**	**plf**
	INTERIOR w=	**87**	**136.6089**	**plf**

Total Wind Load at Floor

w=WL*(r+hs+hf/2)

		Main	**Alternate**	
TRANSVERSE	**END ZONE w=**	**248**	**179.8109**	**plf**
	INTERIOR w=	**166**	**22.416**	**plf**
LONGITUDINAL	**END ZONE w=**	**248**	**179.8109**	**plf**
	INTERIOR w=	**166**	**120.0998**	**plf**

SEISMIC FORCES
EQUIVALENT LATERAL FORCE PROCEDURE (ASCE 7-10 12.8)

ZIP CODE = 0

LAT 36.0706° LONG :-114.9447°

Risk Category = II

Seismic Design Category = **D** IBC Tables 1613.3.5(1),1613.3.5(2)

Site Class = **D**

Seismic Importance Factor, I_e= 1.00

S_S= 0.486

S_1= 0.163

Response modification coefficient, R = 6.5 ASCE 7-10 TABLE 12.2-1

Upper roof area (Ar_2) = 1098 ft^2

Lower roof area (Ar_1) = 7153 ft^2

Floor area (Af) = 821 ft^2

2nd story or roof length (Lr) = 24.92 ft

2nd story or roof width (Wr) = 37.83 ft

1st story or floor length (Lf) = 98.5 ft

1st story or floor width (Wf) = 76.5 ft

Height of 2nd Story Wall (hs) = 9.08 ft

Height of First Story Wall (hf) = 9.08 ft

Weight of Exterior Walls (Ww) = 15 psf

Roof Dead Load (Rdl) = 25 psf

Floor Dead Load + parition (Fdl) = 15 psf

Trib. wt @ roof, w_2 =Ar*Rdl+hs*Ww*(Lr+Wr) =

w_2 = 38193 **lbs**

Trib. wt @ floor, w_1 = Fdl*Af+Ar_1*Rdl+Ww*(Lf+Wf)*(hs/2+hf/2) =

w_1 = 254758 **lbs**

T_L = 8.00 **sec**

Fv= 2.15

Fa= 1.41

S_{MS} = Fa*S_S = 0.686

S_{M1} = Fv*S_1 = 0.350

S_{DS} = 2/3*S_{MS} = 0.421

S_{D1} = 2/3*S_{M1} = 0.163

Cs = S_{DS}/(R/Ie) = 0.0648

Csmax = SD1/(T*(R/Ie)) = 0.111

Cs_{min} = 0.019

T_s = S_{D1}/S_{DS} = 0.387 **sec**

x = 0.75

C_t = 0.020

Period, T = $C_t h_n^x$ = 0.227 **sec**

Frequency = 1/T = 4.408 **Hz**

LOADS AND EQUATIONS

Total wt, $W = w_1+w_2$ = 292951 **lbs**

$V = C_sW$ = 18974 **lbs**

k = 1.000

769962

$w_1h_1^k$ = 2822719

$C_{v2} = w_2h_2^k/(w_1h_1^k+w_2h_2k)$ = 0.21

$C_{v1} = w_1h_1^k/(w_1h_1^k+w_2h_2^k)$ = 0.79

Redundancy factor calculation 1.00 < ρ_x < 1.30

Long roof % = 1.000 ρ_x = 1.300

Long floor % = 1.000 ρ_x = 1.300

Trans roof % = 1.000 ρ_x = 1.300

Trans floor % = 1.000 ρ_x = 1.300

Maximum Roof story ρ_x = 1.300

Maximum Floor ρ_x = 1.300

Story forces (ASCE 7-10 EQN 12.8-11)

Story force @ roof, $F_2 = C_{V2}$*V = 4066 **lbs**

Story force @ floor, $F_2 = C_{V1}$*V = 14908 **lbs**

Story shear @ roof, V_2 = 4066 **lbs**

Story shear @ floor, V_1 = 14908 **lbs**

Total story shear forces

Long. Diaphragm load @ roof, F_2/W_r = 107 plf

Long. Diaphragm load @ floor, $(F_1+F_2)/W_f$ = 195 plf

Trans. Diaphragm load @ roof, F_2/L_r = 163 plf

Trans. Diaphragm load @ floor, $(F_1+F_2)/L_f$ = 151 plf

Longitudinal Seismic Loads

Seismic load @ roof, 0.7*E = 0.7*ρQ_E = 98 plf

Seismic load @ floor, 0.7*E = 0.7*ρQ_E = 177 plf

Transverse Seismic Loads

Seismic load @ roof, 0.7*E = 0.7*ρQ_E = 148 plf

Seismic load @ floor, 0.7*E = 0.7*ρQ_E = 138 plf

ROOF LEVEL LONGITUDINAL END ZONES GOVERNED BY WIND LOADS (135 plf)
ROOF LEVEL LONGITUDINAL INTERIOR ZONES GOVERNED BY SEISMIC LOADS (98 plf)
ROOF LEVEL TRANSVERSE END ZONES GOVERNED BY SEISMIC LOADS (148 plf)
ROOF LEVEL TRANSVERSE INTERIOR ZONES GOVERNED BY SEISMIC LOADS (148 plf)
FLOOR LEVEL LONGITUDINAL END ZONES GOVERNED BY WIND LOADS (248 plf)
FLOOR LEVEL LONGITUDINAL INTERIOR ZONES GOVERNED BY SEISMIC LOADS (177 plf)
FLOOR LEVEL TRANSVERSE END ZONES GOVERNED BY WIND LOADS (248 plf)
FLOOR LEVEL TRANSVERSE INTERIOR ZONES GOVERNED BY WIND LOADS (166 plf)

LATERAL LOAD ANALYSIS FOR MILAN LOT 2

Grid Line P1

MAIN OR ALT. ROOF?	ALT.
LONGITUDINAL OR TRANSVERSE?	T
END ZONE OR INTERIOR?	E

Formula		Results	Units
P=WL/Vs*sls/2	**Wind Shear Load (P)=**	**1838**	**lbs**
Us=P/Sw	**Unit Shear (Us)=**	**50**	**plf**
P=WL/Vs*sls/2	**Seismic Shear Load (P)=**	**1522**	**lbs**
Us=P/Sw	**Unit Shear (Us)=**	**42**	**plf**

At Roof **Wind governs shear wall design**

End Zone Wind Load (WL/Vs)=	210	plf
Interior Zone Wind Load (WL/Vs)=	137	plf
Seismic Load ,(WL/Vs) =	148	plf
Shear Load Span (sls)=	20.50	ft
Roof Dead Load (Rdl)=	25	psf
Wall Weight (wwt)=	15	psf
Length of Shear Wall (Sw)=	36.50	ft

Wind end zone width = **6.00** **ft**

Wind interior zone width = **4.25** **ft**

INTERIOR SHEAR WALLS: **SW-1**

EXTERIOR SHEAR WALLS: **SW-1**

Wall Overturning

w1 **Seismic controls overturning, 0.6D+0.7E**

Short wall segment (sws)=	36.50	ft
Wall height (h)=	16.08	ft
Roof Load Width (rlw)=	12.25	ft
Dead load Reduct (swred)=	0.60	
Allowable story drift = .02*h =	3.86	

Formula		Wind		Seismic	
Mot=Us*sws*h	**Mot=**	**29552**	**ft-lbs**	**24475**	**ft-lbs**
Hdl=wwt*h+Rdl*rlw	**Hdl=**	**547**	**plf**	**515**	**plf**
Mres=(swred *Hdl*sws^2)/2	**Mres=**	**218802**	**ft-lbs**	**205906**	**ft-lbs**
Hd-uplift=(Mot-Mres)/sws	**Hd-uplift=**	**-5185**	**lbs**	**-4971**	**lbs**

$\Delta_s = 8vh^3/(EAb) + vh/(Gt) + 0.75*h*e_n + h/b*d_a =$ **0.36** **OK**

h/w ratio OK for wind forces

h/w ratio OK for seismic forces

Below = **Wood framing**

NO HOLDOWNS REQUIRED

P1

LATERAL LOAD ANALYSIS FOR MILAN LOT 2

Roof length, L_R = **61.58** Ω = **1.0** ASCE 7-10 Table 12.2-1

υ_{RW} = W/L_R = **29.84** **(Wind)**

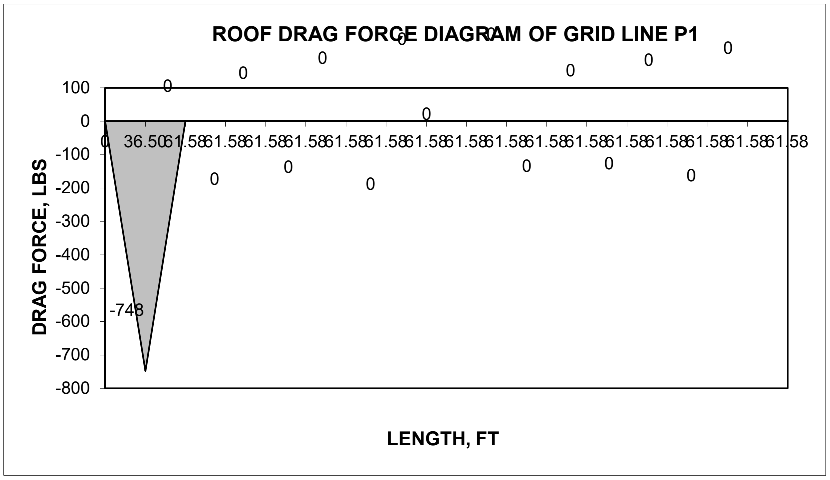
υ_W = P/Sw - υ_R = **-20.51** **(Wind)**

υ_{RE} = E/L_R = **24.72** **(Seismic)**

υ_W = P/Sw - υ_R = **-16.98** **(Seismic)**

DRAG FORCE CALCULATIONS

WALL/OPENING	LENGTH	Σ LENGTH	Wind DRAG, LBS	Seismic DRAG, LBS	E_m LEVEL
	0	0	**0**	**0**	**0**
W1	**36.50**	36.50	**-748**	**-620**	**-620**
OPENING	**25.08**	61.58	**0**	**0**	**0**
		61.58	**0**	**0**	**0**
		61.58	**0**	**0**	**0**
		61.58	**0**	**0**	**0**
		61.58	**0**	**0**	**0**
		61.58	**0**	**0**	**0**
		61.58	**0**	**0**	**0**
		61.58	**0**	**0**	**0**
		61.58	**0**	**0**	**0**
		61.58	**0**	**0**	**0**
		61.58	**0**	**0**	**0**
		61.58	**0**	**0**	**0**
		61.58	**0**	**0**	**0**
		61.58	**0**	**0**	**0**
		61.58	**0**	**0**	**0**
		61.58	**0**	**0**	**0**



LATERAL LOAD ANALYSIS FOR MILAN LOT 2

Grid Line P2

MAIN OR ALT. ROOF?	ALT.
LONGITUDINAL OR TRANSVERSE?	T
END ZONE OR INTERIOR?	I

Formula		**Results**	**Units**
P=WL/Vs*sls/2	**Wind Shear Load (P)=**	**3341**	**lbs**
Us=P/Sw	**Unit Shear (Us)=**	**102**	**plf**
P=WL/Vs*sls/2	**Seismic Shear Load (P)=**	**3632**	**lbs**
Us=P/Sw	**Unit Shear (Us)=**	**111**	**plf**

At Roof **Seismic governs shear wall design**

End Zone Wind Load (WL/Vs)=	210	plf
Interior Zone Wind Load (WL/Vs)=	137	plf
Seismic Load ,(WL/Vs) =	148	plf
Shear Load Span (sls)=	48.92	ft
Roof Dead Load (Rdl)=	25	psf
Wall Weight (wwt)=	15	psf
Length of Shear Wall (Sw)=	32.67	ft

Wind end zone width =	**0.00**	**ft**
Wind interior zone width =	**24.46**	**ft**

INTERIOR SHEAR WALLS: **SW-1**

EXTERIOR SHEAR WALLS: **SW-1**

Wall Overturning

w1 **Seismic controls overturning, 0.6D+0.7E**

Short wall segment (sws)=	32.67	ft
Wall height (h)=	16.08	ft
Roof Load Width (rlw)=	10.25	ft
Dead load Reduct (swred)=	0.60	
Allowable story drift = .02*h =	3.86	

Formula		**Wind**		**Seismic**	
Mot=Us*sws*h	**Mot=**	**53731**	**ft-lbs**	**58405**	**ft-lbs**
Hdl=wwt*h+Rdl*rlw	**Hdl=**	**497**	**plf**	**468**	**plf**
Mres=(swred *Hdl*sws^2)/2	**Mres=**	**159283**	**ft-lbs**	**149895**	**ft-lbs**
Hd-uplift=(Mot-Mres)/sws	**Hd-uplift=**	**-3231**	**lbs**	**-2800**	**lbs**

$\Delta_s = 8vh^3/(EAb) + vh/(Gt) + 0.75*h*e_n + h/b*d_a =$ **0.49** **OK**

h/w ratio OK for wind forces

h/w ratio OK for seismic forces

Below = **Wood framing**

NO HOLDOWNS REQUIRED

P2

LATERAL LOAD ANALYSIS FOR MILAN LOT 2

Roof length, L_R = **61.92** Ω = **1.0** ASCE 7-10 Table 12.2-1

υ_{RW} = W/L_R = **53.96** **(Wind)**

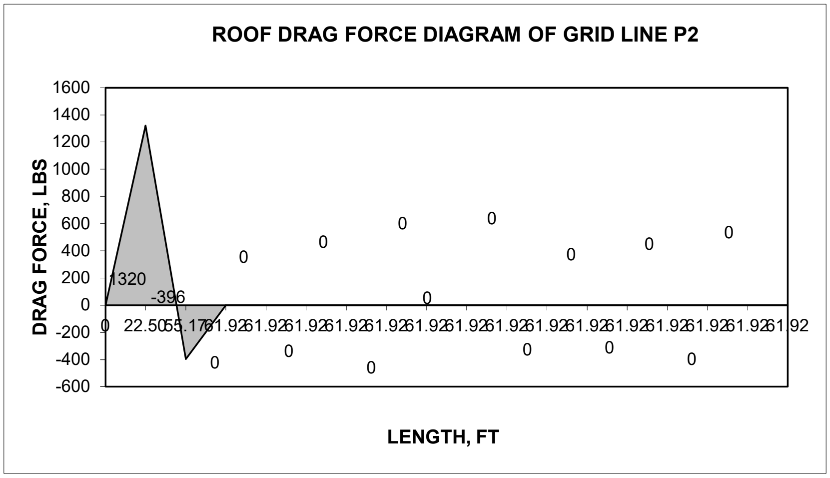
υ_W = P/Sw - υ_R = **-48.31** **(Wind)**

υ_{RE} = E/L_R = **58.66** **(Seismic)**

υ_W = P/Sw - υ_R = **-52.52** **(Seismic)**

DRAG FORCE CALCULATIONS

WALL/OPENING	LENGTH	Σ LENGTH	Wind DRAG, LBS	Seismic DRAG, LBS	E_m LEVEL
	0	0	**0**	**0**	**0**
OPENING	**22.50**	22.50	**1214**	**1320**	**1320**
W1	**32.67**	55.17	**-364**	**-396**	**-396**
OPENING	**6.75**	61.92	**0**	**0**	**0**
		61.92	**0**	**0**	**0**
		61.92	**0**	**0**	**0**
		61.92	**0**	**0**	**0**
		61.92	**0**	**0**	**0**
		61.92	**0**	**0**	**0**
		61.92	**0**	**0**	**0**
		61.92	**0**	**0**	**0**
		61.92	**0**	**0**	**0**
		61.92	**0**	**0**	**0**
		61.92	**0**	**0**	**0**
		61.92	**0**	**0**	**0**
		61.92	**0**	**0**	**0**
		61.92	**0**	**0**	**0**
		61.92	**0**	**0**	**0**



LATERAL LOAD ANALYSIS FOR MILAN LOT 2

Grid Line P3

MAIN OR ALT. ROOF?	ALT.
LONGITUDINAL OR TRANSVERSE?	T
END ZONE OR INTERIOR?	I

Formula		Results	Units
P=WL/Vs*sls/2	**Wind Shear Load (P)=**	**3341**	**lbs**
Us=P/Sw	**Unit Shear (Us)=**	**132**	**plf**
P=WL/Vs*sls/2	**Seismic Shear Load (P)=**	**3632**	**lbs**
Us=P/Sw	**Unit Shear (Us)=**	**143**	**plf**
	Wind end zone width =	**0.00**	**ft**
	Wind interior zone width =	**24.46**	**ft**
	INTERIOR SHEAR WALLS:	**SW-2**	
	EXTERIOR SHEAR WALLS:	**SW-2**	

At Roof **Seismic governs shear wall design**

End Zone Wind Load (WL/Vs)=	210	plf
Interior Zone Wind Load (WL/Vs)=	137	plf
Seismic Load ,(WL/Vs) =	148	plf
Shear Load Span (sls)=	48.92	ft
Roof Dead Load (Rdl)=	25	psf
Wall Weight (wwt)=	15	psf
Length of Shear Wall (Sw)=	25.34	ft

Wall Overturning

w1 **Seismic controls overturning, 0.6D+0.7E**

Short wall segment (sws)=	21.42	ft
2nd Story Wall height (h)=	9.08	ft
Roof Load Width (rlw)=	16.21	ft
Dead load Reduct (swred)=	0.60	
Allowable story drift = .02*h =	2.18	

Formula		Wind		Seismic	
Mot=Us*sws*h	**Mot=**	**25647**	**ft-lbs**	**27878**	**ft-lbs**
Hdl=wwt*h+Rdl*rlw	**Hdl=**	**541**	**plf**	**510**	**plf**
Mres=(swred *Hdl*sws^2)/2	**Mres=**	**74528**	**ft-lbs**	**70135**	**ft-lbs**
Hd-uplift=(Mot-Mres)/sws	**Hd-uplift=**	**-2282**	**lbs**	**-1973**	**lbs**

$\Delta_s = 8vh^3/(EAb) + vh/(Gt) + 0.75*h*e_n + h/b*d_a =$ **0.61** **OK**

h/w ratio OK for wind forces **h/w ratio OK for seismic forces**

Below = **Wood framing** ***NO HOLDOWNS REQUIRED***

w2 **Seismic controls overturning, 0.6D+0.7E**

Short wall segment (sws)=	3.92	ft
2nd Story Wall height (h)=	9.08	ft
Roof Load Width (rlw)=	4.00	ft
Dead load Reduct (swred)=	0.60	
Allowable story drift = .02*h =	2.18	

		Wind		Seismic	
Mot=Us*sws*h	**Mot=**	**4694**	**ft-lbs**	**5102**	**ft-lbs**
Hdl=wwt*h+Rdl*rlw	**Hdl=**	**236**	**plf**	**222**	**plf**
Mres=(swred *Hdl*sws^2)/2	**Mres=**	**1089**	**ft-lbs**	**1025**	**ft-lbs**
Hd-uplift=(Mot-Mres)/sws	**Hd-uplift=**	**920**	**lbs**	**1040**	**lbs**

$\Delta_s = 8vh^3/(EAb) + vh/(Gt) + 0.75*h*e_n + h/b*d_a =$ **1.62** **OK**

h/w ratio OK for wind forces ***2:1 < h/w ratio < 3.5:1, tabulated shear value mulipled by 2w/h, use SW-2***

Below = **Concrete** **Hold down location =** **Corner** **USE SIMPSON HOLDOWN: LSTHD8 OR HTT4**

P3

LATERAL LOAD ANALYSIS FOR MILAN LOT 2

Roof length, L_R = **76.42** Ω = **1.0** ASCE 7-10 Table 12.2-1

υ_{RW} = W/L_R = **43.72** **(Wind)**

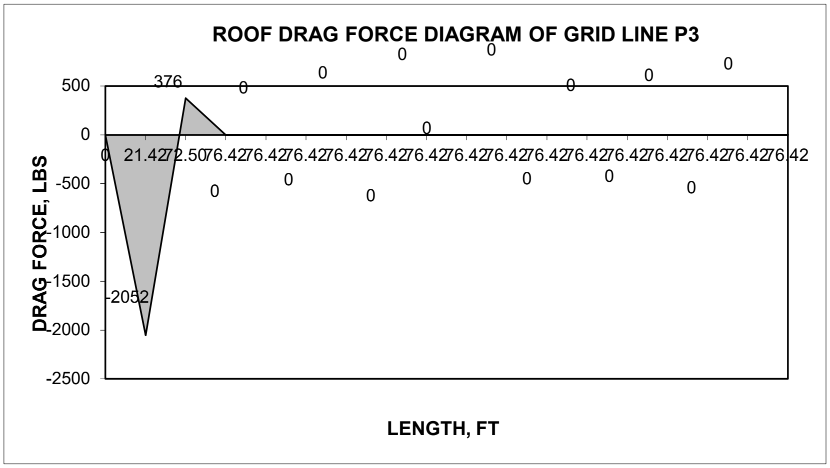
υ_W = P/Sw - υ_R = **-88.14** **(Wind)**

υ_{RE} = E/L_R = **47.53** **(Seismic)**

υ_W = P/Sw - υ_R = **-95.81** **(Seismic)**

DRAG FORCE CALCULATIONS

WALL/OPENING	LENGTH	Σ LENGTH	Wind DRAG, LBS	Seismic DRAG, LBS	E_m LEVEL
	0	0	0	0	0
W1	21.42	21.42	-1888	-2052	-2052
OPENING	51.08	72.50	346	376	376
W2	3.92	76.42	0	0	0
		76.42	0	0	0
		76.42	0	0	0
		76.42	0	0	0
		76.42	0	0	0
		76.42	0	0	0
		76.42	0	0	0
		76.42	0	0	0
		76.42	0	0	0
		76.42	0	0	0
		76.42	0	0	0
		76.42	0	0	0
		76.42	0	0	0
		76.42	0	0	0
		76.42	0	0	0



LATERAL LOAD ANALYSIS FOR MILAN LOT 2

Grid Line P4

MAIN OR ALT. ROOF?	MAIN
LONGITUDINAL OR TRANSVERSE?	T
END ZONE OR INTERIOR?	I

Formula		Results	Units
P=WL/Vs*sls/2	**Wind Shear Load (P)=**	**1946**	**lbs**
Us=P/Sw	**Unit Shear (Us)=**	**60**	**plf**
P=WL/Vs*sls/2	**Seismic Shear Load (P)=**	**3335**	**lbs**
Us=P/Sw	**Unit Shear (Us)=**	**103**	**plf**

At Roof **Seismic governs shear wall design**

End Zone Wind Load (WL/Vs)=	135	plf
Interior Zone Wind Load (WL/Vs)=	87	plf
Seismic Load ,(WL/Vs) =	148	plf
Shear Load Span (sls)=	44.92	ft
Roof Dead Load (Rdl)=	25	psf
Wall Weight (wwt)=	15	psf
Length of Shear Wall (Sw)=	32.33	ft

Wind end zone width = 0.00 ft

Wind interior zone width = 22.46 ft

INTERIOR SHEAR WALLS: SW-1

EXTERIOR SHEAR WALLS: SW-1

Wall Overturning

w1 **Seismic controls overturning, 0.6D+0.7E**

Short wall segment (sws)=	32.33	ft
2nd Story Wall height (h)=	9.08	ft
Roof Load Width (rlw)=	14.46	ft
Dead load Reduct (swred)=	0.60	
Allowable story drift = .02*h =	2.18	

Formula		Wind		Seismic	
Mot=Us*sws*h	**Mot=**	**17673**	**ft-lbs**	**30283**	**ft-lbs**
Hdl=wwt*h+Rdl*rlw	**Hdl=**	**498**	**plf**	**468**	**plf**
Mres=(swred *Hdl*sws^2)/2	**Mres=**	**156063**	**ft-lbs**	**146865**	**ft-lbs**
Hd-uplift=(Mot-Mres)/sws	**Hd-uplift=**	**-4281**	**lbs**	**-3606**	**lbs**

$\Delta_s = 8vh^3/(EAb) + vh/(Gt) + 0.75*h*e_n + h/b*d_a =$ **0.26** **OK**

h/w ratio OK for wind forces

h/w ratio OK for seismic forces

Below = **Wood framing**

NO HOLDOWNS REQUIRED

LATERAL LOAD ANALYSIS FOR MILAN LOT 2

Grid Line P4

MAIN OR ALT. ROOF?	MAIN
LONGITUDINAL OR TRANSVERSE?	T
END ZONE OR INTERIOR?	I

Formula		Results	Units
P=WL/Vs*sls/2	**Wind Shear Load (P)=**	**5664**	**lbs**
Us=P/Sw	**Unit Shear (Us)=**	**111**	**plf**
P=WL/Vs*sls/2	**Seismic Shear Load (P)=**	**6428**	**lbs**
Us=P/Sw	**Unit Shear (Us)=**	**125**	**plf**
	Wind end zone width =	**0.00**	**ft**
	Wind interior zone width =	**22.46**	**ft**
	INTERIOR SHEAR WALLS:	**SW-1**	
`	**EXTERIOR SHEAR WALLS:**	**SW-1**	

At Floor **Wind governs shear wall design**

End Zone Wind Load (WL/Vs)=	248	plf
Interior Zone Wind Load (WL/Vs)=	166	plf
Seismic Load ,(WL/Vs) =	138	plf
Shear Load Span (sls)=	44.92	ft
Roof Dead Load (Rdl)=	25	psf
Floor Dead Load (Fdl)=	15	psf
Wall Weight (wwt)=	15	psf
Length of Shear Wall (Sw)=	51.25	ft

Wall Overturning

Perforated Wall (SDPWS 2008 Table 4.3.3.5)

C_o = 0.854

Required Shear wall = ***SW-2***

Sill plate uplift anchorage: **SW-1**

Perforated wall Length (sws)=		33.33	ft	
Full ht segment lengths =	15.75	12.83		
% Full Height sheathing =		86%		
Max Opening Ht =		6.67	ft	
2nd Story Wall height (h)=		9.08	ft	
1st Story Wall height (h)=		9.08	ft	
Roof Load Width (rlw)=		12.00	ft	
Floor Load Width (flw)=		12.00	ft	
Dead load Reduct (swred)=		0.60		
Allowable story drift = .02*h =		2.18		

		Wind		Seismic	
Mot=Us*sws*h	**Mot=**	**33445**	**ft-lbs**	**37961**	**ft-lbs**
DL=(wwt*(hf+hs))+(rlw*Rdl)+(flw*Fdl)	**DL=**	**752**	**plf**	**708**	**plf**
Mres=(swred *DL*sws^2)/2	**Mres=**	**250750**	**ft-lbs**	**235971**	**ft-lbs**
T/C =V*h/(Co*ΣL)	**T/C =**	**1371**	**lbs**	**1556**	**lbs**
Hd-uplift=(Mot-Mres)/sws	**Hd-uplift=**	**-6152**	**lbs**	**-5524**	**lbs**
	Uplift from wall above =		**lbs**		**lbs**
	Total HD Uplift =	**-6152**	**lbs**	**-5524**	**lbs**

$\Delta_s = 8vh^3/(EAb) + vh/(Gt) + 0.75*h*e_n + h/b*d_a =$ **0.46** **OK**

h/w ratio OK for wind forces **h/w ratio OK for seismic forces**

Below = **Concrete** ***NO HOLDOWNS REQUIRED***

w1 **Seismic controls overturning, 0.6D+0.7E**

Short wall segment (sws)=	17.92	ft
2nd Story Wall height (h)=	0.00	ft
1st Story Wall height (h)=	9.08	ft
Roof Load Width (rlw)=	18.00	ft
Floor Load Width (flw)=		ft
Dead load Reduct (swred)=	0.60	
Allowable story drift = .02*h =	2.18	

Formula		Wind		Seismic	
Mot=Us*sws*h	**Mot=**	**17982**	**ft-lbs**	**20410**	**ft-lbs**
DL=(wwt*(hf+hs))+(rlw*Rdl)+(flw*Fdl)	**DL=**	**586**	**plf**	**552**	**plf**
Mres=(swred *DL*sws^2)/2	**Mres=**	**56473**	**ft-lbs**	**53145**	**ft-lbs**
Hd-uplift=(Mot-Mres)/sws	**Hd-uplift=**	**-2148**	**lbs**	**-1827**	**lbs**
	Uplift from wall above =		**lbs**		**lbs**
	Total HD Uplift =	**-2148**	**lbs**	**-1827**	**lbs**

$\Delta_s = 8vh^3/(EAb) + vh/(Gt) + 0.75*h*e_n + h/b*d_a =$ **0.39** **OK**

h/w ratio OK for wind forces **h/w ratio OK for seismic forces**

Below = **Concrete** ***NO HOLDOWNS REQUIRED***

P4

LATERAL LOAD ANALYSIS FOR MILAN LOT 2

Roof length, $L_R =$ **32.33** $\Omega =$ **1.0** ASCE 7-10 Table 12.2-1

$\upsilon_{RW} = W/L_R =$ **60.20** **(Wind)**

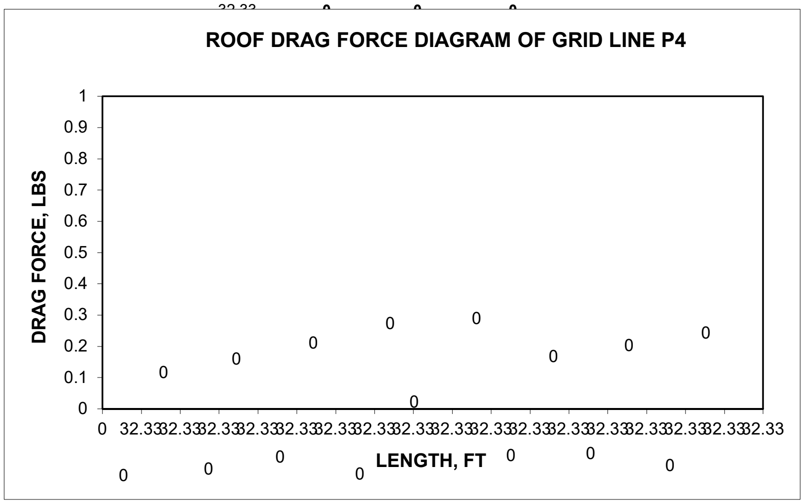
$\upsilon_W = P/Sw - \upsilon_R =$ **0.00** **(Wind)**

$\upsilon_{RE} = E/L_R =$ **103.16** **(Seismic)**

$\upsilon_W = P/Sw - \upsilon_R =$ **0.00** **(Seismic)**

DRAG FORCE CALCULATIONS

WALL/OPENING	LENGTH	Σ LENGTH	Wind DRAG, LBS	Seismic DRAG, LBS	E_m LEVEL
	0	0	**0**	**0**	**0**
W1	**32.33**	32.33	**0**	**0**	**0**
		32.33	**0**	**0**	**0**
		32.33	**0**	**0**	**0**
		32.33	**0**	**0**	**0**
		32.33	**0**	**0**	**0**
		32.33	**0**	**0**	**0**
		32.33	**0**	**0**	**0**
		32.33	**0**	**0**	**0**
		32.33	**0**	**0**	**0**
		32.33	**0**	**0**	**0**
		32.33	**0**	**0**	**0**
		32.33	**0**	**0**	**0**
		32.33	**0**	**0**	**0**
		32.33	**0**	**0**	**0**
		32.33	**0**	**0**	**0**
		32.33	**0**	**0**	**0**



LATERAL LOAD ANALYSIS FOR MILAN LOT 2

Floor length, L_F = **75.00**

Ω = **1.0** ASCE 7-10 Table 12.2-1

υ_{RW} = W/L_F = **49.56** **(Wind)**

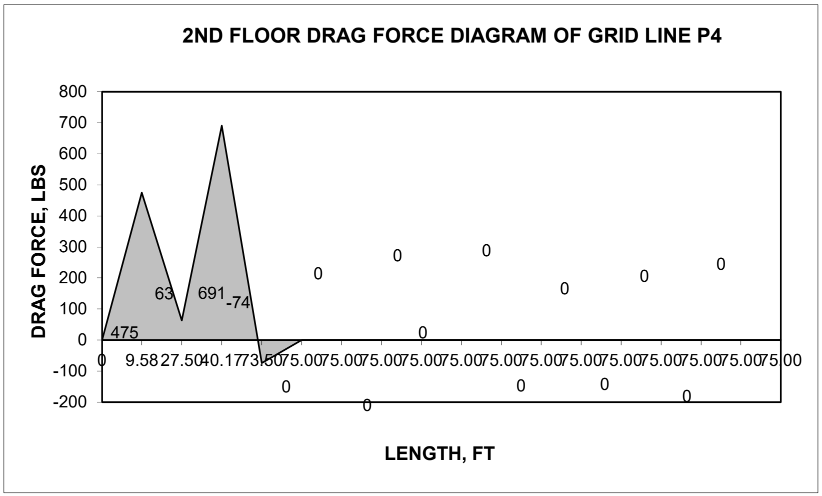
υ_W = P/Sw - υ_R = **-22.97** **(Wind)**

υ_{RE} = E/L_F = **41.24** **(Seismic)**

υ_W = P/Sw - υ_R = **-19.11** **(Seismic)**

DRAG FORCE CALCULATIONS

			Wind	Seismic	
WALL/OPENING	LENGTH	Σ LENGTH	DRAG, LBS	DRAG, LBS	E_m LEVEL
	0	0	**0**	**0**	**0**
OPENING	**9.58**	9.58	**475**	**395**	**395**
W1	**17.92**	27.50	**63**	**53**	**53**
OPENING	**12.67**	40.17	**691**	**575**	**575**
W2	**33.33**	73.50	**-74**	**-62**	**-62**
OPENING	**1.50**	75.00	**0**	**0**	**0**
		75.00	**0**	**0**	**0**
		75.00	**0**	**0**	**0**
		75.00	**0**	**0**	**0**
		75.00	**0**	**0**	**0**
		75.00	**0**	**0**	**0**
		75.00	**0**	**0**	**0**
		75.00	**0**	**0**	**0**
		75.00	**0**	**0**	**0**
		75.00	**0**	**0**	**0**
		75.00	**0**	**0**	**0**
		75.00	**0**	**0**	**0**
		75.00	**0**	**0**	**0**



LATERAL LOAD ANALYSIS FOR MILAN LOT 2

Grid Line P5

MAIN OR ALT. ROOF?	MAIN
LONGITUDINAL OR TRANSVERSE?	T
END ZONE OR INTERIOR?	E

Formula		**Results**	**Units**
P=WL/Vs*sls/2	**Wind Shear Load (P)=**	**1373**	**lbs**
Us=P/Sw	**Unit Shear (Us)=**	**83**	**plf**
P=WL/Vs*sls/2	**Seismic Shear Load (P)=**	**1850**	**lbs**
Us=P/Sw	**Unit Shear (Us)=**	**112**	**plf**

At Roof **Wind governs shear wall design**

End Zone Wind Load (WL/Vs)=	135	plf
Interior Zone Wind Load (WL/Vs)=	87	plf
Seismic Load ,(WL/Vs) =	148	plf
Shear Load Span (sls)=	24.92	ft
Roof Dead Load (Rdl)=	25	psf
Wall Weight (wwt)=	15	psf
Length of Shear Wall (Sw)=	16.50	ft

Wind end zone width =	**6.00**	**ft**
Wind interior zone width =	**6.46**	**ft**

INTERIOR SHEAR WALLS: SW-1

EXTERIOR SHEAR WALLS: SW-1

Wall Overturning

w1 **Seismic controls overturning, 0.6D+0.7E**

Short wall segment (sws)=	16.50	ft
2nd Story Wall height (h)=	9.08	ft
Roof Load Width (rlw)=	14.46	ft
Dead load Reduct (swred)=	0.60	
Allowable story drift = .02*h =	2.18	

Formula		**Wind**		**Seismic**	
Mot=Us*sws*h	**Mot=**	**12463**	**ft-lbs**	**16800**	**ft-lbs**
Hdl=wwt*h+Rdl*rlw	**Hdl=**	**498**	**plf**	**468**	**plf**
Mres=(swred *Hdl*sws^2)/2	**Mres=**	**40650**	**ft-lbs**	**38254**	**ft-lbs**
Hd-uplift=(Mot-Mres)/sws	**Hd-uplift=**	**-1708**	**lbs**	**-1300**	**lbs**

$\Delta_s = 8vh^3/(EAb) + vh/(Gt) + 0.75*h*e_n + h/b*d_a =$ **0.40** **OK**

h/w ratio OK for wind forces

h/w ratio OK for seismic forces

Below = Wood framing

NO HOLDOWNS REQUIRED

P5

LATERAL LOAD ANALYSIS FOR MILAN LOT 2

Grid Line P5

MAIN OR ALT. ROOF?	MAIN
LONGITUDINAL OR TRANSVERSE?	T
END ZONE OR INTERIOR?	E

Formula		Results	Units
P=WL/Vs*sls/2	**Wind Shear Load (P)=**	**3929**	**lbs**
Us=P/Sw	**Unit Shear (Us)=**	**99**	**plf**
P=WL/Vs*sls/2	**Seismic Shear Load (P)=**	**3566**	**lbs**
Us=P/Sw	**Unit Shear (Us)=**	**90**	**plf**

At Floor **Wind governs shear wall design**

End Zone Wind Load (WL/Vs)=	248	plf
Interior Zone Wind Load (WL/Vs)=	166	plf
Seismic Load ,(WL/Vs) =	138	plf
Shear Load Span (sls)=	24.92	ft
Roof Dead Load (Rdl)=	25	psf
Floor Dead Load (Fdl)=	15	psf
Wall Weight (wwt)=	15	psf
Length of Shear Wall (Sw)=	39.59	ft

Wind end zone width =	**6.00**	**ft**
Wind interior zone width =	**6.46**	**ft**

INTERIOR SHEAR WALLS: **SW-1**

EXTERIOR SHEAR WALLS: **SW-1**

Wall Overturning

w1 **Seismic controls overturning, 0.6D+0.7E**

Short wall segment (sws)=	31.67	ft
2nd Story Wall height (h)=	0.00	ft
1st Story Wall height (h)=	9.08	ft
Roof Load Width (rlw)=	18.00	ft
Floor Load Width (flw)=	0.00	ft
Dead load Reduct (swred)=	0.60	
Allowable story drift = .02*h =	2.18	

Formula		Wind		Seismic	
Mot=Us*sws*h	**Mot=**	**28535**	**ft-lbs**	**25904**	**ft-lbs**
DL=(wwt*(hf+hs))+(rlw*Rdl)+(flw*Fdl)	**DL=**	**586**	**plf**	**552**	**plf**
Mres=(swred *DL*sws^2)/2	**Mres=**	**176386**	**ft-lbs**	**165989**	**ft-lbs**
Hd-uplift=(Mot-Mres)/sws	**Hd-uplift=**	**-4668**	**lbs**	**-4423**	**lbs**
	Uplift from wall above =		**lbs**		**lbs**
	Total HD Uplift =	**-4668**	**lbs**	**-4423**	**lbs**
	$\Delta_s = 8vh^3/(EAb) + vh/(Gt) + 0.75*h*e_n + h/b*d_a$ =	**0.25**	**OK**		

h/w ratio OK for wind forces

Below = **Concrete**

h/w ratio OK for seismic forces

NO HOLDOWNS REQUIRED

W2 **Seismic controls overturning, 0.6D+0.7E**

Short wall segment (sws)=	7.92	ft
2nd Story Wall height (h)=	9.08	ft
1st Story Wall height (h)=	9.08	ft
Roof Load Width (rlw)=	14.00	ft
Floor Load Width (flw)=	12.00	ft
Dead load Reduct (swred)=	0.60	
Allowable story drift = .02*h =	2.18	

Formula		Wind		Seismic	
Mot=Us*sws*h	**Mot=**	**7136**	**ft-lbs**	**6478**	**ft-lbs**
DL=(wwt*(hf+hs))+(rlw*Rdl)+(flw*Fdl)	**DL=**	**802**	**plf**	**755**	**plf**
Mres=(swred *DL*sws^2)/2	**Mres=**	**15099**	**ft-lbs**	**14210**	**ft-lbs**
Hd-uplift=(Mot-Mres)/sws	**Hd-uplift=**	**-1005**	**lbs**	**-976**	**lbs**
	Uplift from wall above =		**lbs**		**lbs**
	Total HD Uplift =	**-1005**	**lbs**	**-976**	**lbs**
	$\Delta_s = 8vh^3/(EAb) + vh/(Gt) + 0.75*h*e_n + h/b*d_a$ =	**0.70**	**OK**		

h/w ratio OK for wind forces

Below = **Concrete**

h/w ratio OK for seismic forces

NO HOLDOWNS REQUIRED

LATERAL LOAD ANALYSIS FOR MILAN LOT 2

Roof length, L_R = **32.83** Ω = **1.0** ASCE 7-10 Table 12.2-1

υ_{RW} = W/L_R = **41.81** **(Wind)**

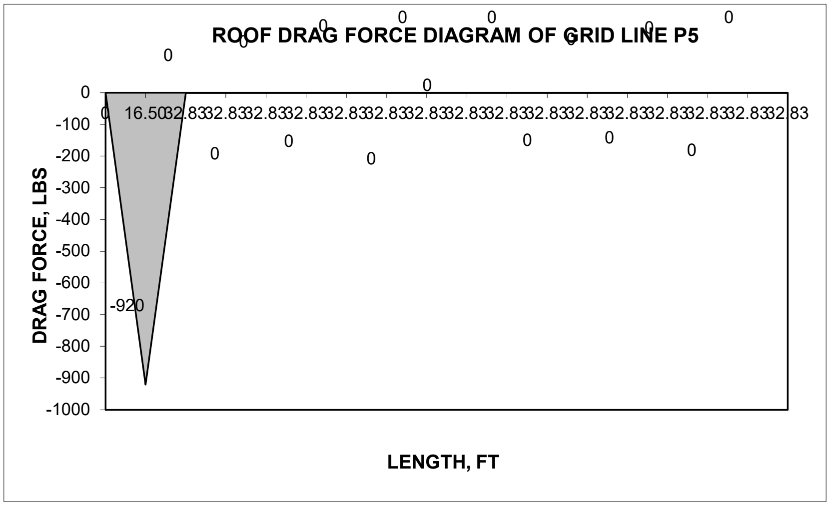
υ_{W} = P/Sw - υ_R = **-41.38** **(Wind)**

υ_{RE} = E/L_R = **56.36** **(Seismic)**

υ_{W} = P/Sw - υ_R = **-55.78** **(Seismic)**

DRAG FORCE CALCULATIONS

WALL/OPENING	LENGTH	Σ LENGTH	Wind DRAG, LBS	Seismic DRAG, LBS	E_m LEVEL
	0	0	**0**	**0**	**0**
W1	**16.50**	16.50	**-683**	**-920**	**-920**
OPENING	**16.33**	32.83	**0**	**0**	**0**
		32.83	**0**	**0**	**0**
		32.83	**0**	**0**	**0**
		32.83	**0**	**0**	**0**
		32.83	**0**	**0**	**0**
		32.83	**0**	**0**	**0**
		32.83	**0**	**0**	**0**
		32.83	**0**	**0**	**0**
		32.83	**0**	**0**	**0**
		32.83	**0**	**0**	**0**
		32.83	**0**	**0**	**0**
		32.83	**0**	**0**	**0**
		32.83	**0**	**0**	**0**
		32.83	**0**	**0**	**0**
		32.83	**0**	**0**	**0**
		32.83	**0**	**0**	**0**



LATERAL LOAD ANALYSIS FOR MILAN LOT 2

Floor length, L_F = **72.83**

Ω = **1.0** ASCE 7-10 Table 12.2-1

υ_{RW} = W/L_F = **35.10** **(Wind)**

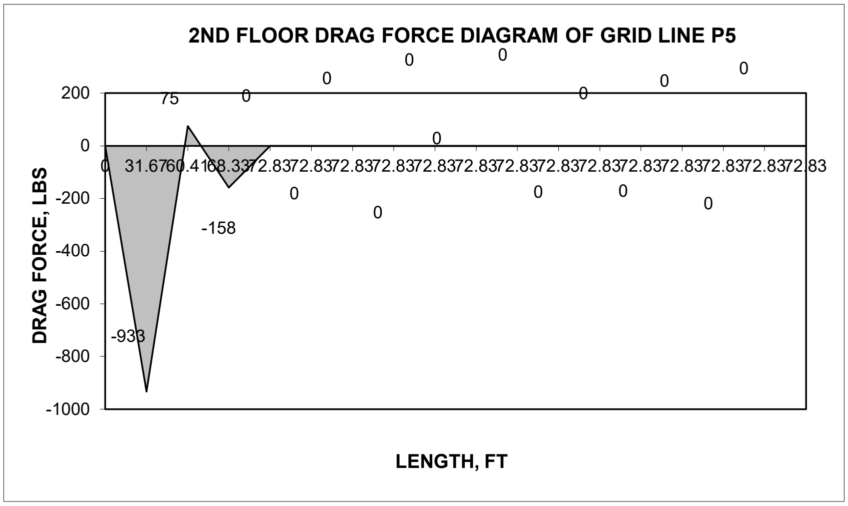
υ_W = P/Sw - υ_R = **-29.47** **(Wind)**

υ_{RE} = E/L_F = **23.56** **(Seismic)**

υ_W = P/Sw - υ_R = **-19.78** **(Seismic)**

DRAG FORCE CALCULATIONS

WALL/OPENING	LENGTH	Σ LENGTH	Wind DRAG, LBS	Seismic DRAG, LBS	E_m LEVEL
	0	0	**0**	**0**	**0**
W1	**31.67**	31.67	**-933**	**-627**	**-627**
OPENING	**28.74**	60.41	**75**	**51**	**51**
W2	**7.92**	68.33	**-158**	**-106**	**-106**
OPENING	**4.50**	72.83	**0**	**0**	**0**
		72.83	**0**	**0**	**0**
		72.83	**0**	**0**	**0**
		72.83	**0**	**0**	**0**
		72.83	**0**	**0**	**0**
		72.83	**0**	**0**	**0**
		72.83	**0**	**0**	**0**
		72.83	**0**	**0**	**0**
		72.83	**0**	**0**	**0**
		72.83	**0**	**0**	**0**
		72.83	**0**	**0**	**0**
		72.83	**0**	**0**	**0**
		72.83	**0**	**0**	**0**
		72.83	**0**	**0**	**0**



LATERAL LOAD ANALYSIS FOR MILAN LOT 2

Grid Line P6

MAIN OR ALT. ROOF?	MAIN
LONGITUDINAL OR TRANSVERSE?	L
END ZONE OR INTERIOR?	E

Formula		Results	Units
P=WL/Vs*sls/2	**Wind Shear Load (P)=**	**1792**	**lbs**
Us=P/Sw	**Unit Shear (Us)=**	**138**	**plf**
P=WL/Vs*sls/2	**Seismic Shear Load (P)=**	**1606**	**lbs**
Us=P/Sw	**Unit Shear (Us)=**	**124**	**plf**

At Roof **Wind governs shear wall design**

End Zone Wind Load (WL/Vs)=	135	plf
Interior Zone Wind Load (WL/Vs)=	87	plf
Seismic Load ,(WL/Vs) =	98	plf
Shear Load Span (sls)=	32.83	ft
Roof Dead Load (Rdl)=	25	psf
Wall Weight (wwt)=	15	psf
Length of Shear Wall (Sw)=	13.00	ft

Wind end zone width =	**7.57**	**ft**
Wind interior zone width =	**8.85**	**ft**

INTERIOR SHEAR WALLS: SW-1
EXTERIOR SHEAR WALLS: SW-1

Wall Overturning

w1 **Wind controls overturning, 0.6D+0.6W**

Short wall segment (sws)=	6.50	ft
2nd Story Wall height (h)=	9.08	ft
Roof Load Width (rlw)=	6.00	ft
Dead load Reduct (swred)=	0.60	
Allowable story drift = .02*h =	2.18	

Formula		Wind		Seismic	
Mot=Us*sws*h	**Mot=**	**8135**	**ft-lbs**	**7290**	**ft-lbs**
Hdl=wwt*h+Rdl*rlw	**Hdl=**	**286**	**plf**	**269**	**plf**
Mres=(swred *Hdl*sws^2)/2	**Mres=**	**3628**	**ft-lbs**	**3414**	**ft-lbs**
Hd-uplift=(Mot-Mres)/sws	**Hd-uplift=**	**693**	**lbs**	**596**	**lbs**

$\Delta_s = 8vh^3/(EAb) + vh/(Gt) + 0.75*h*e_n + h/b*d_a =$ **0.86** **OK**

h/w ratio OK for wind forces

h/w ratio OK for seismic forces

Below = **Wood framing**

USE SIMPSON HOLDOWN: CS16

w2 **Wind controls overturning, 0.6D+0.6W**

Short wall segment (sws)=	6.50	ft
2nd Story Wall height (h)=	9.08	ft
Roof Load Width (rlw)=	6.00	ft
Dead load Reduct (swred)=	0.60	
Allowable story drift = .02*h =	2.18	

		Wind		Seismic	
Mot=Us*sws*h	**Mot=**	**8135**	**ft-lbs**	**7290**	**ft-lbs**
Hdl=wwt*h+Rdl*rlw	**Hdl=**	**286**	**plf**	**269**	**plf**
Mres=(swred *Hdl*sws^2)/2	**Mres=**	**3628**	**ft-lbs**	**3414**	**ft-lbs**
Hd-uplift=(Mot-Mres)/sws	**Hd-uplift=**	**693**	**lbs**	**596**	**lbs**

$\Delta_s = 8vh^3/(EAb) + vh/(Gt) + 0.75*h*e_n + h/b*d_a =$ **0.86** **OK**

h/w ratio OK for wind forces

h/w ratio OK for seismic forces

Below = **Wood framing**

USE SIMPSON HOLDOWN: CS16

LATERAL LOAD ANALYSIS FOR MILAN LOT 2

Grid Line P6

MAIN OR ALT. ROOF?	MAIN
LONGITUDINAL OR TRANSVERSE?	L
END ZONE OR INTERIOR?	E

Formula		**Results**	**Units**
P=WL/Vs*sls/2	**Wind Shear Load (P)=**	**3710**	**lbs**
Us=P/Sw	**Unit Shear (Us)=**	**172**	**plf**
P=WL/Vs*sls/2	**Seismic Shear Load (P)=**	**2994**	**lbs**
Us=P/Sw	**Unit Shear (Us)=**	**139**	**plf**
	Wind end zone width =	**7.57**	**ft**
	Wind interior zone width =	**0.26**	**ft**

INTERIOR SHEAR WALLS: SW-2
EXTERIOR SHEAR WALLS: SW-2

At Floor **Wind governs shear wall design**

End Zone Wind Load (WL/Vs)=	248	plf
Interior Zone Wind Load (WL/Vs)=	166	plf
Seismic Load ,(WL/Vs) =	177	plf
Shear Load Span (sls)=	15.66	ft
Roof Dead Load (Rdl)=	25	psf
Floor Dead Load (Fdl)=	15	psf
Wall Weight (wwt)=	15	psf
Length of Shear Wall (Sw)=	21.59	ft

Wall Overturning

Force Transfer around openings

Perforated wall Length (sws)=			10.00	ft
Vert. Wall strips =	2.00	2.00		ft
Horz. Wall strips =		3.00	1.08	ft
Opening Widths =			6.00	ft
Max opening height =			5.00	ft
2nd Story Wall height (h)=			9.08	ft
1st Story Wall height (h)=			9.08	ft
Roof Load Width (rlw)=				ft
Floor Load Width (flw)=				ft
Dead load Reduct (swred)=			0.60	
Allowable story drift = .02*h =			2.18	

Required Shear wall = ***SW-4***

		Wind		**Seismic**	
Mot=Us*sws*h	**Mot=**	**15604**	**ft-lbs**	**12593**	**ft-lbs**
Horizontal shear at sides of openings =		**430**	**plf**	**347**	**plf**
Horizontal T/C force at openings =		**1147**	**lbs**	**926**	**lbs**
Vertical shear above and below opening =		**382**	**plf**	**309**	**plf**
Vertical T/C force at openings =		**1074**	**lbs**	**867**	**lbs**
DL=(wwt*(hf+hs))+(rlw*Rdl)+(flw*Fdl)	**DL=**	**272**	**plf**	**256**	**plf**
Mres=(swred *DL*sws^2)/2	**Mres=**	**8172**	**ft-lbs**	**7690**	**ft-lbs**
Hd-uplift=(Mot-Mres)/sws	**Hd-uplift=**	**743**	**lbs**	**490**	**lbs**
	Uplift from wall above =	**2085**	**lbs**	**1855**	**lbs**
	Total HD Uplift =	**2828**	**lbs**	**2345**	**lbs**
Δ_s = 8vh^3/(EAb) + vh/(Gt) + 0.75*h*e$_n$ + h/b*d$_a$ =		**0.82**	**OK**		

h/w ratio OK for wind forces

2:1 < h/w ratio < 3.5:1, design shear force mulipled by h/(2w)

Below = **Concrete** **Hold down location =** **Corner** **USE SIMPSON HOLDOWN: STHD10 OR HTT4**

w1 **Wind controls overturning, 0.6D+0.6W**

Short wall segment (sws)=	4.42	ft
2nd Story Wall height (h)=	0.00	ft
1st Story Wall height (h)=	9.08	ft
Roof Load Width (rlw)=	6.00	ft
Floor Load Width (flw)=		ft
Dead load Reduct (swred)=	0.60	
Allowable story drift = .02*h =	2.18	

Formula		**Wind**		**Seismic**	
Mot=Us*sws*h	**Mot=**	**6897**	**ft-lbs**	**5566**	**ft-lbs**
DL=(wwt*(hf+hs))+(rlw*Rdl)+(flw*Fdl)	**DL=**	**286**	**plf**	**269**	**plf**
Mres=(swred *DL*sws^2)/2	**Mres=**	**1677**	**ft-lbs**	**1579**	**ft-lbs**
Hd-uplift=(Mot-Mres)/sws	**Hd-uplift=**	**1181**	**lbs**	**902**	**lbs**
	Uplift from wall above =		**lbs**		**lbs**
	Total HD Uplift =	**1181**	**lbs**	**902**	**lbs**
Δ_s = 8vh^3/(EAb) + vh/(Gt) + 0.75*h*e$_n$ + h/b*d$_a$ =		**1.48**	**OK**		

h/w ratio OK for wind forces

2:1 < h/w ratio < 3.5:1, tabulated shear value mulipled by 2w/h, use SW-2

Below = **Concrete** **Hold down location =** **Corner** **USE SIMPSON HOLDOWN: LSTHD8 OR HTT4**

w2 **Wind controls overturning, 0.6D+0.6W**

Short wall segment (sws)=	7.17	ft
2nd Story Wall height (h)=	0.00	ft
1st Story Wall height (h)=	9.08	ft
Roof Load Width (rlw)=	6.00	ft
Floor Load Width (flw)=		ft
Dead load Reduct (swred)=	0.60	
Allowable story drift = .02*h =	2.18	

		Wind		**Seismic**	
Mot=Us*sws*h	**Mot=**	**11188**	**ft-lbs**	**9029**	**ft-lbs**
DL=(wwt*(hf+hs))+(rlw*Rdl)+(flw*Fdl)	**DL=**	**286**	**plf**	**269**	**plf**
Mres=(swred *DL*sws^2)/2	**Mres=**	**4414**	**ft-lbs**	**4154**	**ft-lbs**
Hd-uplift=(Mot-Mres)/sws	**Hd-uplift=**	**945**	**lbs**	**680**	**lbs**
	Uplift from wall above =		**lbs**		**lbs**
	Total HD Uplift =	**945**	**lbs**	**680**	**lbs**
Δ_s = 8vh^3/(EAb) + vh/(Gt) + 0.75*h*e$_n$ + h/b*d$_a$ =		**1.06**	**OK**		

h/w ratio OK for wind forces **h/w ratio OK for seismic forces**

Below = **Concrete** **Hold down location =** **Corner** **USE SIMPSON HOLDOWN: LSTHD8 OR HTT4**

P6

LATERAL LOAD ANALYSIS FOR MILAN LOT 2

Roof length, L_R = **24.92** Ω = **1.0** ASCE 7-10 Table 12.2-1

υ_{RW} = W/L_R = **71.90** **(Wind)**

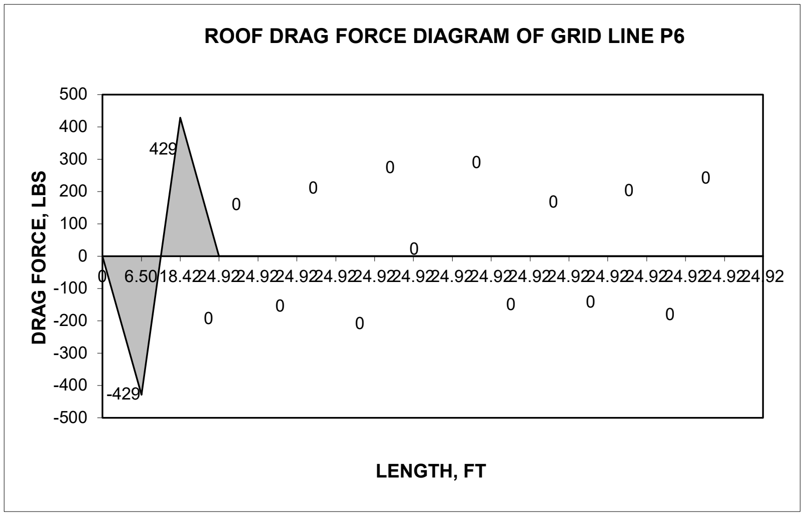
υ_W = P/Sw - υ_R = **-65.93** **(Wind)**

υ_{RE} = E/L_R = **64.43** **(Seismic)**

υ_W = P/Sw - υ_R = **-59.08** **(Seismic)**

DRAG FORCE CALCULATIONS

WALL/OPENING	LENGTH	Σ LENGTH	Wind DRAG, LBS	Seismic DRAG, LBS	E_m LEVEL
	0	0	**0**	**0**	**0**
W1	**6.50**	6.50	**-429**	**-384**	**-384**
OPENING	**11.92**	18.42	**429**	**384**	**384**
W2	**6.50**	24.92	**0**	**0**	**0**
		24.92	**0**	**0**	**0**
		24.92	**0**	**0**	**0**
		24.92	**0**	**0**	**0**
		24.92	**0**	**0**	**0**
		24.92	**0**	**0**	**0**
		24.92	**0**	**0**	**0**
		24.92	**0**	**0**	**0**
		24.92	**0**	**0**	**0**
		24.92	**0**	**0**	**0**
		24.92	**0**	**0**	**0**
		24.92	**0**	**0**	**0**
		24.92	**0**	**0**	**0**
		24.92	**0**	**0**	**0**
		24.92	**0**	**0**	**0**



P6

LATERAL LOAD ANALYSIS FOR MILAN LOT 2

Floor length, L_F = **93.83**

Ω = **1.0** ASCE 7-10 Table 12.2-1

υ_{RW} = W/L_F = **20.45** **(Wind)**

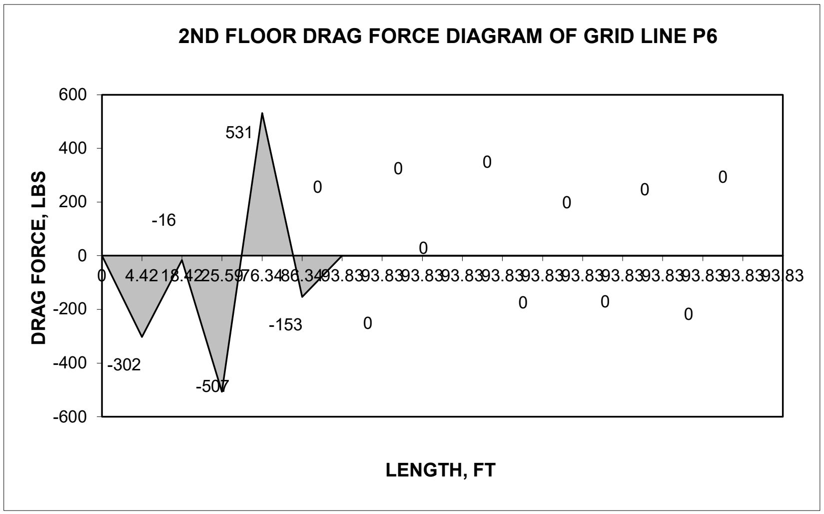
υ_W = P/Sw - υ_R = **-68.42** **(Wind)**

υ_{RE} = E/L_F = **14.80** **(Seismic)**

υ_W = P/Sw - υ_R = **-49.52** **(Seismic)**

DRAG FORCE CALCULATIONS

WALL/OPENING	LENGTH	Σ LENGTH	Wind DRAG, LBS	Seismic DRAG, LBS	E_m LEVEL
	0	0	**0**	**0**	**0**
W1	**4.42**	4.42	**-302**	**-219**	**-219**
OPENING	**14.00**	18.42	**-16**	**-12**	**-12**
W2	**7.17**	25.59	**-507**	**-367**	**-367**
OPENING	**50.75**	76.34	**531**	**384**	**384**
W1	**10.00**	86.34	**-153**	**-111**	**-111**
OPENING	**7.49**	93.83	**0**	**0**	**0**
		93.83	**0**	**0**	**0**
		93.83	**0**	**0**	**0**
		93.83	**0**	**0**	**0**
		93.83	**0**	**0**	**0**
		93.83	**0**	**0**	**0**
		93.83	**0**	**0**	**0**
		93.83	**0**	**0**	**0**
		93.83	**0**	**0**	**0**
		93.83	**0**	**0**	**0**
		93.83	**0**	**0**	**0**
		93.83	**0**	**0**	**0**



LATERAL LOAD ANALYSIS FOR MILAN LOT 2
Grid Line P7

MAIN OR ALT. ROOF?	MAIN
LONGITUDINAL OR TRANSVERSE?	L
END ZONE OR INTERIOR?	I

Formula		Results	Units
P=WL/Vs*sls/2	**Wind Shear Load (P)=**	**2807**	**lbs**
Us=P/Sw	**Unit Shear (Us)=**	**140**	**plf**
P=WL/Vs*sls/2	**Seismic Shear Load (P)=**	**3008**	**lbs**
Us=P/Sw	**Unit Shear (Us)=**	**150**	**plf**

At Floor **Wind governs shear wall design**

End Zone Wind Load (WL/Vs)=	248	plf
Interior Zone Wind Load (WL/Vs)=	166	plf
Seismic Load ,(WL/Vs) =	177	plf
Shear Load Span (sls)=	33.92	ft
Roof Dead Load (Rdl)=	25	psf
Floor Dead Load (Fdl)=	15	psf
Wall Weight (wwt)=	15	psf
Length of Shear Wall (Sw)=	20.00	ft

Wind end zone width =	**0.00**	**ft**
Wind interior zone width =	**16.96**	**ft**

INTERIOR SHEAR WALLS: SW-2
EXTERIOR SHEAR WALLS: SW-2

Wall Overturning

w1	**Seismic controls overturning, 0.6D+0.7E**		**Formula**		**Wind**		**Seismic**	
Short wall segment (sws)=	12.00	ft	Mot=Us*sws*h	**Mot=**	**27083**	**ft-lbs**	**29017**	**ft-lbs**
2nd Story Wall height (h)=	0.00	ft	DL=(wwt*(hf+hs))+(rlw*Rdl)+(flw*Fdl)	**DL=**	**391**	**plf**	**368**	**plf**
1st Story Wall height (h)=	16.08	ft	Mres=(swred *DL*sws^2)/2	**Mres=**	**16900**	**ft-lbs**	**15904**	**ft-lbs**
Roof Load Width (rlw)=	6.00	ft	Hd-uplift=(Mot-Mres)/sws	**Hd-uplift=**	**849**	**lbs**	**1093**	**lbs**
Floor Load Width (flw)=		ft		**Uplift from wall above =**		**lbs**		**lbs**
Dead load Reduct (swred)=	0.60			**Total HD Uplift =**	**849**	**lbs**	**1093**	**lbs**
Allowable story drift = .02*h =	3.86		$\Delta_s = 8vh^3/(EAb) + vh/(Gt) + 0.75*h*e_n + h/b*d_a =$		**1.51**	**OK**		

h/w ratio OK for wind forces **h/w ratio OK for seismic forces**

Below = Concrete Hold down location = Corner USE SIMPSON HOLDOWN: LSTHD8 OR HTT4

w2	**Seismic controls overturning, 0.6D+0.7E**				**Wind**		**Seismic**	
Short wall segment (sws)=	4.00	ft	Mot=Us*sws*h	**Mot=**	**7860**	**ft-lbs**	**8421**	**ft-lbs**
2nd Story Wall height (h)=	14.00	ft	Hdl=wwt*h+Rdl*rlw	**Hdl=**	**498**	**plf**	**468**	**plf**
Roof Load Width (rlw)=	11.50	ft	Mres=(swred *Hdl*sws^2)/2	**Mres=**	**2388**	**ft-lbs**	**2247**	**ft-lbs**
Dead load Reduct (swred)=	0.60		Hd-uplift=(Mot-Mres)/sws	**Hd-uplift=**	**1368**	**lbs**	**1543**	**lbs**
Allowable story drift = .02*h =	3.36		$\Delta_s = 8vh^3/(EAb) + vh/(Gt) + 0.75*h*e_n + h/b*d_a =$		**2.64**	**OK**		

h/w ratio OK for wind forces ***2:1 < h/w ratio < 3.5:1, tabulated shear value muliplied by 2w/h, use SW-3***

Below = Concrete Hold down location = Corner USE SIMPSON HOLDOWN: LSTHD8 OR HTT4

w3	**Seismic controls overturning, 0.6D+0.7E**				**Wind**		**Seismic**	
Short wall segment (sws)=	4.00	ft	Mot=Us*sws*h	**Mot=**	**7860**	**ft-lbs**	**8421**	**ft-lbs**
2nd Story Wall height (h)=	14.00	ft	Hdl=wwt*h+Rdl*rlw	**Hdl=**	**498**	**plf**	**468**	**plf**
Roof Load Width (rlw)=	11.50	ft	Mres=(swred *Hdl*sws^2)/2	**Mres=**	**2388**	**ft-lbs**	**2247**	**ft-lbs**
Dead load Reduct (swred)=	0.60		Hd-uplift=(Mot-Mres)/sws	**Hd-uplift=**	**1368**	**lbs**	**1543**	**lbs**
Allowable story drift = .02*h =	3.36		$\Delta_s = 8vh^3/(EAb) + vh/(Gt) + 0.75*h*e_n + h/b*d_a =$		**2.64**	**OK**		

h/w ratio OK for wind forces ***2:1 < h/w ratio < 3.5:1, tabulated shear value muliplied by 2w/h, use SW-3***

Below = Concrete Hold down location = Corner USE SIMPSON HOLDOWN: LSTHD8 OR HTT4

P7

LATERAL LOAD ANALYSIS FOR MILAN LOT 2

Floor length, L_F = **93.83**

Ω = **1.0** ASCE 7-10 Table 12.2-1

υ_{RW} = W/L_F = **29.92** **(Wind)**

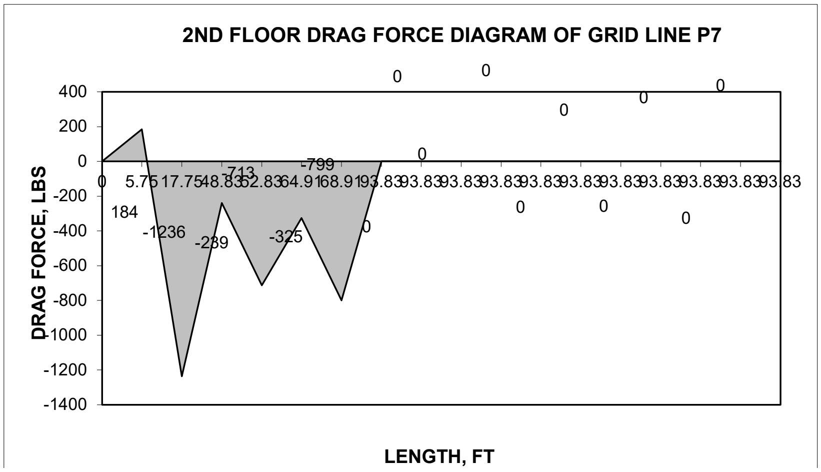
υ_W = P/Sw - υ_R = **-110.44** **(Wind)**

υ_{RE} = E/L_F = **32.05** **(Seismic)**

υ_W = P/Sw - υ_R = **-118.33** **(Seismic)**

DRAG FORCE CALCULATIONS

			Wind	Seismic	
WALL/OPENING	**LENGTH**	**Σ LENGTH**	**DRAG, LBS**	**DRAG, LBS**	**E_m LEVEL**
	0	0	**0**	**0**	**0**
OPENING	**5.75**	5.75	**172**	**184**	**184**
W1	**12.00**	17.75	**-1153**	**-1236**	**-1236**
OPENING	**31.08**	48.83	**-223**	**-239**	**-239**
W2	**4.00**	52.83	**-665**	**-713**	**-713**
OPENING	**12.08**	64.91	**-304**	**-325**	**-325**
W3	**4.00**	68.91	**-746**	**-799**	**-799**
OPENING	**24.92**	93.83	**0**	**0**	**0**
		93.83	**0**	**0**	**0**
		93.83	**0**	**0**	**0**
		93.83	**0**	**0**	**0**
		93.83	**0**	**0**	**0**
		93.83	**0**	**0**	**0**
		93.83	**0**	**0**	**0**
		93.83	**0**	**0**	**0**
		93.83	**0**	**0**	**0**
		93.83	**0**	**0**	**0**
		93.83	**0**	**0**	**0**



LATERAL LOAD ANALYSIS FOR MILAN LOT 2

Grid Line P8

MAIN OR ALT. ROOF?	MAIN
LONGITUDINAL OR TRANSVERSE?	L
END ZONE OR INTERIOR?	E

Formula		Results	Units
P=WL/Vs*sls/2	**Wind Shear Load (P)=**	**1760**	**lbs**
Us=P/Sw	**Unit Shear (Us)=**	**161**	**plf**
P=WL/Vs*sls/2	**Seismic Shear Load (P)=**	**1570**	**lbs**
Us=P/Sw	**Unit Shear (Us)=**	**144**	**plf**

At Roof **Wind governs shear wall design**

End Zone Wind Load (WL/Vs)=	135	plf
Interior Zone Wind Load (WL/Vs)=	87	plf
Seismic Load ,(WL/Vs) =	98	plf
Shear Load Span (sls)=	32.10	ft
Roof Dead Load (Rdl)=	25	psf
Wall Weight (wwt)=	15	psf
Length of Shear Wall (Sw)=	10.91	ft

Wind end zone width =	**7.57**	**ft**
Wind interior zone width =	**8.48**	**ft**

INTERIOR SHEAR WALLS: SW-2
EXTERIOR SHEAR WALLS: SW-2

Wall Overturning

w1 **Wind controls overturning, 0.6D+0.6W**

Short wall segment (sws)=	5.08	ft
2nd Story Wall height (h)=	9.08	ft
Roof Load Width (rlw)=	3.00	ft
Dead load Reduct (swred)=	0.60	
Allowable story drift = .02*h =	2.18	

Formula		Wind		Seismic	
Mot=Us*sws*h	**Mot=**	**7442**	**ft-lbs**	**6638**	**ft-lbs**
Hdl=wwt*h+Rdl*rlw	**Hdl=**	**211**	**plf**	**199**	**plf**
Mres=(swred *Hdl*sws^2)/2	**Mres=**	**1635**	**ft-lbs**	**1539**	**ft-lbs**
Hd-uplift=(Mot-Mres)/sws	**Hd-uplift=**	**1143**	**lbs**	**1004**	**lbs**

$\Delta_s = 8vh^3/(EAb) + vh/(Gt) + 0.75*h*e_n + h/b*d_a =$ **1.34** **OK**

h/w ratio OK for wind forces

h/w ratio OK for seismic forces

Below = **Wood framing**

USE SIMPSON HOLDOWN: CS16

w2 **Wind controls overturning, 0.6D+0.6W**

Short wall segment (sws)=	5.83	ft
2nd Story Wall height (h)=	9.08	ft
Roof Load Width (rlw)=	3.00	ft
Dead load Reduct (swred)=	0.60	
Allowable story drift = .02*h =	2.18	

		Wind		Seismic	
Mot=Us*sws*h	**Mot=**	**8540**	**ft-lbs**	**7618**	**ft-lbs**
Hdl=wwt*h+Rdl*rlw	**Hdl=**	**211**	**plf**	**199**	**plf**
Mres=(swred *Hdl*sws^2)/2	**Mres=**	**2154**	**ft-lbs**	**2027**	**ft-lbs**
Hd-uplift=(Mot-Mres)/sws	**Hd-uplift=**	**1095**	**lbs**	**959**	**lbs**

$\Delta_s = 8vh^3/(EAb) + vh/(Gt) + 0.75*h*e_n + h/b*d_a =$ **1.22** **OK**

h/w ratio OK for wind forces

h/w ratio OK for seismic forces

Below = **Wood framing**

USE SIMPSON HOLDOWN: CS16

LATERAL LOAD ANALYSIS FOR MILAN LOT 2

Grid Line P8

MAIN OR ALT. ROOF?	MAIN
LONGITUDINAL OR TRANSVERSE?	L
END ZONE OR INTERIOR?	I

Formula		Results	Units
P=WL/Vs*sls/2	**Wind Shear Load (P)=**	**6843**	**lbs**
Us=P/Sw	**Unit Shear (Us)=**	**159**	**plf**
P=WL/Vs*sls/2	**Seismic Shear Load (P)=**	**7016**	**lbs**
Us=P/Sw	**Unit Shear (Us)=**	**163**	**plf**
	Wind end zone width =	**0.00**	**ft**
	Wind interior zone width =	**30.71**	**ft**

INTERIOR SHEAR WALLS: SW-2

EXTERIOR SHEAR WALLS: SW-2

At Floor **Wind governs shear wall design**

End Zone Wind Load (WL/Vs)=	248	plf
Interior Zone Wind Load (WL/Vs)=	166	plf
Seismic Load ,(WL/Vs) =	177	plf
Shear Load Span (sls)=	61.42	ft
Roof Dead Load (Rdl)=	25	psf
Floor Dead Load (Fdl)=	15	psf
Wall Weight (wwt)=	15	psf
Length of Shear Wall (Sw)=	43.00	ft

Wall Overturning

w1	**Wind controls overturning, 0.6D+0.6W**		**Formula**		**Wind**		**Seismic**	
Short wall segment (sws)=	19.75	ft	Mot=Us*sws*h	**Mot=**	**28538**	**ft-lbs**	**29260**	**ft-lbs**
2nd Story Wall height (h)=	9.08	ft	DL=(wwt*(hf+hs))+(rlw*Rdl)+(flw*Fdl)	**DL=**	**462**	**plf**	**435**	**plf**
1st Story Wall height (h)=	9.08	ft	Mres=(swred *DL*sws^2)/2	**Mres=**	**54109**	**ft-lbs**	**50920**	**ft-lbs**
Roof Load Width (rlw)=	7.00	ft	Hd-uplift=(Mot-Mres)/sws	**Hd-uplift=**	**-1295**	**lbs**	**-1097**	**lbs**
Floor Load Width (flw)=	1.00	ft		**Uplift from wall above =**	**1143**	**lbs**	**554**	**lbs**
Dead load Reduct (swred)=	0.60			**Total HD Uplift =**	**-152**	**lbs**	**-543**	**lbs**
Allowable story drift = .02*h =	2.18		$\Delta_s = 8vh^3/(EAb) + vh/(Gt) + 0.75*h*e_n + h/b*d_a$ =		**0.65**	**OK**		

h/w ratio OK for wind forces **h/w ratio OK for seismic forces**

Below = **Concrete** **Corner** ***NO HOLDOWNS REQUIRED***

Perforated Wall (SDPWS 2008 Table 4.3.3.5) **C_o =** 0.695 **Required Shear wall =** ***SW-4***

Perforated wall Length (sws)=	23.25	ft		**Sill plate uplift anchorage:**	**SW-2**			
Full ht segment lengths =			8.00	4.25	**Wind**		**Seismic**	
			Mot=Us*sws*h	**Mot=**	**51800**	**ft-lbs**	**53109**	**ft-lbs**
% Full Height sheathing =	53%							
Max Opening Ht =	9.00	ft	DL=(wwt*(hf+hs))+(rlw*Rdl)+(flw*Fdl)	**DL=**	**498**	**plf**	**468**	**plf**
2nd Story Wall height (h)=	0.00	ft	Mres=(swred *DL*sws^2)/2	**Mres=**	**80679**	**ft-lbs**	**75924**	**ft-lbs**
1st Story Wall height (h)=	14.00	ft	T/C =V*h/(Co*ΣL)	**T/C =**	**6086**	**lbs**	**6240**	**lbs**
Roof Load Width (rlw)=	11.50	ft	Hd-uplift=(Mot-Mres)/sws	**Hd-uplift=**	**2616**	**lbs**	**2975**	**lbs**
Floor Load Width (flw)=		ft		**Uplift from wall above =**		**lbs**		**lbs**
Dead load Reduct (swred)=	0.60			**Total HD Uplift =**	**2616**	**lbs**	**2975**	**lbs**
Allowable story drift = .02*h =	3.36		$\Delta_s = 8vh^3/(EAb) + vh/(Gt) + 0.75*h*e_n + h/b*d_a$ =		**0.99**	**OK**		

h/w ratio OK for wind forces ***2:1 < h/w ratio < 3.5:1, shear force muliplied by h/2w***

Below = **Concrete** **Hold down location =** **Corner** **USE SIMPSON HOLDOWN: STHD14 OR HTT4**

P8

LATERAL LOAD ANALYSIS FOR MILAN LOT 2

Roof length, L_R = **24.92** Ω = **1.0** ASCE 7-10 Table 12.2-1

υ_{RW} = W/L_R = **70.63** **(Wind)**

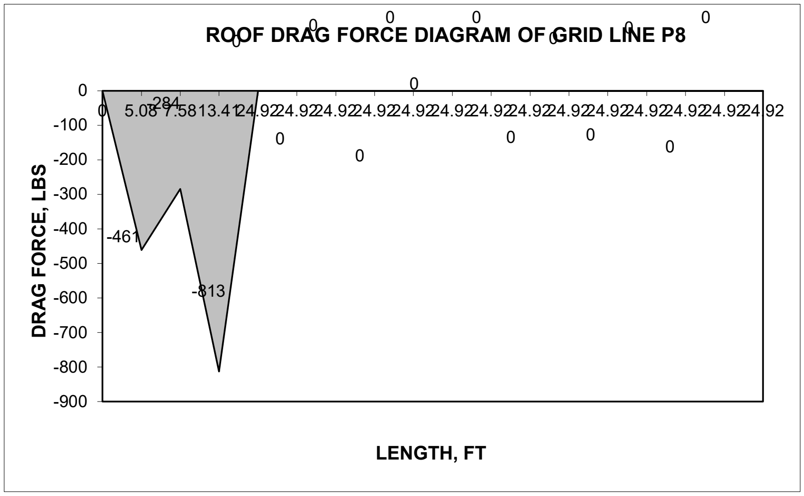
υ_W = P/Sw - υ_R = **-90.70** **(Wind)**

υ_{RE} = E/L_R = **63.00** **(Seismic)**

υ_W = P/Sw - υ_R = **-80.90** **(Seismic)**

DRAG FORCE CALCULATIONS

WALL/OPENING	LENGTH	Σ LENGTH	Wind DRAG, LBS	Seismic DRAG, LBS	E_m LEVEL
	0	0	0	0	0
W1	5.08	5.08	-461	-411	-411
OPENING	2.50	7.58	-284	-253	-253
W2	5.83	13.41	-813	-725	-725
OPENING	11.51	24.92	0	0	0
		24.92	0	0	0
		24.92	0	0	0
		24.92	0	0	0
		24.92	0	0	0
		24.92	0	0	0
		24.92	0	0	0
		24.92	0	0	0
		24.92	0	0	0
		24.92	0	0	0
		24.92	0	0	0
		24.92	0	0	0
		24.92	0	0	0
		24.92	0	0	0



LATERAL LOAD ANALYSIS FOR MILAN LOT 2

Floor length, L_F = **93.83**

Ω = **1.0** ASCE 7-10 Table 12.2-1

υ_{RW} = W/L_F = **54.17** **(Wind)**

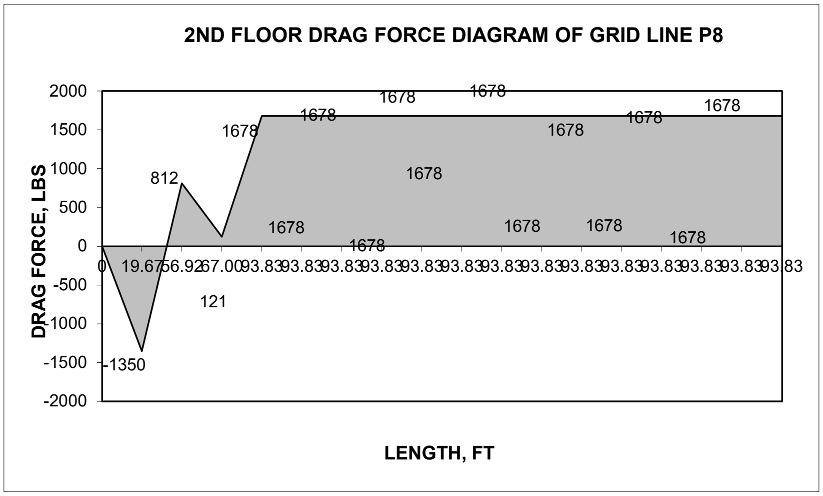
υ_W = P/Sw - υ_R = **-64.03** **(Wind)**

υ_{RE} = E/L_F = **58.04** **(Seismic)**

υ_W = P/Sw - υ_R = **-68.61** **(Seismic)**

DRAG FORCE CALCULATIONS

WALL/OPENING	LENGTH	Σ LENGTH	Wind DRAG, LBS	Seismic DRAG, LBS	E_m LEVEL
	0	0	**0**	**0**	**0**
w1	**19.67**	19.67	**-1260**	**-1350**	**-1350**
OPENING	**37.25**	56.92	**758**	**812**	**812**
W2	**10.08**	67.00	**113**	**121**	**121**
OPENING	**26.83**	93.83	**1566**	**1678**	**1678**
		93.83	**1566**	**1678**	**1678**
		93.83	**1566**	**1678**	**1678**
		93.83	**1566**	**1678**	**1678**
		93.83	**1566**	**1678**	**1678**
		93.83	**1566**	**1678**	**1678**
		93.83	**1566**	**1678**	**1678**
		93.83	**1566**	**1678**	**1678**
		93.83	**1566**	**1678**	**1678**
		93.83	**1566**	**1678**	**1678**
		93.83	**1566**	**1678**	**1678**
		93.83	**1566**	**1678**	**1678**
		93.83	**1566**	**1678**	**1678**
		93.83	**1566**	**1678**	**1678**



LATERAL LOAD ANALYSIS FOR MILAN LOT 2

Grid Line P9

MAIN OR ALT. ROOF?	ALT.
LONGITUDINAL OR TRANSVERSE?	L
END ZONE OR INTERIOR?	E

Formula		Results	Units
P=WL/Vs*sls/2	**Wind Shear Load (P)=**	**3455**	**lbs**
Us=P/Sw	**Unit Shear (Us)=**	**276**	**plf**
P=WL/Vs*sls/2	**Seismic Shear Load (P)=**	**2079**	**lbs**
Us=P/Sw	**Unit Shear (Us)=**	**166**	**plf**

At Roof **Wind governs shear wall design**

End Zone Wind Load (WL/Vs)=	210	plf
Interior Zone Wind Load (WL/Vs)=	137	plf
Seismic Load ,(WL/Vs) =	98	plf
Shear Load Span (sls)=	42.50	ft
Roof Dead Load (Rdl)=	25	psf
Wall Weight (wwt)=	15	psf
Length of Shear Wall (Sw)=	12.50	ft

Wind end zone width = 7.57 ft

Wind interior zone width = 13.68 ft

INTERIOR SHEAR WALLS: SW-3

EXTERIOR SHEAR WALLS: SW-3

Wall Overturning

w1 **Wind controls overturning, 0.6D+0.6W**

			Formula		Wind		Seismic	
Short wall segment (sws)=	6.25	ft	Mot=Us*sws*h	**Mot=**	**15684**	**ft-lbs**	**9437**	**ft-lbs**
2nd Story Wall height (h)=	9.08	ft	Hdl=wwt*h+Rdl*rlw	**Hdl=**	**236**	**plf**	**222**	**plf**
Roof Load Width (rlw)=	4.00	ft	Mres=(swred *Hdl*sws^2)/2	**Mres=**	**2768**	**ft-lbs**	**2605**	**ft-lbs**
Dead load Reduct (swred)=	0.60		Hd-uplift=(Mot-Mres)/sws	**Hd-uplift=**	**2067**	**lbs**	**1093**	**lbs**
Allowable story drift = .02*h =	2.18		$\Delta_s = 8vh^3/(EAb) + vh/(Gt) + 0.75*h*e_n + h/b*d_a =$		**1.10**	**OK**		

h/w ratio OK for wind forces **h/w ratio OK for seismic forces**

Below = Concrete Hold down location = Endwall USE SIMPSON HOLDOWN: STHD10 OR HTT4

w2 **Wind controls overturning, 0.6D+0.6W**

					Wind		Seismic	
Short wall segment (sws)=	6.25	ft	Mot=Us*sws*h	**Mot=**	**15684**	**ft-lbs**	**9437**	**ft-lbs**
2nd Story Wall height (h)=	9.08	ft	Hdl=wwt*h+Rdl*rlw	**Hdl=**	**236**	**plf**	**222**	**plf**
Roof Load Width (rlw)=	4.00	ft	Mres=(swred *Hdl*sws^2)/2	**Mres=**	**2768**	**ft-lbs**	**2605**	**ft-lbs**
Dead load Reduct (swred)=	0.60		Hd-uplift=(Mot-Mres)/sws	**Hd-uplift=**	**2067**	**lbs**	**1093**	**lbs**
Allowable story drift = .02*h =	2.18		$\Delta_s = 8vh^3/(EAb) + vh/(Gt) + 0.75*h*e_n + h/b*d_a =$		**1.10**	**OK**		

h/w ratio OK for wind forces **h/w ratio OK for seismic forces**

Below = Concrete Hold down location = Corner USE SIMPSON HOLDOWN: LSTHD8 OR HTT4

LATERAL LOAD ANALYSIS FOR MILAN LOT 2

Roof length, L_R = **49.00**

Ω = **1.0** ASCE 7-10 Table 12.2-1

υ_{RW} = W/L_R = **70.50** **(Wind)**

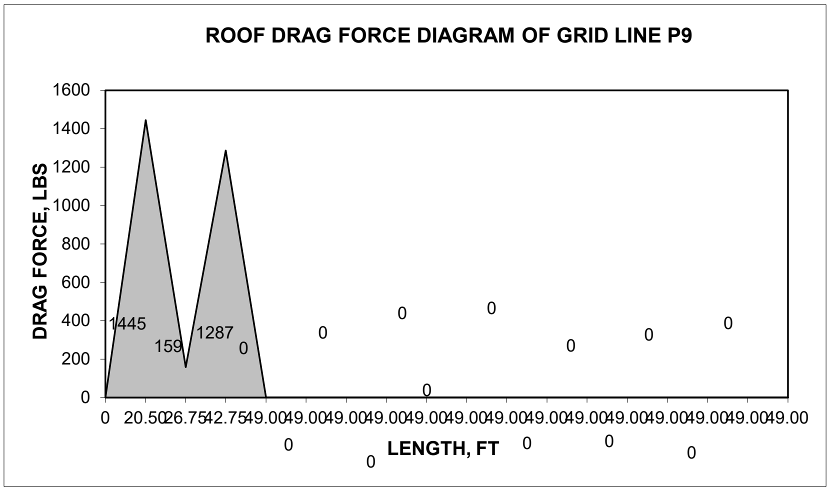
υ_W = P/Sw - υ_R = **-205.87** **(Wind)**

υ_{RE} = E/L_R = **42.42** **(Seismic)**

υ_W = P/Sw - υ_R = **-123.87** **(Seismic)**

DRAG FORCE CALCULATIONS

WALL/OPENING	LENGTH	Σ LENGTH	Wind DRAG, LBS	Seismic DRAG, LBS	E_m LEVEL
	0	0	**0**	**0**	**0**
OPENING	**20.50**	20.50	**1445**	**870**	**870**
W1	**6.25**	26.75	**159**	**95**	**95**
OPENING	**16.00**	42.75	**1287**	**774**	**774**
W2	**6.25**	49.00	**0**	**0**	**0**
		49.00	**0**	**0**	**0**
		49.00	**0**	**0**	**0**
		49.00	**0**	**0**	**0**
		49.00	**0**	**0**	**0**
		49.00	**0**	**0**	**0**
		49.00	**0**	**0**	**0**
		49.00	**0**	**0**	**0**
		49.00	**0**	**0**	**0**
		49.00	**0**	**0**	**0**
		49.00	**0**	**0**	**0**
		49.00	**0**	**0**	**0**
		49.00	**0**	**0**	**0**
		49.00	**0**	**0**	**0**



LATERAL LOAD ANALYSIS FOR MILAN LOT 2

Grid Line P10

MAIN OR ALT. ROOF?	ALT.
LONGITUDINAL OR TRANSVERSE?	L
END ZONE OR INTERIOR?	E

Formula		Results	Units
P=WL/Vs*sls/2	**Wind Shear Load (P)=**	**2377**	**lbs**
Us=P/Sw	**Unit Shear (Us)=**	**144**	**plf**
P=WL/Vs*sls/2	**Seismic Shear Load (P)=**	**2457**	**lbs**
Us=P/Sw	**Unit Shear (Us)=**	**149**	**plf**

At Roof **Wind governs shear wall design**

End Zone Wind Load (WL/Vs)=	149	plf
Interior Zone Wind Load (WL/Vs)=	93	plf
Seismic Load ,(WL/Vs) =	117	plf
Shear Load Span (sls)=	42.00	ft
Roof Dead Load (Rdl)=	25	psf
Wall Weight (wwt)=	15	psf
Length of Shear Wall (Sw)=	16.50	ft

Wind end zone width = 7.57 ft

Wind interior zone width = 13.43 ft

INTERIOR SHEAR WALLS: SW-2

EXTERIOR SHEAR WALLS: SW-2

Wall Overturning

Perforated Wall (SDPWS 2008 Table 4.3.3.5)

C_o = 0.737

Required Shear wall = ***SW-2***

Sill plate uplift anchorage: SW-2

Perforated wall Length (sws)=			16.50	ft
Full ht segment lengths =	3.83	6.67		
% Full Height sheathing =			64%	
Max Opening Ht =			6.00	ft
2nd Story Wall height (h)=			9.08	ft
Roof Load Width (rlw)=			3.00	ft
Dead load Reduct (swred)=			0.60	
Allowable story drift = .02*h =			2.18	

		Wind		Seismic	
Mot=Us*sws*h	**Mot=**	**21580**	**ft-lbs**	**22310**	**ft-lbs**
Hdl=wwt*h+Rdl*rlw	**Hdl=**	**211**	**plf**	**199**	**plf**
Mres=(swred *Hdl*sws^2)/2	**Mres=**	**17250**	**ft-lbs**	**16233**	**ft-lbs**
T/C =V*h/(Co*ΣL)	**T/C =**	**2789**	**lbs**	**2884**	**lbs**
Hd-uplift=(Mot-Mres)/sws	**Hd-uplift=**	**1744**	**lbs**	**1900**	**lbs**

$\Delta_s = 8vh^3/(EAb) + vh/(Gt) + 0.75*h*e_n + h/b*d_a =$ **0.75 OK**

h/w ratio OK for wind forces

2:1 < h/w ratio < 3.5:1, shear force muliplied by h/2w

Below = Concrete Hold down location = Corner USE SIMPSON HOLDOWN: LSTHD8 OR HTT4

LATERAL LOAD ANALYSIS FOR MILAN LOT 2

Roof length, L_R = **31.92**

Ω = **1.0** ASCE 7-10 Table 12.2-1

υ_{RW} = W/L_R = **74.46** **(Wind)**

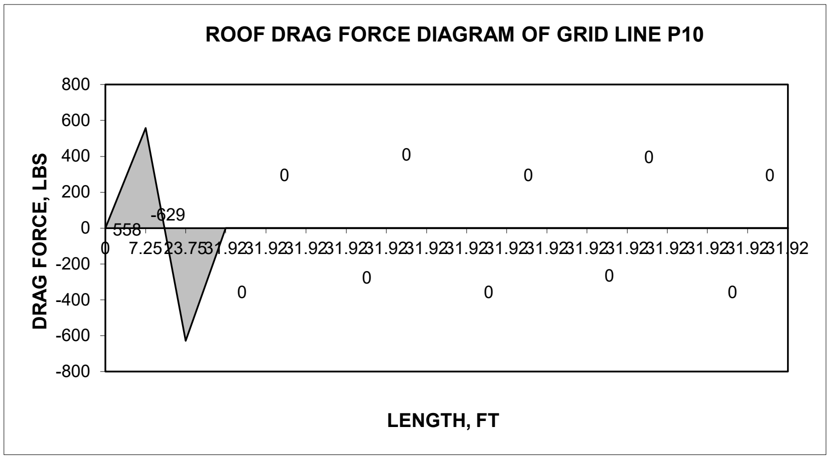
υ_W = P/Sw - υ_R = **-69.58** **(Wind)**

υ_{RE} = E/L_R = **76.97** **(Seismic)**

υ_W = P/Sw - υ_R = **-71.94** **(Seismic)**

DRAG FORCE CALCULATIONS

WALL/OPENING	LENGTH	Σ LENGTH	Wind DRAG, LBS	Seismic DRAG, LBS	E_m LEVEL
	0	0	**0**	**0**	**0**
OPENING	**7.25**	7.25	**540**	**558**	**558**
W1	**16.50**	23.75	**-608**	**-629**	**-629**
OPENING	**8.17**	31.92	**0**	**0**	**0**
		31.92	**0**	**0**	**0**
		31.92	**0**	**0**	**0**
		31.92	**0**	**0**	**0**
		31.92	**0**	**0**	**0**
		31.92	**0**	**0**	**0**
		31.92	**0**	**0**	**0**
		31.92	**0**	**0**	**0**
		31.92	**0**	**0**	**0**
		31.92	**0**	**0**	**0**
		31.92	**0**	**0**	**0**
		31.92	**0**	**0**	**0**
		31.92	**0**	**0**	**0**
		31.92	**0**	**0**	**0**
		31.92	**0**	**0**	**0**



LATERAL LOAD ANALYSIS FOR MILAN LOT 2
2ND FLOOR DIAPHRAGM AND TOP CHORD FORCES PER ASCE 7-10 12.10

WORST CASE TRANSVERSE

Diaphragm type = Supported
Case = Case 1
Depth of diaphragm (b) = 32.00 ft — Ratio = 0.8:1
Length of diaphragm (L) = 24.67 ft — ***OK***
Transverse Wind Lateral Load @ roof = 87 plf
Transverse Wind Lateral Load @ floor = 166 plf
Transverse Seismic Lateral Load @ roof = 114 plf
Transverse Seismic Lateral Load @ floor = 106 plf
Max Seismic diaphragm load, $F_p = 0.4I_ES_{DS}w_{px}$ = 305 plf (ASCE 7-10 12.10-3)
Min Seismic diaphragm load, $F_p = 0.2I_ES_{DS}w_{px}$ = 152 plf (ASCE 7-10 12.10-2)
w_2 = tributary wt to roof = 38193 lbs
w_1 = tributary wt to floor = 254758 lbs

		Results		
Formula		**Wind**	**Seismic**	
ASCE 7-10 12.10-1	$F_{px} = \Sigma F_{it}/\Sigma w_i * w_{px}$ =	166	152	plf
Msls=WL/Vs*L/2	**Max Shear (Msls)**	***64***	**59**	**plf**
Mols=Msls*L^2/8	Moment (Mols)	12591	11597	ft-lbs
Tcl=Mols/W	**Top Chord Force (Tcl)**	***393***	**362**	**lbs**
	Required Diaphragm =	**FD-1**	**FD-1**	
IBC 2012 EQN 23-1	Δ = 5vl^3/(8Eab) + vL/(4Gt) + 0.188*L*en + Σ(ΔcX)/2b =	**0.11899887**	**0.116**	**in**
		L/2487	**L/2550**	

FD-1
Use 3/4" APA rated sheathing, exp. 1 unblocked with 10d common nails at 6" o.c.
along panel edges, shear walls and perimeter with intermediate nails at 12" o.c. in the field.
Allowable Shear = 400 plf (W), 285 plf (E)

TC-1
Splice double top plate w/ 8-16d nails each side in 4'-0"splice
Allowable tension = 1190 lbs

WORST CASE LONGITUDINAL

Diaphragm type = Supported
Case = Case 3
Depth of diaphragm (b) = 24.67 ft — Ratio = 1.3:1
Length of diaphragm (L) = 32.00 ft — ***OK***
Longitudinal Wind Lateral Load @ roof = 87 plf
Longitudinal Wind Lateral Load @ floor = 166 plf
Longitudinal Seismic Lateral Load @ roof = 75 plf
Longitudinal Seismic Lateral Load @ floor = 136 plf
Max Seismic diaphragm load, $F_p = 0.4I_ES_{DS}w_{px}$ = 393 plf (ASCE 7-10 12.10-3)
Min Seismic diaphragm load, $F_p = 0.2I_ES_{DS}w_{px}$ = 196 plf (ASCE 7-10 12.10-2)
w_2 = tributary wt to roof = 38193 lbs
w_1 = tributary wt to floor = 254758 lbs

		Results		
Formula		**Wind**	**Seismic**	
ASCE 7-10 12.10-1	$F_{px} = \Sigma F_{it}/\Sigma w_i * w_{px}$ =	166	196	plf
Msls=WL/Vs*L/2	**Max Shear (Msls)**	**107**	***127***	**plf**
Mols=Msls*L^2/8	Moment (Mols)	21185	25124	ft-lbs
Tcl=Mols/W	**Top Chord Force (Tcl)**	**859**	***1018***	**lbs**
	Required Diaphragm =	**FD-1**	**FD-1**	
IBC 2012 EQN 23-1	Δ = 5vl^3/(8Eab) + vL/(4Gt) + 0.188*L*en + Σ(ΔcX)/2b =	**0.1135723**	**0.125**	**in**
		L/3381	**L/3083**	

FD-1
Use 3/4" APA rated sheathing, exp. 1 unblocked with 10d common nails at 6" o.c.
along panel edges, shear walls and perimeter with intermediate nails at 12" o.c. in the field.
Allowable Shear = 300 plf (W), 215 plf (E)

LATERAL LOAD ANALYSIS FOR MILAN LOT 2
ROOF DIAPHRAGM AND TOP CHORD FORCES PER ASCE 7-10 12.10

WORST CASE TRANSVERSE

Diaphragm type = Supported
Case = Case 1
Depth of diaphragm (b) = 41.00 ft Ratio = 0.7:1
Length of diaphragm (L) = 28.00 ft *OK*
Transverse Wind Lateral Load @ roof = 87 plf
Transverse Wind Lateral Load @ floor = 166 plf
Transverse Seismic Lateral Load @ roof = 114 plf
Transverse Seismic Lateral Load @ floor = 106 plf
Max Seismic diaphragm load, $F_p = 0.4I_ES_{DS}w_{px}$ = 181 plf (ASCE 7-10 12.10-3)
Min Seismic diaphragm load, $F_p = 0.2I_ES_{DS}w_{px}$ = 90 plf (ASCE 7-10 12.10-2)
w_2 = tributary wt to roof = 38193 lbs
w_1 = tributary wt to floor = 254758 lbs

		Results		
Formula		**Wind**	**Seismic**	
ASCE 7-10 12.10-1	$F_{px} = \Sigma F_{it}/\Sigma w_i * w_{px}$ =	87	114	plf
Msls=WL/Vs*L/2	**Max Shear (Msls)**	**30**	***39***	**plf**
Mols=Msls*L^2/8	Moment (Mols)	8493	11194	ft-lbs
Tcl=Mols/W	**Top Chord Force (Tcl)**	**207**	***273***	**lbs**
	Required Diaphragm =	**RD-1**	**RD-1**	
IBC 2012 EQN 23-1	Δ = 5vl^3/(8Eab) + vL/(4Gt) + 0.188*L*en + Σ(ΔcX)/2b =	**0.14795935**	**0.155**	**in**
		L/2270	**L/2167**	

RD-1
Use 7/16" APA rated sheathing, exp. 1 unblocked with 8d common nails at 6" o.c.
along panel edges, shear walls and perimeter with intermediate nails at 12" o.c. in the field.
Allowable Shear = 323 plf (W), 230 plf (E)

TC-1
Splice double top plate w/ 8-16d nails each side in 4'-0"splice
Allowable tension = 1190 lbs

WORST CASE LONGITUDINAL

Diaphragm type = Supported
Case = Case 3
Depth of diaphragm (b) = 28.00 ft Ratio = 1.5:1
Length of diaphragm (L) = 41.00 ft *OK*
Longitudinal Wind Lateral Load @ roof = 87 plf
Longitudinal Wind Lateral Load @ floor = 166 plf
Longitudinal Seismic Lateral Load @ roof = 75 plf
Longitudinal Seismic Lateral Load @ floor = 136 plf
Max Seismic diaphragm load, $F_p = 0.4I_ES_{DS}w_{px}$ = 119 plf (ASCE 7-10 12.10-3)
Min Seismic diaphragm load, $F_p = 0.2I_ES_{DS}w_{px}$ = 60 plf (ASCE 7-10 12.10-2)
w_2 = tributary wt to roof = 38193 lbs
w_1 = tributary wt to floor = 254758 lbs

		Results		
Formula		**Wind**	**Seismic**	
ASCE 7-10 12.10-1	$F_{px} = \Sigma F_{it}/\Sigma w_i * w_{px}$ =	87	75	plf
Msls=WL/Vs*L/2	**Max Shear (Msls)**	***63***	**55**	**plf**
Mols=Msls*L^2/8	Moment (Mols)	18209	15811	ft-lbs
Tcl=Mols/W	**Top Chord Force (Tcl)**	***650***	**565**	**lbs**
	Required Diaphragm =	**RD-1**	**RD-1**	
IBC 2012 EQN 23-1	Δ = 5vl^3/(8Eab) + vL/(4Gt) + 0.188*L*en + Σ(ΔcX)/2b =	**0.12993507**	**0.124**	**in**
		L/3786	**L/3968**	

RD-1
Use 7/16" APA rated sheathing, exp. 1 unblocked with 8d common nails at 6" o.c.
along panel edges, shear walls and perimeter with intermediate nails at 12" o.c. in the field.
Allowable Shear = 238 plf (W), 170 plf (E)

ASD Wood Member Design v7.3.1 (2-17-17)

PROJECT MILAN LOT 2 (RB-1)

Member Dimensions Beam ● Joist ○

	Cantilever	Span	Cantilever	Total Length
Span =		16.50		16.50
Unbraced length =		16.00	ft	
Number of plys =		2		
Member width, b =		1.75	in ☐ Custom width	
Member depth, d =		11.875	in ☐ Custom depth	
Orientation =		Strong		

Member Material Properties

Lumber type = Engineered
Type = LVL
Grade = 1.9E
Member unit weight = 42 pcf
Bearing length @ support A = 3.00 in ≥ 1.5 in
Bearing length @ support B = 3.00 in ≥ 1.5 in

Loads

Roof DL = 25 psf
Roof Lr = 20 psf
Snow, S = 15 psf
Rain, R = 5 psf
Floor DL = 15 psf
Floor LL = 40 psf

*****TL deflection controls member design (68%)*****

Unfactored Load Reactions

Load type	*R_A*	*R_B*	
D =	1513	1513	*lbs*
L =	0	0	*lbs*
Lr =	1073	1073	*lbs*
S =	0	0	*lbs*
R =	0	0	*lbs*
W =	0	0	*lbs*
E =	0	0	*lbs*

Point Loads

	P_L	P_D	P_T	a	LL Type	Load Description
P_1 =			0		Roof	
P_2 =			0			
P_3 =			0			
P_4 =			0			
P_5 =			0			
P_6 =			0			

Uniform Loads **Roof slope =** 4.00 :12 **Member slope =** :12

Live, psf	Dead, psf	Trib. Width		W_L	W_D	W_T	Start @	End @	LL Type	Load Description
20	25	6.50	W_1 =	130	171	301	0.00	16.50	Roof	
			W_2 =			0				
			W_3 =			0	0.00			
			W_4 =			0				
			W_5 =			0				
			W_6 =			0				

Triangular Loads (Starting or ending load must be 0)

	Start W_L	Start W_D	End W_L	End W_D	Start W_T	End W_T	Start @	End @	LL Type	Load Description
T_1 =					0	0				
T_2 =					0	0				
T_3 =					0	0				
T_4 =					0	0				

Member Shear Design ***Member design controlled by D+(Lr or S or R)***

Fv = 285 psi
Fv' = Fv*($C_DC_MC_tC_i$) = 356 psi
Max V = 2586 lbs
Design V = 2236 lbs
A = b*d = 41.56 in^2
fv = 1.5*V/A = 93 psi
OK

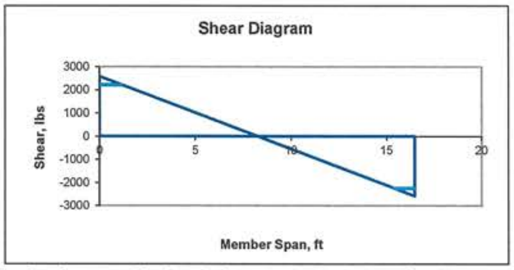
Adjustment Factors

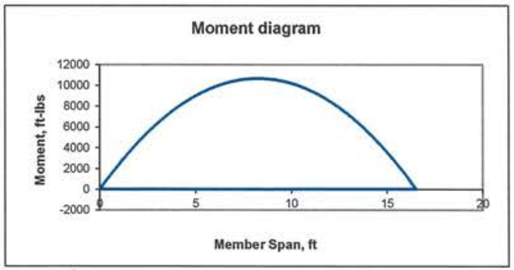
C_D =	1.25	For roof live load
C_M =	1.00	For MC < 19%
C_t =	1.00	Insulated against 100+ F
C_F =		No size increase
C_V =	1.00	Volume Factor
C_{fu} =		Narrow face loaded
C_i =		No incising
C_r =	1.00	Not a repetitive member
C_f =		Rectangular shaped
C_T =		Buckling stiffness factor
C_b =	1.00	Bearing area factor

Member Bending Design ***Member design controlled by D+(Lr or S or R)***

	Fb^+	Fb^-	
Fb =	2600	2600	psi
Fb* = Fb*($C_DC_MC_tC_FC_iC_rC_f$) =	3250	3250	psi
Emin =	965710	965710	psi
Emin' = Emin*(CMCtCiCT) =	965710	965710	psi
unbraced length, lu =	16.00	16.00	ft
lu/d =	16.17	16.17	
le =	353	353	in
$R_B = (le*d/b^2)^{1/2}$ =	18.51	18.51	***≤ 50, OK***
$F_{bE} = 1.20*E'min/(R_B)^2$ =	3384	3384	psi
C_L =	0.833	0.833	
Fb' = Fb*($C_DC_MC_tC_LC_FC_VC_{fu}C_iC_rC_f$) =	2712	2712	psi

	+ Moment	**- Moment**	
Max moment, M =	10666		lb-ft
S = $bd^2/6$ =	82.26	82.26	in^3
fb = M/S =	1556	0	psi
	OK	***OK***	





Member Bearing **Member design controlled by D+(Lr or S or R)**

Fc⊥ = 625 psi
F'c⊥ = Fc⊥*($C_MC_tC_iC_b$) = 625 psi

	C_b	P, lbs	A, in^2	fc⊥ = P/A		
Support @ A =	1.00	2586	10.50	246	psi	***OK***
Support @ B =	1.00	2586	10.50	246	psi	***OK***

Member Deflection

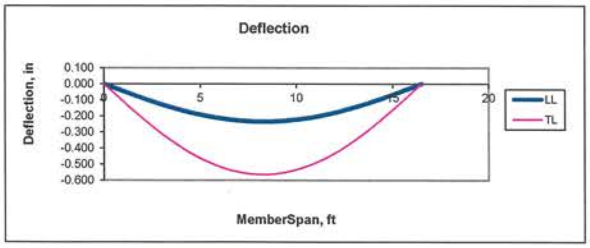
Moment of Inertia, I = $bd^3/12$ = 488.413 in^4
E = 1900000 psi
E' = E*($C_MC_tC_i$) = 1900000 psi

Mid Span Deflection

Loading	$Ratio_{allow}$	$\Delta_{allowed}$	Δ_{actual}	$Ratio_{actual}$	Check
Δ_{LL}	360	0.550	0.234	L/847	***OK***
Δ_{TL}	240	0.825	0.563	L/351	***OK***

Cantilever Deflection

Loading	$Ratio_{allow}$	$\Delta_{allowed}$	Δ_{actual}	$Ratio_{actual}$	Check
Δ_{LL}	180	0.000	0.000	N/A	***OK***
Δ_{TL}	120	0.000	0.000	N/A	***OK***



(2) 1.75" x 11.875" LVL 1.9E

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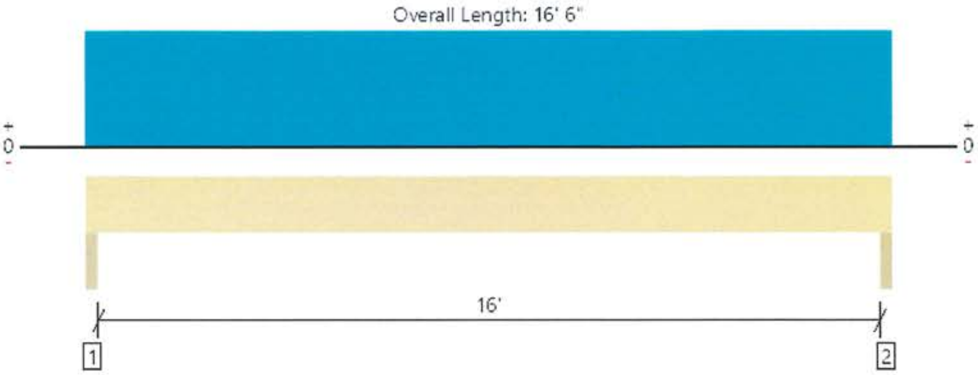
MEMBER REPORT

PASSED

Level, RB-2

2 piece(s) 1 3/4" x 11 7/8" 2.0E Microllam® LVL



All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal.

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	2394 @ 1 1/2"	7875 (3.00")	Passed (30%)	--	1.0 D + 1.0 Lr (All Spans)
Shear (lbs)	2035 @ 1' 2 7/8"	9871	Passed (21%)	1.25	1.0 D + 1.0 Lr (All Spans)
Moment (Ft-lbs)	9580 @ 8' 3"	22310	Passed (43%)	1.25	1.0 D + 1.0 Lr (All Spans)
Live Load Defl. (in)	0.204 @ 8' 3"	0.813	Passed (L/957)	--	1.0 D + 1.0 Lr (All Spans)
Total Load Defl. (in)	0.493 @ 8' 3"	1.083	Passed (L/396)	--	1.0 D + 1.0 Lr (All Spans)

System : Roof
Member Type : Drop Beam
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD
Member Pitch : 0/12

- Deflection criteria: LL (L/240) and TL (L/180).
- Top Edge Bracing (Lu): Top compression edge must be braced at 15' 7" o/c based on loads applied, unless detailed otherwise.
- Bottom Edge Bracing (Lu): Bottom compression edge must be braced at 16' 6" o/c based on loads applied, unless detailed otherwise.

	Bearing Length			Loads to Supports (lbs)			
Supports	Total	Available	Required	Dead	Roof Live	Total	Accessories
1 - Trimmer - DF	3.00"	3.00"	1.50"	1404	990	2394	None
2 - Trimmer - DF	3.00"	3.00"	1.50"	1404	990	2394	None

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Roof Live (non-snow: 1.25)	Comments
0 - Self Weight (PLF)	0 to 16' 6"	N/A	12.1	--	
1 - Uniform (PSF)	0 to 16' 6" (Front)	6'	26.4	20.0	Default Load

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
L RIED POPE PE, PLS LR POPE ENGINEERING INC (435) 628-1676 lrpope@lrpope.com	

Weyerhaeuser

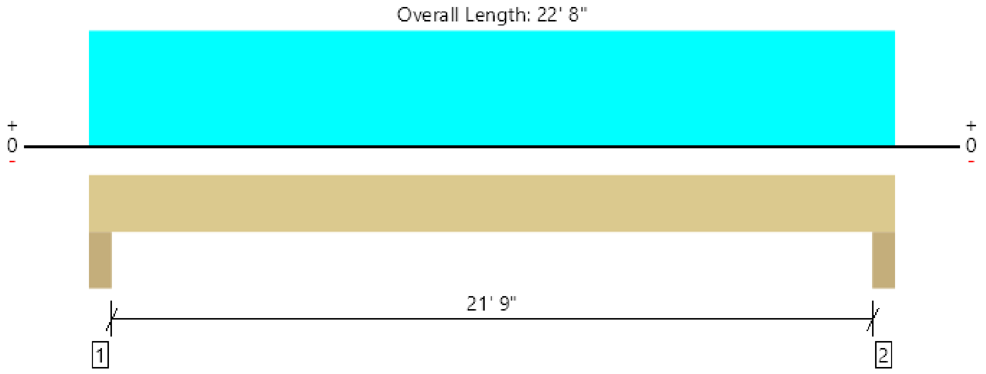
6/1/2020 8:40:11 PM UTC
ForteWEB v2.4, Engine: V8.0.1.5, Data: V7.3.2.0
File Name: MILAN LOT 2

MEMBER REPORT

PASSED

Level, RB-3

1 piece(s) 5 1/2" x 21" 24F-V4 DF Glulam



All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal.

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	15955 @ 4"	19663 (5.50")	Passed (81%)	--	1.0 D + 1.0 Lr (All Spans)
Shear (lbs)	12846 @ 2' 2 1/2"	25506	Passed (50%)	1.25	1.0 D + 1.0 Lr (All Spans)
Pos Moment (Ft-lbs)	85170 @ 11' 4"	94449	Passed (90%)	1.25	1.0 D + 1.0 Lr (All Spans)
Live Load Defl. (in)	0.379 @ 11' 4"	1.100	Passed (L/696)	--	1.0 D + 1.0 Lr (All Spans)
Total Load Defl. (in)	0.971 @ 11' 4"	1.467	Passed (L/272)	--	1.0 D + 1.0 Lr (All Spans)

System : Roof
Member Type : Drop Beam
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD
Member Pitch : 0/12

- Deflection criteria: LL (L/240) and TL (L/180).
- Allowed moment does not reflect the adjustment for the beam stability factor.
- Critical positive moment adjusted by a volume factor of 0.93 that was calculated using length L = 22'.
- The effects of positive or negative camber have not been accounted for when calculating deflection.
- The specified glulam is assumed to have its strong laminations at the bottom of the beam. Install with proper side up as indicated by the manufacturer.
- Applicable calculations are based on NDS.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Roof Live	Total	
1 - Column - DF	5.50"	5.50"	4.46"	9721	6233	15954	None
2 - Column - DF	5.50"	5.50"	4.46"	9721	6233	15954	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	18' 1" o/c	
Bottom Edge (Lu)	22' 8" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Roof Live (non-snow: 1.25)	Comments
0 - Self Weight (PLF)	0 to 22' 8"	N/A	28.1	--	
1 - Uniform (PSF)	0 to 22' 8" (Front)	27' 6"	26.4	20.0	Default Load
2 - Uniform (PSF)	0 to 22' 8" (Front)	7'	15.0	-	WALL

Weyerhaeuser Notes

Weyerhaeuser warrants that the sizing of its products will be in accordance with Weyerhaeuser product design criteria and published design values. Weyerhaeuser expressly disclaims any other warranties related to the software. Use of this software is not intended to circumvent the need for a design professional as determined by the authority having jurisdiction. The designer of record, builder or framer is responsible to assure that this calculation is compatible with the overall project. Accessories (Rim Board, Blocking Panels and Squash Blocks) are not designed by this software. Products manufactured at Weyerhaeuser facilities are third-party certified to sustainable forestry standards. Weyerhaeuser Engineered Lumber Products have been evaluated by ICC-ES under evaluation reports ESR-1153 and ESR-1387 and/or tested in accordance with applicable ASTM standards. For current code evaluation reports, Weyerhaeuser product literature and installation details refer to www.weyerhaeuser.com/woodproducts/document-library.

The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

ForteWEB Software Operator	Job Notes
L RIED POPE PE, PLS LR POPE ENGINEERING INC (435) 628-1676 lrpope@lrpope.com	

Weyerhaeuser

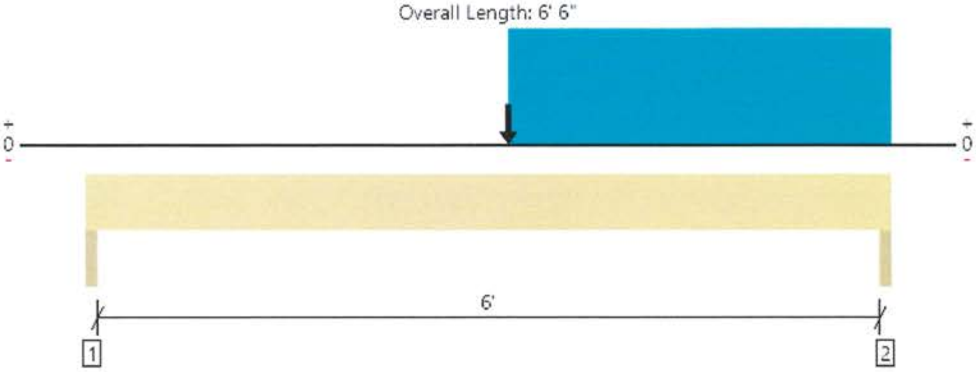
3/29/2021 5:14:16 AM UTC
ForteWEB v3.1, Engine: V8.1.6.2, Data: V8.0.1.0
File Name: MILAN LOT 2

MEMBER REPORT

PASSED

Level, RB-4

2 piece(s) 1 3/4" x 7 1/4" 2.0E Microllam® LVL



All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal.

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	3836 @ 6' 4 1/2"	7875 (3.00")	Passed (49%)	--	1.0 D + 1.0 Lr (All Spans)
Shear (lbs)	2761 @ 5' 7 3/4"	6027	Passed (46%)	1.25	1.0 D + 1.0 Lr (All Spans)
Moment (Ft-lbs)	4130 @ 3' 5"	6403	Passed (64%)	0.90	1.0 D (All Spans)
Live Load Defl. (in)	0.059 @ 3' 5"	0.313	Passed (L/999+)	--	1.0 D + 0.45 W + 0.75 L + 0.75 Lr (All Spans)
Total Load Defl. (in)	0.192 @ 3' 5"	0.417	Passed (L/391)	--	1.0 D + 0.45 W + 0.75 L + 0.75 Lr (All Spans)

System : Roof
Member Type : Drop Beam
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD
Member Pitch : 0/12

- Deflection criteria: LL (L/240) and TL (L/180).
- Top Edge Bracing (Lu): Top compression edge must be braced at 6' 6" o/c based on loads applied, unless detailed otherwise.
- Bottom Edge Bracing (Lu): Bottom compression edge must be braced at 6' 6" o/c based on loads applied, unless detailed otherwise.

Supports	Bearing Length			Loads to Supports (lbs)				Accessories
	Total	Available	Required	Dead	Roof Live	Wind	Total	
1 - Trimmer - DF	3.00"	3.00"	1.50"	1268	378	596	2242	None
2 - Trimmer - DF	3.00"	3.00"	1.50"	2549	1287	664	4500	None

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Roof Live (non-snow: 1.25)	Wind (1.60)	Comments
0 - Self Weight (PLF)	0 to 6' 6"	N/A	7.4	--	--	
1 - Uniform (PSF)	3' 5" to 6' 6" (Front)	27'	26.4	20.0	-	Default Load
2 - Point (lb)	3' 5" (Front)	N/A	1575	-	1260	

Weyerhaeuser Notes

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The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

ForteWEB Software Operator	Job Notes
L RIED POPE PE, PLS LR POPE ENGINEERING INC (435) 628-1676 lrpope@lrpope.com	

Weyerhaeuser

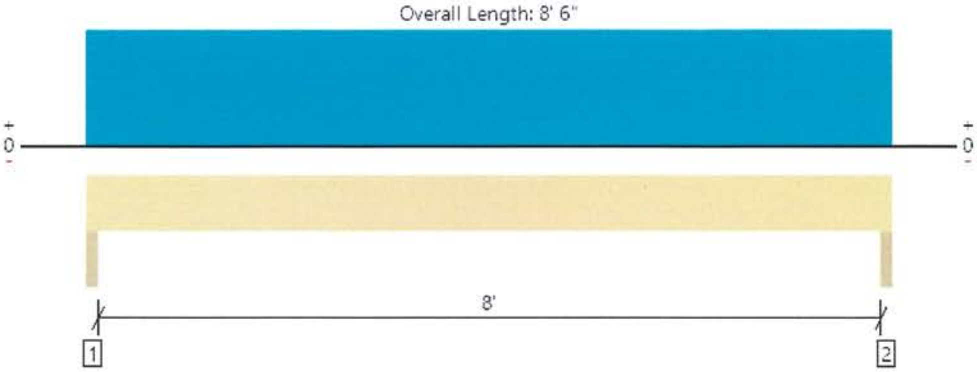
6/2/2020 12:08:46 AM UTC
ForteWEB v2.4, Engine: V8.0.1.5, Data: V7.3.2.0
File Name: MILAN LOT 2

MEMBER REPORT

PASSED

Level, RB-5

2 piece(s) 2 x 8 Douglas Fir-Larch No. 2



All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal.

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	1107 @ 1 1/2"	5625 (3.00")	Passed (20%)	--	1.0 D + 1.0 Lr (All Spans)
Shear (lbs)	884 @ 10 1/4"	3263	Passed (27%)	1.25	1.0 D + 1.0 Lr (All Spans)
Moment (Ft-lbs)	2216 @ 4' 3"	2957	Passed (75%)	1.25	1.0 D + 1.0 Lr (All Spans)
Live Load Defl. (in)	0.075 @ 4' 3"	0.412	Passed (L/999+)	--	1.0 D + 1.0 Lr (All Spans)
Total Load Defl. (in)	0.178 @ 4' 3"	0.550	Passed (L/556)	--	1.0 D + 1.0 Lr (All Spans)

System : Roof
Member Type : Drop Beam
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD
Member Pitch : 0/12

- Deflection criteria: LL (L/240) and TL (L/180).
- Top Edge Bracing (Lu): Top compression edge must be braced at 8' 6" o/c based on loads applied, unless detailed otherwise.
- Bottom Edge Bracing (Lu): Bottom compression edge must be braced at 8' 6" o/c based on loads applied, unless detailed otherwise.
- Applicable calculations are based on NDS.

	Bearing Length			Loads to Supports (lbs)			
Supports	Total	Available	Required	Dead	Roof Live	Total	Accessories
1 - Trimmer - DF	3.00"	3.00"	1.50"	639	468	1107	None
2 - Trimmer - DF	3.00"	3.00"	1.50"	639	468	1107	None

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Roof Live (non-snow: 1.25)	Comments
0 - Self Weight (PLF)	0 to 8' 6"	N/A	5.5	--	
1 - Uniform (PSF)	0 to 8' 6" (Front)	5' 6"	26.4	20.0	Default Load

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The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

ForteWEB Software Operator	Job Notes
L RIED POPE PE, PLS LR POPE ENGINEERING INC (435) 628-1676 lrpope@lrpope.com	

Weyerhaeuser

6/2/2020 12:12:12 AM UTC
ForteWEB v2.4, Engine: V8.0.1.5, Data: V7.3.2.0
File Name: MILAN LOT 2

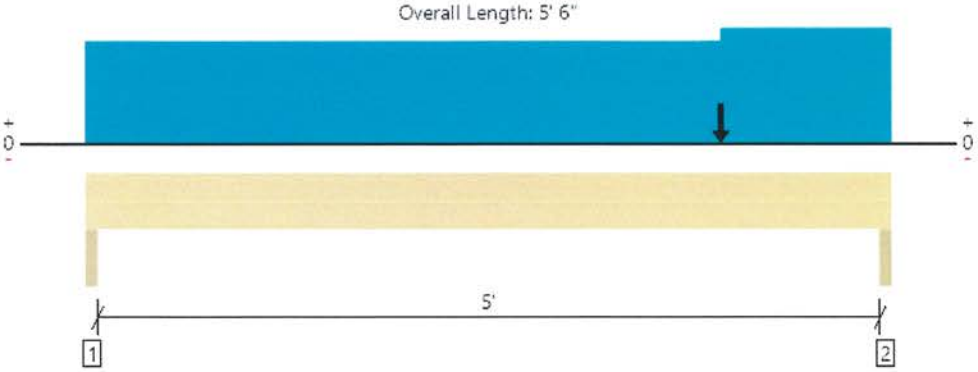
FORTE WEB

MEMBER REPORT

PASSED

Level, RB-6

2 piece(s) 1 3/4" x 9 1/2" 2.0E Microllam® LVL



All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal.

Design Results	**Actual @ Location**	**Allowed**	**Result**	**LDF**	**Load: Combination (Pattern)**
Member Reaction (lbs)	3990 @ 5' 4 1/2"	7875 (3.00")	Passed (51%)	--	1.0 D + 1.0 Lr (All Spans)
Shear (lbs)	2791 @ 4' 5 1/2"	7897	Passed (35%)	1.25	1.0 D + 1.0 Lr (All Spans)
Moment (Ft-lbs)	4272 @ 3' 3/16"	14719	Passed (29%)	1.25	1.0 D + 1.0 Lr (All Spans)
Live Load Defl. (in)	0.023 @ 2' 9 11/16"	0.262	Passed (L/999+)	--	1.0 D + 1.0 Lr (All Spans)
Total Load Defl. (in)	0.058 @ 2' 9 5/8"	0.350	Passed (L/999+)	--	1.0 D + 1.0 Lr (All Spans)

System : Roof
Member Type : Drop Beam
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD
Member Pitch : 0/12

- Deflection criteria: LL (L/240) and TL (L/180).
- Top Edge Bracing (Lu): Top compression edge must be braced at 5' 6" o/c based on loads applied, unless detailed otherwise.
- Bottom Edge Bracing (Lu): Bottom compression edge must be braced at 5' 6" o/c based on loads applied, unless detailed otherwise.

Supports	**Bearing Length**			**Loads to Supports (lbs)**			**Accessories**
	Total	**Available**	**Required**	**Dead**	**Roof Live**	**Total**	
1 - Trimmer - DF	3.00"	3.00"	1.50"	1854	1231	3085	None
2 - Trimmer - DF	3.00"	3.00"	1.52"	2369	1621	3990	None

Vertical Loads	**Location (Side)**	**Tributary Width**	**Dead (0.90)**	**Roof Live (non-snow: 1.25)**	**Comments**
0 - Self Weight (PLF)	0 to 5' 6"	N/A	9.7	--	
1 - Uniform (PSF)	0 to 4' 4" (Front)	20' 3"	26.4	20.0	Default Load
2 - Uniform (PSF)	4' 4" to 5' 6" (Front)	23'	26.4	20.0	
3 - Uniform (PSF)	0 to 5' 6" (Front)	5'	15.0	-	WALL
4 - Point (lb)	4' 4" (Front)	N/A	738	560	GIRDER

Weyerhaeuser Notes

ForteWEB Software Operator	**Job Notes**
L RIED POPE PE, PLS LR POPE ENGINEERING INC (435) 628-1676 lrpope@lrpope.com	

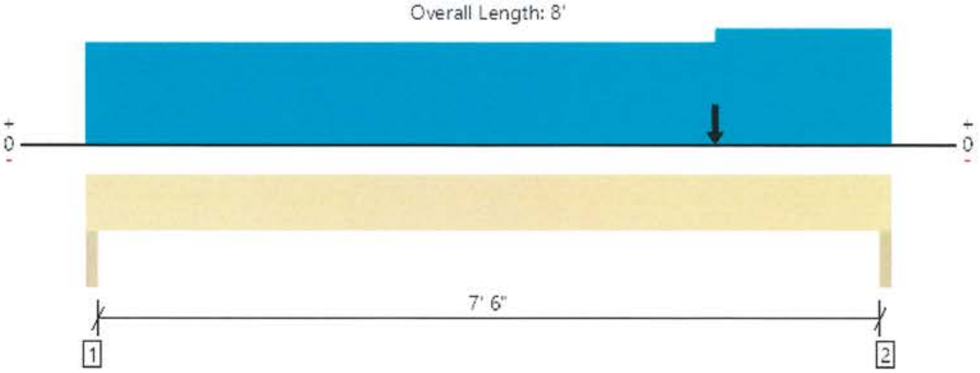
Weyerhaeuser

6/1/2020 8:58:31 PM UTC
ForteWEB v2.4, Engine: V8.0.1.5, Data: V7.3.2.0
File Name: MILAN LOT 2

MEMBER REPORT **PASSED**

Level, RB-7

2 piece(s) 1 3/4" x 9 1/2" 2.0E Microllam® LVL



All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal.

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	6295 @ 7' 10 1/2"	7875 (3.00")	Passed (80%)	--	1.0 D + 1.0 Lr (All Spans)
Shear (lbs)	4915 @ 6' 11 1/2"	7897	Passed (62%)	1.25	1.0 D + 1.0 Lr (All Spans)
Moment (Ft-lbs)	10361 @ 4' 4"	14719	Passed (70%)	1.25	1.0 D + 1.0 Lr (All Spans)
Live Load Defl. (in)	0.105 @ 4' 13/16"	0.387	Passed (L/887)	--	1.0 D + 1.0 Lr (All Spans)
Total Load Defl. (in)	0.263 @ 4' 13/16"	0.517	Passed (L/354)	--	1.0 D + 1.0 Lr (All Spans)

System : Roof
Member Type : Drop Beam
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD
Member Pitch : 0/12

- Deflection criteria: LL (L/240) and TL (L/180).
- Top Edge Bracing (Lu): Top compression edge must be braced at 8' o/c based on loads applied, unless detailed otherwise.
- Bottom Edge Bracing (Lu): Bottom compression edge must be braced at 8' o/c based on loads applied, unless detailed otherwise.

	Bearing Length			Loads to Supports (lbs)			
Supports	Total	Available	Required	Dead	Roof Live	Total	Accessories
1 - Trimmer - DF	3.00"	3.00"	1.93"	3044	2026	5070	None
2 - Trimmer - DF	3.00"	3.00"	2.40"	3784	2511	6295	None

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Roof Live (non-snow: 1.25)	Comments
0 - Self Weight (PLF)	0 to 8'	N/A	9.7	--	
1 - Uniform (PSF)	0 to 6' 3" (Front)	23' 5"	26.4	20.0	Default Load
2 - Uniform (PSF)	6' 3" to 8' (Front)	26' 9"	26.4	20.0	
3 - Uniform (PSF)	0 to 8' (Front)	5'	15.0	-	WALL
4 - Point (lb)	6' 3" (Front)	N/A	1060	674	GIRDER

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The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

ForteWEB Software Operator	Job Notes
L RIED POPE PE, PLS LR POPE ENGINEERING INC (435) 628-1676 lrpope@lrpope.com	

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6/1/2020 9:01:22 PM UTC
ForteWEB v2.4, Engine: V8.0.1.5, Data: V7.3.2.0
File Name: MILAN LOT 2

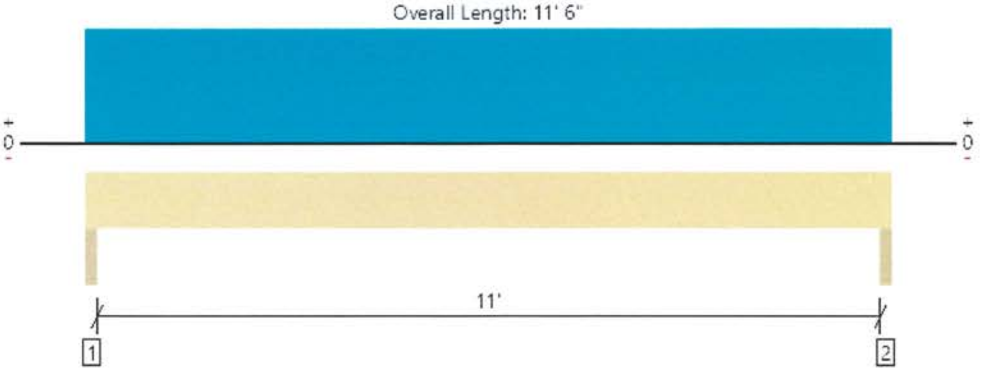
FORTE WEB

MEMBER REPORT

PASSED

Level, RB-8

2 piece(s) 1 3/4" x 9 1/2" 2.0E Microllam® LVL



All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal.

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	3187 @ 1 1/2"	7875 (3.00")	Passed (40%)	--	1.0 D + 1.0 Lr (All Spans)
Shear (lbs)	2610 @ 1' 1/2"	7897	Passed (33%)	1.25	1.0 D + 1.0 Lr (All Spans)
Moment (Ft-lbs)	8770 @ 5' 9"	14719	Passed (60%)	1.25	1.0 D + 1.0 Lr (All Spans)
Live Load Defl. (in)	0.182 @ 5' 9"	0.563	Passed (L/741)	--	1.0 D + 1.0 Lr (All Spans)
Total Load Defl. (in)	0.430 @ 5' 9"	0.750	Passed (L/314)	--	1.0 D + 1.0 Lr (All Spans)

System : Roof
Member Type : Drop Beam
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD
Member Pitch : 0/12

- Deflection criteria: LL (L/240) and TL (L/180).
- Top Edge Bracing (Lu): Top compression edge must be braced at 11' 6" o/c based on loads applied, unless detailed otherwise.
- Bottom Edge Bracing (Lu): Bottom compression edge must be braced at 11' 6" o/c based on loads applied, unless detailed otherwise.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Roof Live	Total	
1 - Trimmer - DF	3.00"	3.00"	1.50"	1836	1351	3187	None
2 - Trimmer - DF	3.00"	3.00"	1.50"	1836	1351	3187	None

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Roof Live (non-snow: 1.25)	Comments
0 - Self Weight (PLF)	0 to 11' 6"	N/A	9.7	--	
1 - Uniform (PSF)	0 to 11' 6" (Front)	11' 9"	26.4	20.0	Default Load

Weyerhaeuser Notes

The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

ForteWEB Software Operator	Job Notes
L RIED POPE PE, PLS LR POPE ENGINEERING INC (435) 628-1676 lrpope@lrpope.com	

Weyerhaeuser

6/2/2020 12:13:45 AM UTC
ForteWEB v2.4, Engine: V8.0.1.5, Data: V7.3.2.0
File Name: MILAN LOT 2

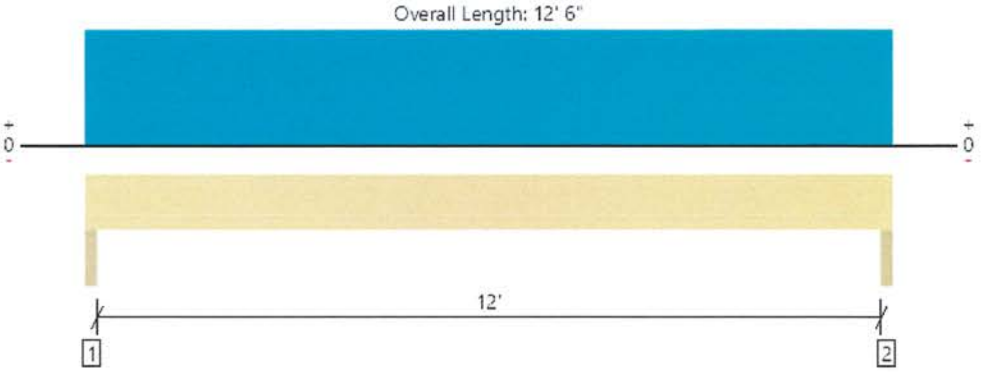
FORTE WEB

MEMBER REPORT

PASSED

Level, RB-9

2 piece(s) 1 3/4" x 9 1/2" 2.0E Microllam® LVL



All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal.

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	3754 @ 1 1/2"	7875 (3.00")	Passed (48%)	--	1.0 D + 1.0 Lr (All Spans)
Shear (lbs)	3129 @ 1' 1/2"	7897	Passed (40%)	1.25	1.0 D + 1.0 Lr (All Spans)
Moment (Ft-lbs)	11268 @ 6' 3"	14719	Passed (77%)	1.25	1.0 D + 1.0 Lr (All Spans)
Live Load Defl. (in)	0.275 @ 6' 3"	0.613	Passed (L/535)	--	1.0 D + 1.0 Lr (All Spans)
Total Load Defl. (in)	0.648 @ 6' 3"	0.817	Passed (L/227)	--	1.0 D + 1.0 Lr (All Spans)

System : Roof
Member Type : Drop Beam
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD
Member Pitch : 0/12

- Deflection criteria: LL (L/240) and TL (L/180).
- Top Edge Bracing (Lu): Top compression edge must be braced at 9' 3" o/c based on loads applied, unless detailed otherwise.
- Bottom Edge Bracing (Lu): Bottom compression edge must be braced at 12' 6" o/c based on loads applied, unless detailed otherwise.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Roof Live	Total	
1 - Trimmer - DF	3.00"	3.00"	1.50"	2161	1594	3755	None
2 - Trimmer - DF	3.00"	3.00"	1.50"	2161	1594	3755	None

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Roof Live (non-snow: 1.25)	Comments
0 - Self Weight (PLF)	0 to 12' 6"	N/A	9.7	--	
1 - Uniform (PSF)	0 to 12' 6" (Front)	12' 9"	26.4	20.0	Default Load

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ForteWEB Software Operator	Job Notes
L RIED POPE PE, PLS LR POPE ENGINEERING INC (435) 628-1676 lrpope@lrpope.com	

Weyerhaeuser

6/2/2020 12:14:37 AM UTC
ForteWEB v2.4, Engine: V8.0.1.5, Data: V7.3.2.0
File Name: MILAN LOT 2

MEMBER REPORT **PASSED**

Level, RB-10

2 piece(s) 1 3/4" x 11 7/8" 2.0E Microllam® LVL

All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal.

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	4498 @ 1 1/2"	7875 (3.00")	Passed (57%)	--	1.0 D + 1.0 Lr (All Spans)
Shear (lbs)	3751 @ 1' 2 7/8"	9871	Passed (38%)	1.25	1.0 D + 1.0 Lr (All Spans)
Moment (Ft-lbs)	16217 @ 7' 5 1/2"	22310	Passed (73%)	1.25	1.0 D + 1.0 Lr (All Spans)
Live Load Defl. (in)	0.291 @ 7' 5 1/2"	0.733	Passed (L/605)	--	1.0 D + 1.0 Lr (All Spans)
Total Load Defl. (in)	0.688 @ 7' 5 1/2"	0.978	Passed (L/256)	--	1.0 D + 1.0 Lr (All Spans)

System : Roof
Member Type : Drop Beam
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD
Member Pitch : 0/12

- Deflection criteria: LL (L/240) and TL (L/180).
- Top Edge Bracing (Lu): Top compression edge must be braced at 7' 8" o/c based on loads applied, unless detailed otherwise.
- Bottom Edge Bracing (Lu): Bottom compression edge must be braced at 14' 11" o/c based on loads applied, unless detailed otherwise.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Roof Live	Total	
1 - Trimmer - DF	3.00"	3.00"	1.71"	2596	1902	4498	None
2 - Trimmer - DF	3.00"	3.00"	1.71"	2596	1902	4498	None

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Roof Live (non-snow: 1.25)	Comments
0 - Self Weight (PLF)	0 to 14' 11"	N/A	12.1	--	
1 - Uniform (PSF)	0 to 14' 11" (Front)	12' 9"	26.4	20.0	Default Load

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The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

ForteWEB Software Operator	Job Notes
L RIED POPE PE, PLS LR POPE ENGINEERING INC (435) 628-1676 lrpope@lrpope.com	

6/2/2020 12:17:14 AM UTC
ForteWEB v2.4, Engine: V8.0.1.5, Data: V7.3.2.0
File Name: MILAN LOT 2

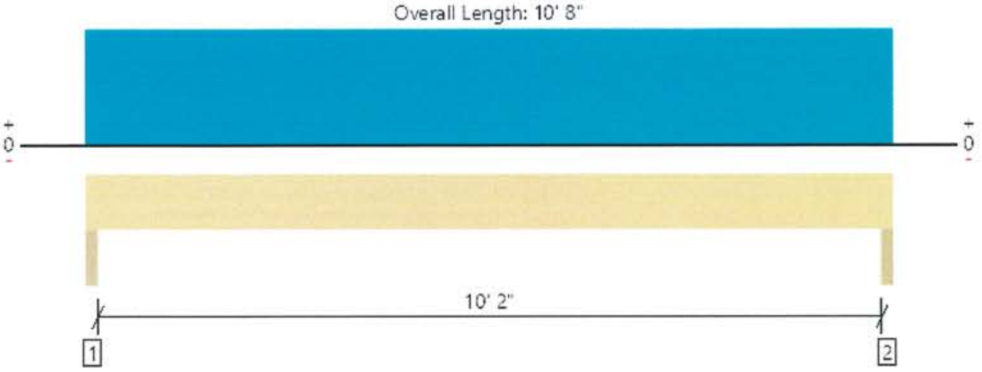
FORTE WEB

MEMBER REPORT

PASSED

Level, RB-11

2 piece(s) 1 3/4" x 9 1/2" 2.0E Microllam® LVL



All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal.

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	3204 @ 1 1/2"	7875 (3.00")	Passed (41%)	--	1.0 D + 1.0 Lr (All Spans)
Shear (lbs)	2578 @ 1' 1/2"	7897	Passed (33%)	1.25	1.0 D + 1.0 Lr (All Spans)
Moment (Ft-lbs)	8147 @ 5' 4"	14719	Passed (55%)	1.25	1.0 D + 1.0 Lr (All Spans)
Live Load Defl. (in)	0.147 @ 5' 4"	0.521	Passed (L/850)	--	1.0 D + 1.0 Lr (All Spans)
Total Load Defl. (in)	0.346 @ 5' 4"	0.694	Passed (L/361)	--	1.0 D + 1.0 Lr (All Spans)

System : Roof
Member Type : Drop Beam
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD
Member Pitch : 0/12

- Deflection criteria: LL (L/240) and TL (L/180).
- Top Edge Bracing (Lu): Top compression edge must be braced at 10' 8" o/c based on loads applied, unless detailed otherwise.
- Bottom Edge Bracing (Lu): Bottom compression edge must be braced at 10' 8" o/c based on loads applied, unless detailed otherwise.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Roof Live	Total	
1 - Trimmer - DF	3.00"	3.00"	1.50"	1844	1360	3204	None
2 - Trimmer - DF	3.00"	3.00"	1.50"	1844	1360	3204	None

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Roof Live (non-snow: 1.25)	Comments
0 - Self Weight (PLF)	0 to 10' 8"	N/A	9.7	--	
1 - Uniform (PSF)	0 to 10' 8" (Front)	12' 9"	26.4	20.0	Default Load

Weyerhaeuser Notes

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The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

ForteWEB Software Operator	Job Notes
L RIED POPE PE, PLS LR POPE ENGINEERING INC (435) 628-1676 lrpope@lrpope.com	

Weyerhaeuser

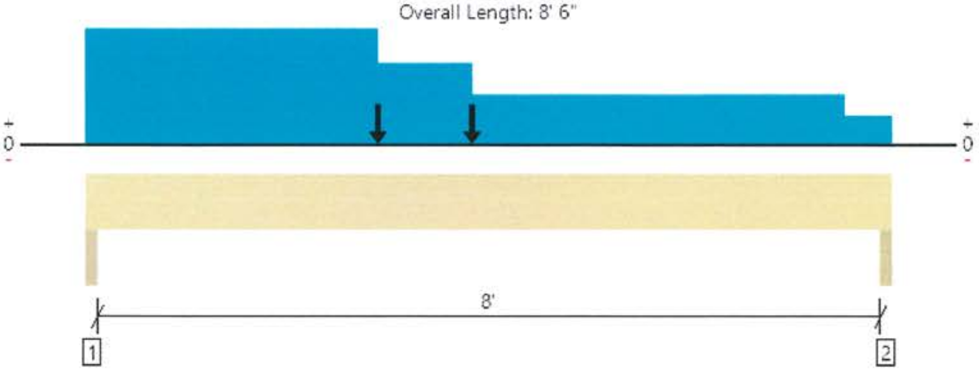
6/2/2020 12:18:04 AM UTC
ForteWEB v2.4, Engine: V8.0.1.5, Data: V7.3.2.0
File Name: MILAN LOT 2

FORTE WEB

MEMBER REPORT **PASSED**

Level, RB-12

2 piece(s) 1 3/4" x 11 7/8" 2.0E Microllam® LVL



All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal.

Design Results	**Actual @ Location**	**Allowed**	**Result**	**LDF**	**Load: Combination (Pattern)**
Member Reaction (lbs)	7246 @ 1 1/2"	7875 (3.00")	Passed (92%)	--	1.0 D + 1.0 Lr (All Spans)
Shear (lbs)	5659 @ 1' 2 7/8"	9871	Passed (57%)	1.25	1.0 D + 1.0 Lr (All Spans)
Moment (Ft-lbs)	15752 @ 4' 1/4"	22310	Passed (71%)	1.25	1.0 D + 1.0 Lr (All Spans)
Live Load Defl. (in)	0.093 @ 4' 1"	0.412	Passed (L/999+)	--	1.0 D + 1.0 Lr (All Spans)
Total Load Defl. (in)	0.227 @ 4' 1"	0.550	Passed (L/436)	--	1.0 D + 1.0 Lr (All Spans)

System : Roof
Member Type : Drop Beam
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD
Member Pitch : 0/12

- Deflection criteria: LL (L/240) and TL (L/180).
- Top Edge Bracing (Lu): Top compression edge must be braced at 8' 1" o/c based on loads applied, unless detailed otherwise.
- Bottom Edge Bracing (Lu): Bottom compression edge must be braced at 8' 6" o/c based on loads applied, unless detailed otherwise.

	Bearing Length			**Loads to Supports (lbs)**			
Supports	**Total**	**Available**	**Required**	**Dead**	**Roof Live**	**Total**	**Accessories**
1 - Trimmer - DF	3.00"	3.00"	2.76"	4280	2967	7247	None
2 - Trimmer - DF	3.00"	3.00"	1.83"	2893	1915	4808	None

Vertical Loads	**Location (Side)**	**Tributary Width**	**Dead (0.90)**	**Roof Live (non-snow: 1.25)**	**Comments**
0 - Self Weight (PLF)	0 to 8' 6"	N/A	12.1	--	
1 - Uniform (PSF)	0 to 4' 1" (Front)	12' 6"	26.4	20.0	UPPER ROOF
2 - Uniform (PSF)	4' 1" to 8' (Front)	5'	26.4	20.0	UPPER ROOF
3 - Uniform (PSF)	0 to 8' 6" (Front)	5'	15.0	-	WALL
4 - Point (lb)	4' 1" (Front)	N/A	1384	1050	GIRDER
5 - Uniform (PSF)	0 to 3' 1" (Front)	13' 3"	26.4	20.0	LOWER ROOF
6 - Uniform (PSF)	3' 1" to 8' 6" (Front)	5'	26.4	20.0	LOWER ROOF
7 - Point (lb)	3' 1" (Front)	N/A	1397	1060	GIRDER

Weyerhaeuser Notes

ForteWEB Software Operator	**Job Notes**
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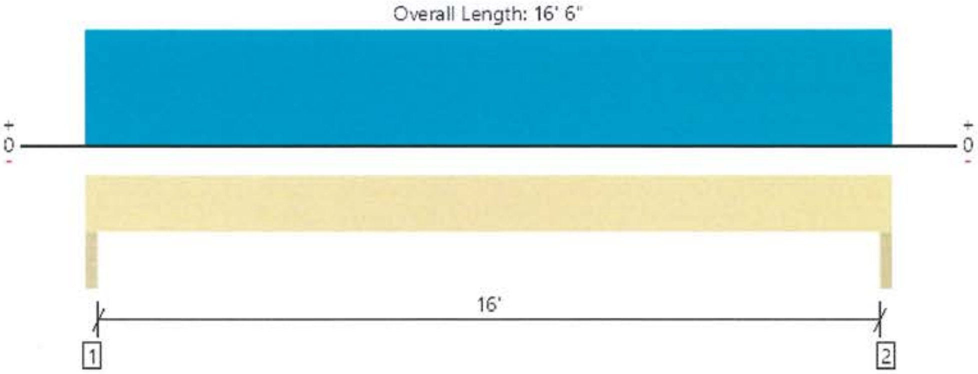
6/1/2020 9:12:42 PM UTC
ForteWEB v2.4, Engine: V8.0.1.5, Data: V7.3.2.0
File Name: MILAN LOT 2

Level, RB-13

2 piece(s) 1 3/4" x 9 1/2" 2.0E Microllam® LVL



All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal.

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	2374 @ 1 1/2"	7875 (3.00")	Passed (30%)	--	1.0 D + 1.0 Lr (All Spans)
Shear (lbs)	2075 @ 1' 1/2"	7897	Passed (26%)	1.25	1.0 D + 1.0 Lr (All Spans)
Moment (Ft-lbs)	9500 @ 8' 3"	14719	Passed (65%)	1.25	1.0 D + 1.0 Lr (All Spans)
Live Load Defl. (in)	0.390 @ 8' 3"	0.813	Passed (L/500)	--	1.0 D + 1.0 Lr (All Spans)
Total Load Defl. (in)	0.936 @ 8' 3"	1.083	Passed (L/208)	--	1.0 D + 1.0 Lr (All Spans)

System : Roof
Member Type : Drop Beam
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD
Member Pitch : 0/12

- Deflection criteria: LL (L/240) and TL (L/180).
- Top Edge Bracing (Lu): Top compression edge must be braced at 12' o/c based on loads applied, unless detailed otherwise.
- Bottom Edge Bracing (Lu): Bottom compression edge must be braced at 16' 6" o/c based on loads applied, unless detailed otherwise.

	Bearing Length			Loads to Supports (lbs)			
Supports	Total	Available	Required	Dead	Roof Live	Total	Accessories
1 - Trimmer - DF	3.00"	3.00"	1.50"	1384	990	2374	None
2 - Trimmer - DF	3.00"	3.00"	1.50"	1384	990	2374	None

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Roof Live (non-snow: 1.25)	Comments
0 - Self Weight (PLF)	0 to 16' 6"	N/A	9.7	--	
1 - Uniform (PSF)	0 to 16' 6" (Front)	6'	26.4	20.0	Default Load

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The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

ForteWEB Software Operator	Job Notes
L RIED POPE PE, PLS LR POPE ENGINEERING INC (435) 628-1676 lrpope@lrpope.com	

Weyerhaeuser

ASD Wood Member Design v7.3.1 (2-17-17)

PROJECT MILAN LOT 2 (RB-14)

Member Dimensions Beam ● Joist ○

	Cantilever	Span	Cantilever	Total Length
Span =		4.50		4.50
Unbraced length =		4.00	ft	
Number of plys =		2		
Member width, b =		1.5	in ☐ Custom width	
Member depth, d =		9.25	in ☐ Custom depth	
Orientation =		Strong		

Member Material Properties

Lumber type = Solid Sawn
Species = Douglas Fir - Larch
Grade = No. 2
Member unit weight = 34 pcf
Bearing length @ support A = 3.00 in ≥ 1.52 in
Bearing length @ support B = 3.00 in ≥ 1.5 in

Loads

Roof DL = 25 psf
Roof Lr = 20 psf
Snow, S = 15 psf
Rain, R = 5 psf
Floor DL = 15 psf
Floor LL = 40 psf

*****Pos. Bending stress controls member design (96%)*****

Unfactored Load Reactions

Load type	R_A	R_B	
D =	1617	1383	*lbs*
L =	0	0	*lbs*
Lr =	1216	1039	*lbs*
S =	0	0	*lbs*
R =	0	0	*lbs*
W =	0	0	*lbs*
E =	0	0	*lbs*

Point Loads

	P_L	P_D	P_T	a	LL Type	Load Description
P_1 =	1250	1647	2897	1.58	Roof	GIRDER TRUSS
P_2 =			0			
P_3 =			0			
P_4 =			0			
P_5 =			0			
P_6 =			0			

Uniform Loads **Roof slope =** 4.00 :12 **Member slope =** :12

Live, psf	Dead, psf	Trib. Width		W_L	W_D	W_T	Start @	End @	LL Type	Load Description
20	25	5.00	W_1 =	100	132	232	0.00	1.58	Roof	
20	25	14.50	W_2 =	290	382	672	1.58	4.50	Roof	
			W_3 =			0	0.00			
			W_4 =			0				
			W_5 =			0				
			W_6 =			0				

Triangular Loads (Starting or ending load must be 0)

	Start W_L	Start W_D	End W_L	End W_D	Start W_T	End W_T	Start @	End @	LL Type	Load Description
T_1 =					0	0				
T_2 =					0	0				
T_3 =					0	0				
T_4 =					0	0				

Member Shear Design ***Member design controlled by D+(Lr or S or R)***

Fv =	180	psi
$Fv' = Fv^*(C_DC_MC_tC_i)$ =	225	psi
Max V =	2833	lbs
Design V =	2620	lbs
A = b*d =	27.75	in^2
fv = 1.5*V/A =	153	psi
	OK	

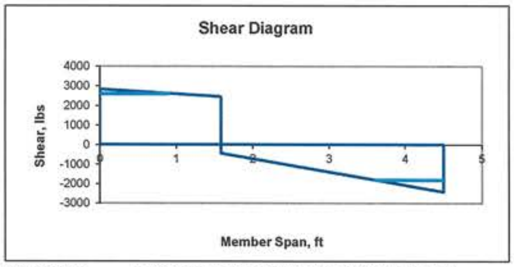
Adjustment Factors

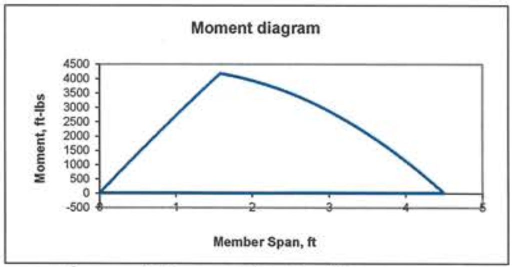
C_D =	1.25	For roof live load
C_M =	1.00	For MC < 19%
C_t =	1.00	Insulated against 100+ F
C_F =	1.10	Size increase
C_V =		Volume Factor
C_{fu} =	1.00	Narrow face loaded
C_i =	1.00	No incising
C_r =	1.00	Not a repetitive member
C_f =	1.00	Rectangular shaped
C_T =	1.00	Buckling stiffness factor
C_b =	1.00	Bearing area factor

Member Bending Design ***Member design controlled by D+(Lr or S or R)***

	Fb^+	Fb^-	
Fb =	900	900	psi
$Fb^* = Fb^*(C_DC_MC_tC_FC_iC_rC_f)$ =	1238	1238	psi
Emin =	580000	580000	psi
Emin' = Emin*(CMCtCiCT) =	580000	580000	psi
unbraced length, lu =	4.00	4.00	ft
lu/d =	5.19	5.19	
le =	99	99	in
$R_B = (le^*d/b^2)^{1/2}$ =	10.08	10.08	***≤ 50, OK***
$F_{bE} = 1.20^*E'min/(R_B)^2$ =	6849	6849	psi
C_L =	0.989	0.989	
$Fb' = Fb^*(C_DC_MC_tC_LC_FC_VC_{fu}C_iC_rC_f)$ =	1224	1224	psi

	+ Moment	**- Moment**	
Max moment, M =	4178		lb-ft
$S = bd^2/6$ =	42.78	42.78	in^3
fb = M/S =	1172	0	psi
	OK	***OK***	





Member Bearing **Member design controlled by D+(Lr or S or R)**

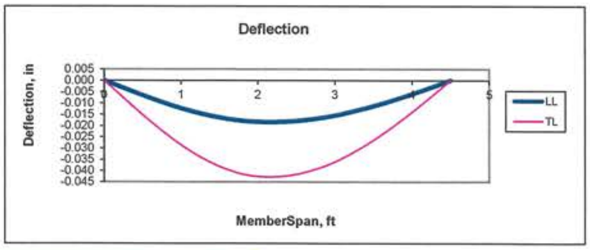
$Fc_\perp$ = 625 psi
$F'c_\perp = Fc_\perp^*(C_MC_tC_iC_b)$ = 625 psi

	C_b	P, lbs	A, in^2	$fc_\perp$ = P/A		
Support @ A =	1.00	2833	9.00	315	psi	***OK***
Support @ B =	1.00	2422	9.00	269	psi	***OK***

Member Deflection

Moment of Inertia, $I = bd^3/12$ = 197.863 in^4
E = 1600000 psi
$E' = E^*(C_MC_tC_i)$ = 1600000 psi

Mid Span Deflection					
Loading	**$Ratio_{allow}$**	$\Delta_{allowed}$	Δ_{actual}	$Ratio_{actual}$	**Check**
Δ_{LL}	360	0.150	0.018	L/2936	***OK***
Δ_{TL}	240	0.225	0.043	L/1261	***OK***
Cantilever Deflection					
Loading	**$Ratio_{allow}$**	$\Delta_{allowed}$	Δ_{actual}	$Ratio_{actual}$	**Check**
Δ_{LL}	180	0.000	0.000	N/A	***OK***
Δ_{TL}	120	0.000	0.000	N/A	***OK***



(2) 1.5" x 9.25" Douglas Fir - Larch No. 2

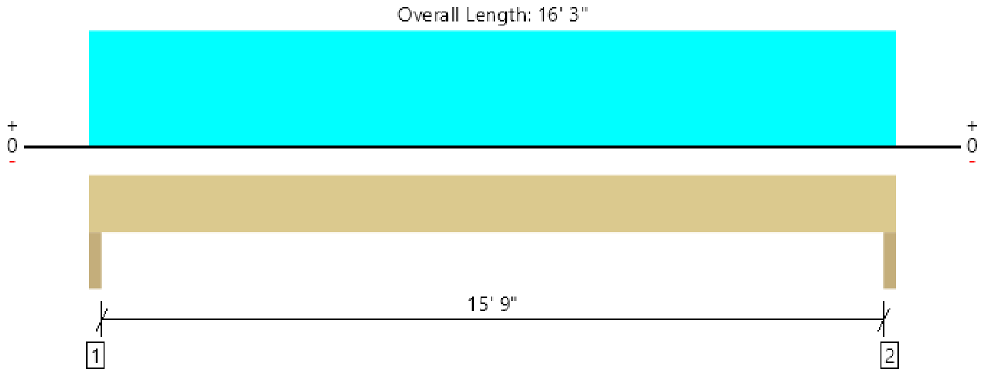
Date: 6/1/20 2:20 PM

MEMBER REPORT

PASSED

Level, RB-14

2 piece(s) 1 3/4" x 11 7/8" 2.0E Microllam® LVL



All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal.

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	4806 @ 1 1/2"	7875 (3.00")	Passed (61%)	--	1.0 D + 1.0 Lr (All Spans)
Shear (lbs)	4073 @ 1' 2 7/8"	9871	Passed (41%)	1.25	1.0 D + 1.0 Lr (All Spans)
Moment (Ft-lbs)	18929 @ 8' 1 1/2"	22310	Passed (85%)	1.25	1.0 D + 1.0 Lr (All Spans)
Live Load Defl. (in)	0.400 @ 8' 1 1/2"	0.800	Passed (L/481)	--	1.0 D + 1.0 Lr (All Spans)
Total Load Defl. (in)	0.945 @ 8' 1 1/2"	1.067	Passed (L/203)	--	1.0 D + 1.0 Lr (All Spans)

System : Roof
Member Type : Drop Beam
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD
Member Pitch : 0/12

- Deflection criteria: LL (L/240) and TL (L/180).
- Allowed moment does not reflect the adjustment for the beam stability factor.

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Roof Live	Total	
1 - Trimmer - DF	3.00"	3.00"	1.83"	2775	2031	4806	None
2 - Trimmer - DF	3.00"	3.00"	1.83"	2775	2031	4806	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	5' 9" o/c	
Bottom Edge (Lu)	16' 3" o/c	

- Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Roof Live (non-snow: 1.25)	Comments
0 - Self Weight (PLF)	0 to 16' 3"	N/A	12.1	--	
1 - Uniform (PSF)	0 to 16' 3" (Front)	12' 6"	26.4	20.0	Default Load

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The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

ForteWEB Software Operator	Job Notes
L RIED POPE PE, PLS LR POPE ENGINEERING INC (435) 628-1676 lrpope@lrpope.com	

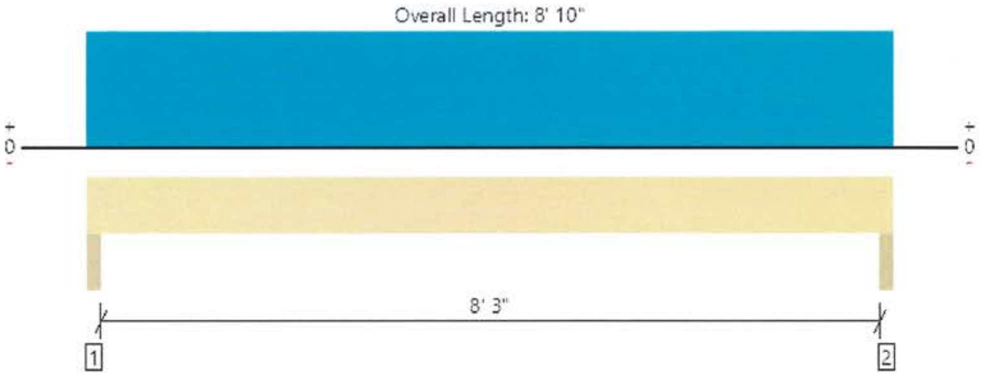
3/29/2021 5:24:05 AM UTC
ForteWEB v3.1, Engine: V8.1.6.2, Data: V8.0.1.0
File Name: MILAN LOT 2

MEMBER REPORT

PASSED

Level, FB-1

2 piece(s) 1 3/4" x 9 1/2" 2.0E Microllam® LVL



All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal.

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	3477 @ 2"	9188 (3.50")	Passed (38%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	2624 @ 1' 1"	6318	Passed (42%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	7109 @ 4' 5"	11775	Passed (60%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.109 @ 4' 5"	0.283	Passed (L/935)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.209 @ 4' 5"	0.425	Passed (L/487)	--	1.0 D + 1.0 L (All Spans)

System : Floor
Member Type : Drop Beam
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

- Deflection criteria: LL (L/360) and TL (L/240).
- Top Edge Bracing (Lu): Top compression edge must be braced at 8' 10" o/c based on loads applied, unless detailed otherwise.
- Bottom Edge Bracing (Lu): Bottom compression edge must be braced at 8' 10" o/c based on loads applied, unless detailed otherwise.

	Bearing Length			Loads to Supports (lbs)				
Supports	Total	Available	Required	Dead	Floor Live	Roof Live	Total	Accessories
1 - Trimmer - DF	3.50"	3.50"	1.50"	1666	1811	213	3690	None
2 - Trimmer - DF	3.50"	3.50"	1.50"	1666	1811	213	3690	None

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Roof Live (non-snow: 1.25)	Comments
0 - Self Weight (PLF)	0 to 8' 10"	N/A	9.7	--	--	
1 - Uniform (PSF)	0 to 8' 10" (Front)	10' 3"	15.0	40.0	-	Default Load
2 - Uniform (PSF)	0 to 8' 10" (Front)	2' 5"	26.4	-	20.0	
3 - Uniform (PLF)	0 to 8' 10" (Front)	N/A	150.0	-	-	

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
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6/2/2020 12:22:19 AM UTC
ForteWEB v2.4, Engine: V8.0.1.5, Data: V7.3.2.0
File Name: MILAN LOT 2

ASD Wood Member Design v7.3.1 (2-17-17)

PROJECT MILAN LOT 2(FB-2) SEISMIC FORCES FROM SHEAR WALL

Member Dimensions Beam ● Joist ○

	Cantilever	Span	Cantilever	Total Length
Span =		6.50		6.50

Unbraced length = 6.00 ft
Number of plys = 2
Member width, b = 1.75 in ☐ Custom width
Member depth, d = 9.5 in ☐ Custom depth
Orientation = Strong

Member Material Properties

Lumber type = Engineered
Type = LVL
Grade = 1.9E
Member unit weight = 42 pcf
Bearing length @ support A = 3.00 in ≥ 2.3 in
Bearing length @ support B = 3.00 in ≥ 1.5 in

Loads

Roof DL = 25 psf
Roof Lr = 20 psf
Snow, S = 15 psf
Rain, R = 5 psf
Floor DL = 15 psf
Floor LL = 40 psf

*****LL deflection controls member design (78%)*****

Unfactored Load Reactions

Load type	R_A	R_B	
D =	1189	811	*lbs*
L =	130	130	*lbs*
Lr =	508	222	*lbs*
S =	0	0	*lbs*
R =	0	0	*lbs*
W =	0	0	*lbs*
E =	5486	2904	*lbs*

Point Loads

	P_L	P_D	P_T	a	LL Type	Load Description
P_1 =	550	725	1275	2.25	Roof	ROOF HDR REACTION
P_2 =	8390		8390	2.25	Seismic	SHEARWALL CHORD FORCE
P_3 =			0			
P_4 =			0			
P_5 =			0			
P_6 =			0			

Uniform Loads **Roof slope =** 4.00 :12 **Member slope =** :12

Live, psf	Dead, psf	Trib. Width		W_L	W_D	W_T	Start @	End @	LL Type	Load Description
20	25	4.00	W_1 =	80	105	185	0.00	2.25	Roof	
0	15	9.00	W_2 =	0	135	135	0.00	6.50	Floor	WALL
40	15	1.00	W_3 =	40	15	55	0.00	6.50	Floor	
			W_4 =			0				
			W_5 =			0				
			W_6 =			0				

Triangular Loads (Starting or ending load must be 0)

	Start W_L	Start W_D	End W_L	End W_D	Start W_T	End W_T	Start @	End @	LL Type	Load Description
T_1 =					0	0				
T_2 =					0	0				
T_3 =					0	0				
T_4 =					0	0				

Member Shear Design ***Member design controlled by D+(0.6W or 0.7E)***

Fv = 285 psi
Fv' = Fv*($C_D C_M C_t C_i$) = 456 psi
Max V = 5029 lbs
Design V = 4786 lbs
A = b*d = 33.25 in^2
fv = 1.5*V/A = 227 psi
OK

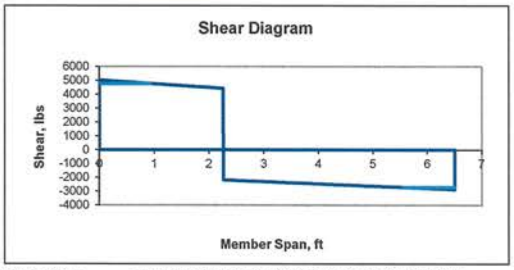
Adjustment Factors

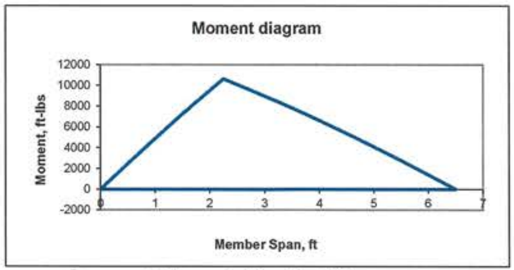
C_D =	1.60	For wind or seismic loads
C_M =	1.00	For MC < 19%
C_t =	1.00	Insulated against 100+ F
C_F =	▨	No size increase
C_V =	1.03	Volume Factor
C_{fu} =	▨	Narrow face loaded
C_i =	▨	No incising
C_r =	1.00	Not a repetitive member
C_f =	▨	Rectangular shaped
C_T =	▨	Buckling stiffness factor
C_b =	1.00	Bearing area factor

Member Bending Design ***Member design controlled by D+(0.6W or 0.7E)***

	Fb^+	Fb^-	
Fb =	2600	2600	psi
Fb* = Fb*($C_D C_M C_t C_F C_i C_r C_f$) =	4160	4160	psi
Emin =	965710	965710	psi
Emin' = Emin*(CMCtCiCT) =	965710	965710	psi
unbraced length, lu =	6.00	6.00	ft
lu/d =	7.58	7.58	
le =	146	146	in
$R_B = (le*d/b^2)^{1/2}$ =	10.64	10.64	*≤ 50, OK*
$F_{bE} = 1.20*E'min/(R_B)^2$ =	10245	10245	psi
C_L =	0.969	0.969	
Fb' = Fb*($C_D C_M C_t C_L C_F C_V C_{fu} C_i C_r C_f$) =	4159	4159	psi

	+ Moment	- Moment	
Max moment, M =	10640		lb-ft
$S = bd^2/6$ =	52.65	52.65	in^3
fb = M/S =	2425	0	psi
	OK	*OK*	





Member Bearing **Member design controlled by D+(0.6W or 0.7E)**

Fc⊥ = 625 psi
F'c⊥ = Fc⊥*($C_M C_t C_i C_b$) = 625 psi

	C_b	P, lbs	A, in^2	fc⊥ = P/A		
Support @ A =	1.00	5029	10.50	479	psi	*OK*
Support @ B =	1.00	2844	10.50	271	psi	*OK*

Member Deflection

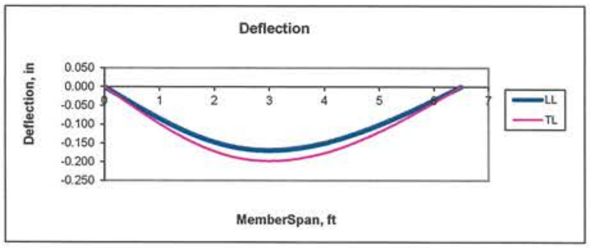
Moment of Inertia, $I = bd^3/12$ = 250.068 in^4
E = 1900000 psi
E' = E*($C_M C_t C_i$) = 1900000 psi

Mid Span Deflection

Loading	$Ratio_{allow}$	$\Delta_{allowed}$	Δ_{actual}	$Ratio_{actual}$	Check
Δ_{LL}	360	0.217	0.169	L/461	*OK*
Δ_{TL}	240	0.325	0.198	L/394	*OK*

Cantilever Deflection

Loading	$Ratio_{allow}$	$\Delta_{allowed}$	Δ_{actual}	$Ratio_{actual}$	Check
Δ_{LL}	180	0.000	0.000	N/A	*OK*
Δ_{TL}	120	0.000	0.000	N/A	*OK*



(2) 1.75" x 9.5" LVL 1.9E

Date: 6/1/20 2:07 PM

ASD Wood Member Design v7.3.1 (2-17-17)

PROJECT MILAN LOT 2(FB-2) WIND FORCES FROM SHEAR WALL

Member Dimensions Beam ● Joist ○

	Cantilever	Span	Cantilever	Total Length
Span =		6.50		6.50

Unbraced length = 6.00 ft
Number of plys = 2
Member width, b = 1.75 in ☐ Custom width
Member depth, d = 9.5 in ☐ Custom depth
Orientation = Strong

Member Material Properties

Lumber type = Engineered
Type = LVL
Grade = 1.9E
Member unit weight = 42 pcf
Bearing length @ support A = 3.00 in ≥ 1.5 in
Bearing length @ support B = 3.00 in ≥ 1.5 in

Loads

Roof DL = 25 psf
Roof Lr = 20 psf
Snow, S = 15 psf
Rain, R = 5 psf
Floor DL = 15 psf
Floor LL = 40 psf

*****Bearing stress controls member design (43%)*****

Unfactored Load Reactions

Load type	R_A	R_B	
D =	1189	811	*lbs*
L =	130	130	*lbs*
Lr =	508	222	*lbs*
S =	0	0	*lbs*
R =	0	0	*lbs*
W =	2592	1372	*lbs*
E =	0	0	*lbs*

Point Loads

	P_L	P_D	P_T	a	LL Type	Load Description
P_1 =	550	725	1275	2.25	Roof	ROOF HDR REACTION
P_2 =	3964		3964	2.25	Wind	SHEARWALL CHORD FORCE
P_3 =			0			
P_4 =			0			
P_5 =			0			
P_6 =			0			

Uniform Loads Roof slope = 4.00 :12 Member slope = :12

Live, psf	Dead, psf	Trib. Width		W_L	W_D	W_T	Start @	End @	LL Type	Load Description
20	25	4.00	W_1 =	80	105	185	0.00	2.25	Roof	
0	15	9.00	W_2 =	0	135	135	0.00	6.50	Floor	WALL
40	15	1.00	W_3 =	40	15	55	0.00	6.50	Floor	
			W_4 =			0				
			W_5 =			0				
			W_6 =			0				

Triangular Loads (Starting or ending load must be 0)

	Start W_L	Start W_D	End W_L	End W_D	Start W_T	End W_T	Start @	End @	LL Type	Load Description
T_1 =					0	0				
T_2 =					0	0				
T_3 =					0	0				
T_4 =					0	0				

Member Shear Design ***Member design controlled by D+0.75L+0.75(Lr or S or R)***

Fv = 285 psi
$Fv' = Fv^*(C_DC_MC_tC_i)$ = 456 psi
Max V = 2834 lbs
Design V = 2509 lbs
A = b*d = 33.25 in^2
fv = 1.5*V/A = 128 psi
OK

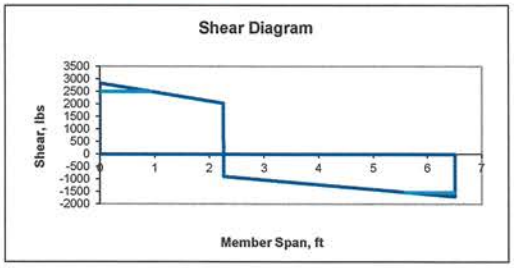
Adjustment Factors

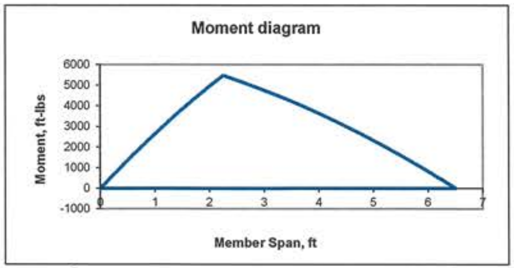
C_D =	1.60	For wind or seismic loads
C_M =	1.00	For MC < 19%
C_t =	1.00	Insulated against 100+ F
C_F =		No size increase
C_V =	1.03	Volume Factor
C_{fu} =		Narrow face loaded
C_i =		No incising
C_r =	1.00	Not a repetitive member
C_f =		Rectangular shaped
C_T =		Buckling stiffness factor
C_b =	1.00	Bearing area factor

Member Bending Design ***Member design controlled by D+0.75L+0.75(Lr or S or R)+0.***

	Fb^+	Fb^-	
Fb =	2600	2600	psi
$Fb^* = Fb^*(C_DC_MC_tC_FC_iC_rC_f)$ =	4160	4160	psi
Emin =	965710	965710	psi
Emin' = Emin*(CMCtCiCT) =	965710	965710	psi
unbraced length, lu =	6.00	6.00	ft
lu/d =	7.58	7.58	
le =	146	146	in
$R_B = (le^*d/b^2)^{1/2}$ =	10.64	10.64	***≤ 50, OK***
$F_{bE} = 1.20^*E'min/(R_B)^2$ =	10245	10245	psi
C_L =	0.969	0.969	
$Fb' = Fb^*(C_DC_MC_tC_LC_FC_VC_{fu}C_iC_rC_f)$ =	4159	4159	psi

	+ Moment	**- Moment**	
Max moment, M =	5476		lb-ft
$S = bd^2/6$ =	52.65	52.65	in^3
fb = M/S =	1248	0	psi
	OK	***OK***	





Member Bearing **Member design controlled by D+0.75L+0.75(Lr or S or R)+0.75(0.6*W)**

Fc⊥ = 625 psi
F'c⊥ = Fc⊥*($C_MC_tC_iC_b$) = 625 psi

	C_b	P, lbs	A, in^2	fc⊥ = P/A		
Support @ A =	1.00	2834	10.50	270	psi	***OK***
Support @ B =	1.00	1692	10.50	161	psi	***OK***

Member Deflection

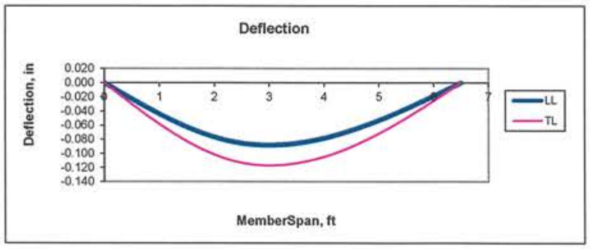
Moment of Inertia, I = $bd^3/12$ = 250.068 in^4
E = 1900000 psi
E' = E*($C_MC_tC_i$) = 1900000 psi

Mid Span Deflection

Loading	$Ratio_{allow}$	$\Delta_{allowed}$	Δ_{actual}	$Ratio_{actual}$	Check
Δ_{LL}	360	0.217	0.088	L/887	***OK***
Δ_{TL}	240	0.325	0.117	L/667	***OK***

Cantilever Deflection

Loading	$Ratio_{allow}$	$\Delta_{allowed}$	Δ_{actual}	$Ratio_{actual}$	Check
Δ_{LL}	180	0.000	0.000	N/A	***OK***
Δ_{TL}	120	0.000	0.000	N/A	***OK***



(2) 1.75" x 9.5" LVL 1.9E

Date: 6/1/20 2:08 PM

ASD Wood Column Design v4.7.1 (2-17-17)

Project: MILAN LOT 2 TRIMMERS FOR FB-2

Date: 6/1/2020 14:11

Allowable Stresses for 2X4 nominal 8 foot DF #2 wood column

Column specifications & dimensions

Lumber Type = Solid Sawn
Species = Douglas Fir - Larch
Grade = No. 2
Plies = 2
Ply width, b = 1.5 in (x-axis)
Ply depth, d = 3.5 in (y-axis)
Column type = Built up - nailed
Support Type = Supported

	X-Axis	Y-Axis	
k =	1.0	1.0	
L =	8.00	1.00	ft

Column Material Properties

Fb = 900 psi
Fc = 1350 psi
Fv = 180 psi
E = 1600000 psi
Emin = 580000 psi
Fc⊥ = 625 psi

Point Loads

Roof DL = 1189 lbs
Lr = 508 lbs
S = lbs
R = lbs
Floor DL = 130 lbs
LL = lbs
W = 2592 lbs
E = 5486 lbs
ex = in
ey = in
Col. Unit wt = 34 pcf

Uniform Loads

Roof DL = 25 psf
Lr = 20 psf
S = 15 psf
R = 5 psf
Floor DL = 15 psf
LL = 40 psf
W = psf
E = psf
Roof Slope = :12
Roof Trib. Area = ft^2
Floor Trib. Area = ft^2
Wind Trib. Area = ft^2
Seismic Trib. Area = ft^2

Lateral Loads

Wx = plf
Ex = plf
Wy = plf
Ey = plf

Default Load Duration Factors

Dead = 0.90
Floor = 1.00
Roof = 1.25
Snow = 1.15
Rain = 1.25
Wind = 1.60
Seismic = 1.60

Compression - governed by D+(0.6W or 0.7E)

Allowable compression stress calculations

$Fc' = Fc^*(C_D C_M C_t C_F C_i C_P)$

C_D = 1.60 For floor live load
C_M = 1.00 for MC < 19%
C_t = 1.00 No prolonged exposure to +100 degrees F
C_F = 1.15 Size increase
C_i = 1.00 No incising
C_T = 1.00

	X-Axis	Y-Axis	
$Fc^* = Fc^*(C_D C_M C_t C_F C_i)$ =	2484	2484	psi
Emin =	580000	580000	psi
$E' = Emin^*(C_M C_t C_i C_T)$ =	580000	580000	psi
k =	1.00	1.00	
L =	8.00	1.00	ft
le = kL =	8	1	ft
le/d =	27.43	4.00	≤ 50, OK
$Fc_{E1}, Fc_{E2} = 0.822^*E'min/(le/d)^2$ =	634	29798	psi
c =	0.8	0.8	
C_P =	0.240	0.982	
Kf =	1.00	0.60	
$Fc' = Fc^*(C_D C_M C_t C_F C_i C_P)$ =	**596**	1464	psi

Flexure - governed by D

Allowable bending stress calculations

$Fb' = Fb^*(C_D C_M C_t C_L C_F C_{fu} C_i C_r C_f)$

C_M = 1.00 For MC < 19%
C_t = 1.00 No prolonged exposure to +100 degrees F
C_F = 1.50 No size increase
C_{fu} = 1.00 No flat side loading
C_i = 1.00 No incising
C_r = 1.00 Not a repetitive member
C_f = 1.00 Not circular or diamond shaped
C_T = 1.00

	X-Axis	Y-Axis	
C_D =	0.90	0.90	
$Fb^* = Fb^*(C_D C_M C_t C_F C_{fu} C_i C_r C_f)$ =	1215	1215	psi
Emin =	580000	580000	psi
$E' = E^*(C_M C_t C_i C_T)$ =	580000	580000	psi
unbraced length, lu =	1.00	8.00	ft
lu/d =	3.43	32.00	
le =	24.72	166.98	in
$R_B = (le^*d/b^2)^{1/2}$ =	3.10	6.39	≤ 50, OK
$Fb_{E1}, Fb_{E2} = 1.20^*E'min/(R_B)^2$ =	72399	17020	psi
C_L =	0.999	1.000	
$Fb'_1, Fb'_2 = Fb^*(C_D C_M C_t C_L C_F C_{fu} C_i C_r C_f)$ =	**1214**	**1215**	psi

Actual compression stress calculations

P = 5179 lbs
A = b*d = 10.5 in^2
fc = P/A = 493 psi
OK

Flexural stress calculations

	X-Axis	Y-Axis	
Gravity Load Flexure = P*e =	0	0	lb-ft
Lateral Load Flexure = (0.7E or 0.6W)*ht =	0	0	lb-ft
Total design load =	0	0	lb-ft
$Sx = bd^2/6, Sy = db^2/6$ =	6.13	5.25	in^3
f_{b1}, f_{b2} =	0	0	psi
	OK	OK	

Flexure + Compression governed by D+(0.6W or 0.7E)

$(f_c/F'_c)^2+f_{b1}/(F_b'_1{}^*[1-f_c/F_{cE1}])+f_{b2}/(F'_{b2}{}^*[1-(f_c/F_{cE2})-(f_{b1}/F_{bE})^2])$ = N/A OK

F'_c = 596 psi
F_{cE1} = 634 psi (x-axis)
F_{cE2} = 29798 psi (y-axis)
F'_{b1} = 2158 psi (x-axis)
F'_{b2} = 2160 psi (y-axis)
F_{bE1} = 72399 psi (x-axis)
F_{bE2} = 17020 psi (y-axis)
fc = 493 psi
fb_1 = 0 psi (x-axis)
fb_2 = 0 psi (y-axis)

Shear governed by D

	X-Axis	Y-Axis	
C_D =	0.90	0.90	
$Fv' = Fv^*(C_D C_M C_t C_i)$ =	**162**	**162**	psi
Max V =	0	0	lbs
A = b*d =	10.50	10.50	in^2
fv = 1.5*V/A =	0	0	psi
	OK	OK	

Bearing

Fc⊥' = **625** psi
Fc⊥ = 493 psi OK

Deflection from Lateral loads(Unfactored loads)

X-Axis			Y-Axis		
$I = bd^3/12$ =	11	in^4	$I = db^3/12$ =	8	in^4
E' =	580000	psi	E' =	580000	psi
Δallow = L/240 =	0.40	in	Δallow = L/240 =	0.05	in
Max service load =	0	plf	Max service load =	0	plf
Δ = 5w*h^4/384EI =	0.00	in OK	Δ = 5w*h^4/384EI =	0.00	in OK

Column calculations summary

Load combination	C_D	fc	Fc'	fc/Fc'	fb_1	Fb_1'	fb_1/Fb_1'	fb_2	Fb_2'	fb_2/Fb_2'	$fc+fb_1+fb_2$
D	0.90	128	335	0.38	0	1214	N/A	0	1215	N/A	0.14
D+L	0.90	128	335	0.38	0	1214	N/A	0	1215	N/A	0.14
D+(Lr or S or R)	1.25	176	466	0.38	0	1686	N/A	0	1688	N/A	0.14
D+0.75L+0.75(Lr or S or R)	1.25	164	466	0.35	0	1686	N/A	0	1688	N/A	0.12
D+(0.6W or 0.7E)	1.60	493	596	**0.83**	0	2158	N/A	0	2160	N/A	**0.68**
D+0.75L+0.75(Lr or S or R)+0.75(0.6W)	1.60	275	596	0.46	0	2158	N/A	0	2160	N/A	0.21
D+0.75L+0.75(S)+0.75(0.7E)	1.60	402	596	0.67	0	2158	N/A	0	2160	N/A	0.45
0.6D+0.6W	1.60	225	596	0.38	0	2158	N/A	0	2160	N/A	0.14
0.6D+0.7E	1.60	442	596	0.74	0	2158	N/A	0	2160	N/A	0.55

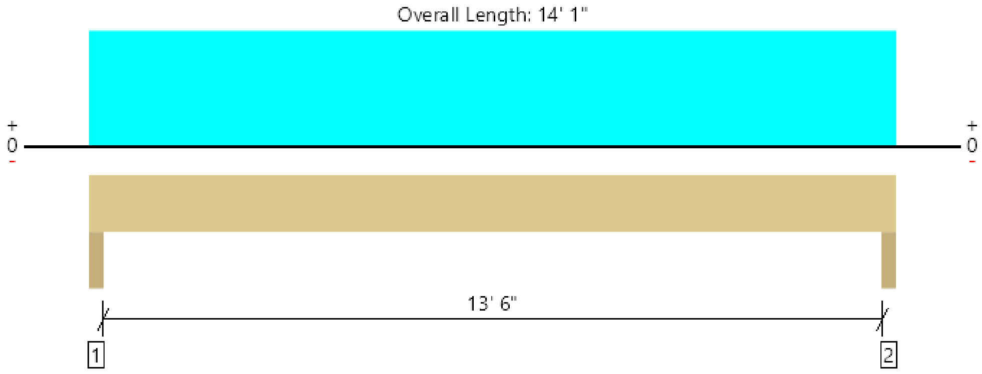
Load combination	C_D	fv_1	Fv_1'	fv_1/Fv_1'	fv_2	Fv_2'	fv_2/Fv_2'
D	0.90	0	162	N/A	0	162	N/A
D+L	0.90	0	162	N/A	0	162	N/A
D+(Lr or S or R)	1.25	0	225	N/A	0	225	N/A
D+0.75L+0.75(Lr or S or R)	1.25	0	225	N/A	0	225	N/A
D+(0.6W or 0.7E)	1.60	0	288	N/A	0	288	N/A
D+0.75L+0.75(Lr or S or R)+0.75(0.6W)	1.60	0	288	N/A	0	288	N/A
D+0.75L+0.75(S)+0.75(0.7E)	1.60	0	288	N/A	0	288	N/A

MEMBER REPORT

PASSED

Level, FB-3

2 piece(s) 1 3/4" x 14" 2.0E Microllam® LVL



All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal.

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	7125 @ 2"	9188 (3.50")	Passed (78%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	5649 @ 1' 5 1/2"	9310	Passed (61%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	23911 @ 7' 1/2"	24258	Passed (99%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.419 @ 7' 1/2"	0.458	Passed (L/394)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.565 @ 7' 1/2"	0.688	Passed (L/292)	--	1.0 D + 1.0 L (All Spans)

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.

System : Floor
Member Type : Drop Beam
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Total	
1 - Trimmer - DF	3.50"	3.50"	2.71"	1843	5281	7124	None
2 - Trimmer - DF	3.50"	3.50"	2.71"	1843	5281	7124	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	1' 6" o/c	
Bottom Edge (Lu)	14' 1" o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Comments
0 - Self Weight (PLF)	0 to 14' 1"	N/A	14.3	--	
1 - Uniform (PSF)	0 to 14' 1" (Front)	12' 6"	15.0	60.0	Default Load
2 - Uniform (PLF)	0 to 14' 1" (Front)	N/A	60.0	-	WALL

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The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

ForteWEB Software Operator	Job Notes
L RIED POPE PE, PLS LR POPE ENGINEERING INC (435) 628-1676 lrpope@lrpope.com	

3/29/2021 5:27:54 AM UTC

ForteWEB v3.1, Engine: V8.1.6.2, Data: V8.0.1.0

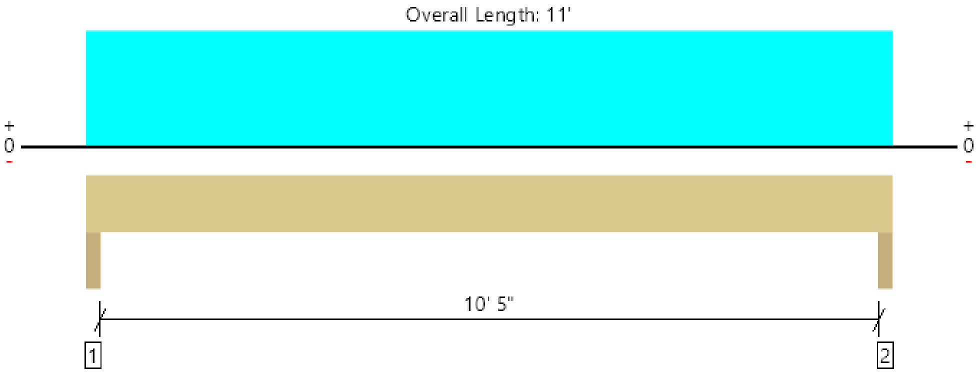
File Name: MILAN LOT 2

MEMBER REPORT

PASSED

Level, FB-4

2 piece(s) 1 3/4" x 11 7/8" 2.0E Microllam® LVL



All locations are measured from the outside face of left support (or left cantilever end). All dimensions are horizontal.

Design Results	Actual @ Location	Allowed	Result	LDF	Load: Combination (Pattern)
Member Reaction (lbs)	5553 @ 2"	9188 (3.50")	Passed (60%)	--	1.0 D + 1.0 L (All Spans)
Shear (lbs)	4259 @ 1' 3 3/8"	7897	Passed (54%)	1.00	1.0 D + 1.0 L (All Spans)
Moment (Ft-lbs)	14359 @ 5' 6"	17848	Passed (80%)	1.00	1.0 D + 1.0 L (All Spans)
Live Load Defl. (in)	0.253 @ 5' 6"	0.356	Passed (L/506)	--	1.0 D + 1.0 L (All Spans)
Total Load Defl. (in)	0.341 @ 5' 6"	0.533	Passed (L/376)	--	1.0 D + 1.0 L (All Spans)

- Deflection criteria: LL (L/360) and TL (L/240).
- Allowed moment does not reflect the adjustment for the beam stability factor.

System : Floor
Member Type : Drop Beam
Building Use : Residential
Building Code : IBC 2018
Design Methodology : ASD

Supports	Bearing Length			Loads to Supports (lbs)			Accessories
	Total	Available	Required	Dead	Floor Live	Total	
1 - Trimmer - DF	3.50"	3.50"	2.12"	1428	4125	5553	None
2 - Trimmer - DF	3.50"	3.50"	2.12"	1428	4125	5553	None

Lateral Bracing	Bracing Intervals	Comments
Top Edge (Lu)	8' 3" o/c	
Bottom Edge (Lu)	11' o/c	

•Maximum allowable bracing intervals based on applied load.

Vertical Loads	Location (Side)	Tributary Width	Dead (0.90)	Floor Live (1.00)	Comments
0 - Self Weight (PLF)	0 to 11'	N/A	12.1	--	
1 - Uniform (PSF)	0 to 11' (Front)	12' 6"	15.0	60.0	Default Load
2 - Uniform (PLF)	0 to 11' (Front)	N/A	60.0	-	WALL

Weyerhaeuser Notes

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The product application, input design loads, dimensions and support information have been provided by ForteWEB Software Operator

ForteWEB Software Operator	Job Notes
L RIED POPE PE, PLS LR POPE ENGINEERING INC (435) 628-1676 lrpope@lrpope.com	

Weyerhaeuser

3/29/2021 5:29:01 AM UTC
ForteWEB v3.1, Engine: V8.1.6.2, Data: V8.0.1.0
File Name: MILAN LOT 2

ASD Wood Member Design v7.3.1 (2-17-17)

PROJECT MILAN LOT 2 (STAIR STRINGER)

Member Dimensions Beam ● Joist ○

	Cantilever	Span	Cantilever	Total Length
Span =		14.50		14.50

Unbraced length = 1.00 ft
Number of plys = 1
Member width, b = 1.75 in ☐ Custom width
Member depth, d = 9.5 in ☐ Custom depth
Orientation = Strong

Member Material Properties

Lumber type = Engineered
Type = LVL
Grade = 1.9E
Member unit weight = 42 pcf
Bearing length @ support A = 3.00 in ≥ 1.5 in
Bearing length @ support B = 3.00 in ≥ 1.5 in

Loads

Roof DL = 25 psf
Roof Lr = 20 psf
Snow, S = 15 psf
Rain, R = 5 psf
Floor DL = 15 psf
Floor LL = 40 psf

LL deflection controls member design (69%)

Unfactored Load Reactions

Load type	R_A	R_B	
D =	253	253	*lbs*
L =	580	580	*lbs*
Lr =	0	0	*lbs*
S =	0	0	*lbs*
R =	0	0	*lbs*
W =	0	0	*lbs*
E =	0	0	*lbs*

Point Loads

	P_L	P_D	P_T	a	LL Type	Load Description
P_1 =			0			
P_2 =			0			
P_3 =			0			
P_4 =			0			
P_5 =			0			
P_6 =			0			

Uniform Loads **Roof slope =** 4.00 :12 **Member slope =** :12

Live, psf	Dead, psf	Trib. Width		W_L	W_D	W_T	Start @	End @	LL Type	Load Description
40	15	2.00	W_1 =	80	30	110	0.00	14.50	Floor	
			W_2 =			0	0.00			
			W_3 =			0	0.00			
			W_4 =			0				
			W_5 =			0				
			W_6 =			0				

Triangular Loads (Starting or ending load must be 0)

	Start W_L	Start W_D	End W_L	End W_D	Start W_T	End W_T	Start @	End @	LL Type	Load Description
T_1 =					0	0				
T_2 =					0	0				
T_3 =					0	0				
T_4 =					0	0				

Member Shear Design ***Member design controlled by D+L***

Fv = 285 psi
$Fv' = Fv^*(C_DC_MC_tC_i)$ = 285 psi
Max V = 833 lbs
Design V = 727 lbs
A = b*d = 16.63 in^2
fv = 1.5*V/A = 75 psi
OK

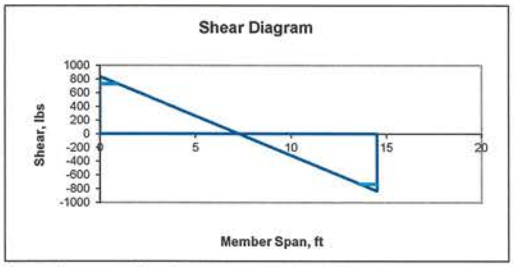
Adjustment Factors

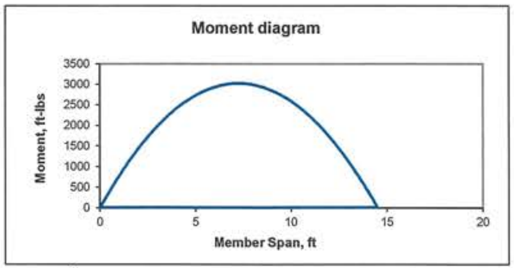
C_D =	1.00	For floor live load
C_M =	1.00	For MC < 19%
C_t =	1.00	Insulated against 100+ F
C_F =	▨	No size increase
C_V =	1.03	Volume Factor
C_{fu} =	▨	Narrow face loaded
C_i =	▨	No incising
C_r =	1.00	Not a repetitive member
C_f =	▨	Rectangular shaped
C_T =	▨	Buckling stiffness factor
C_b =	1.00	Bearing area factor

Member Bending Design ***Member design controlled by D+L***

	Fb^+	Fb^-	
Fb =	2600	2600	psi
$Fb^* = Fb^*(C_DC_MC_tC_FC_iC_rC_f)$ =	2600	2600	psi
Emin =	965710	965710	psi
Emin' = Emin*(CMCtCiCT) =	965710	965710	psi
unbraced length, lu =	1.00	1.00	ft
lu/d =	1.26	1.26	
le =	25	25	in
$R_B = (le^*d/b^2)^{1/2}$ =	8.76	8.76	*≤ 50, OK*
$F_{bE} = 1.20^*E'min/(R_B)^2$ =	15112	15112	psi
C_L =	0.990	0.990	
$Fb' = Fb^*(C_DC_MC_tC_LC_FC_VC_{fu}C_iC_rC_f)$ =	2657	2657	psi

	+ Moment	- Moment	
Max moment, M =	3018		lb-ft
$S = bd^2/6$ =	26.32	26.32	in^3
fb = M/S =	1376	0	psi
	OK	*OK*	





Member Bearing **Member design controlled by D+L**

$Fc\perp$ = 625 psi
$F'c\perp = Fc\perp^*(C_MC_tC_iC_b)$ = 625 psi

	C_b	P, lbs	A, in^2	$fc\perp = P/A$		
Support @ A =	1.00	833	5.25	159	psi	*OK*
Support @ B =	1.00	833	5.25	159	psi	*OK*

Member Deflection

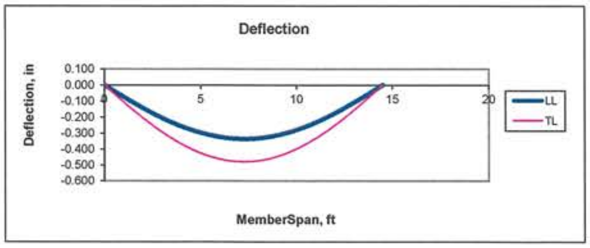
Moment of Inertia, $I = bd^3/12$ = 125.034 in^4
E = 1900000 psi
$E' = E^*(C_MC_tC_i)$ = 1900000 psi

Mid Span Deflection

Loading	$Ratio_{allow}$	$\Delta_{allowed}$	Δ_{actual}	$Ratio_{actual}$	Check
Δ_{LL}	360	0.483	0.335	L/519	*OK*
Δ_{TL}	240	0.725	0.481	L/361	*OK*

Cantilever Deflection

Loading	$Ratio_{allow}$	$\Delta_{allowed}$	Δ_{actual}	$Ratio_{actual}$	Check
Δ_{LL}	180	0.000	0.000	N/A	*OK*
Δ_{TL}	120	0.000	0.000	N/A	*OK*



(1) 1.75" x 9.5" LVL 1.9E

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PROJECT MILAN LOT 2
Design calculations for 2X6 15.5 foot tall DF #2 wood stud wall

Gravity loads (Compression only) D+(Lr or S or R)

Compression allowable stress calculations

Stud grade = DF #2
Fc = 1350 psi NDS 2012 Table 4A
E = 1600000 psi NDS 2012 Table 4A
Actual stud width = 1.50 in
Actual stud depth = 5.50 in
$Fc' = Fc^*(C_DC_MC_tC_FC_iC_P)$
C_D = 1.25 For roof live load
C_M = 1.00 For MC < 19%
C_t = 1.00 Insulated against 100+ F
C_F = 1.10 Size increase
C_i = 1.00 No incising
C_T = 1.00

$Fc^* = Fc^*(C_DC_MC_tC_FC_i)$ = 1856 psi
Emin = 580000 psi
$E' = E^*(C_MC_tC_iC_T)$ = 580000 psi
k = 1.0
L = 15.50 ft
le = kL = 15.50 ft
le/d = 33.82 *≤ 50, OK*
$F_{cE} = 0.822^*E'min/(le/d)^2$ = 417 psi
c = 0.8 For solid sawn lumber
C_P = 0.213
$Fc' = Fc^*(C_DC_MC_tC_FC_iC_P)$ = 395 psi

Compression actual stress calculations

Gravity unit loads
Roof DL = 25 psf
Lr = 20 psf
S = 15 psf
R = 5 psf
Floor DL = 15 psf
LL = 40 psf
Wall weight = 15 psf

Roof slope = 4 :12
Roof Tributary width = 12.50 ft
Floor Tributary width = 0.00 ft
Wall height on top of wall = 0.00 ft
Stud spacing = 12 in o.c.
Roof Tributary Area = 12.50 ft^2
Floor Tributary Area = 0.00 ft^2
width, b = 1.50
depth, d = 5.50
P = 812 lbs
A = b*d = 8.25 in^2
fc = P/A = 98 *psi OK*

Lateral Loads (Bending only) 0.6D+0.6W

Bending allowable stress calculations

Fb = 900 psi NDS 2012 Table 4A
E = 1600000 psi NDS 2012 Table 4A
Actual stud width = 1.5 in
Actual stud depth = 5.5 in

$Fb' = Fb^*(C_DC_MC_tC_LC_FC_{fu}C_iC_rC_f)$
C_D = 1.60 For wind or seismic loads
C_M = 1.00 For MC < 19%
C_t = 1.00 Insulated against 100+ F
C_F = 1.30 Size increase
C_{fu} = 1.00 No flat side
C_i = 1.00 No incising
C_r = 1.15 Repetitive member
C_f = 1.00 Rectangular shaped
C_T = 1.00

$Fb^* = Fb^*(C_DC_MC_tC_FC_{fu}C_iC_rC_f)$ = 2153 psi
Emin = 580000 psi
$E' = E^*(C_MC_tC_iC_T)$ = 580000 psi
unbraced length, lu = 1.00 ft
lu/d = 2.18
le = 24.72 in
$R_B = (le^*d/b^2)^{1/2}$ = 7.77 *≤ 50, OK*
$F_{bE} = 1.20^*E'min/(R_B)^2$ = 11518 psi
C_L = 0.989
$Fb' = Fb^*(C_DC_MC_tC_LC_FC_{fu}C_iC_rC_f)$ = 2129 psi

Bending actual stress calculations

Stud height = 15.5 ft
Stud spacing = 12 in o.c.
p_{net30} = 23.20 psf
λ = 1.32
Wind load, W = 30.62 plf

I_E = 1.00
S_{DS} = 0.464
W_W = 15 psf
Stud spacing = 12 in o.c.
Seismic load, E = 7.14 plf
Design Load 0.6W = 18.37 plf
Design Moment = $wl^2/8$ = 552 ft-lb
$S = bd^2/6 = b^*d^2/6$ = 7.56 in^3
fb = M/S = 875 *psi OK*

Deflection

Allowable deflection, L/360 0.52 in
Actual deflection = $5wL^4/384EI$ = 0.50 *in OK*
L/371

Shear

Fv = 180 psi NDS 2012 Table
$Fv' = Fv^*(C_DC_MC_tC_i)$ = 288 psi
Max V = W*L/2 = 142 psi
A = b*d = 8.25 in^2
fv = 1.5*V/A = 26 *psi OK*

Bending + Compression 0.6D+0.6W

fc = P/A = 41 psi
F'_c = 401 psi
F_{cE} = 417 psi *> fc, OK*
fb = 1459 psi
F'_b = 2129 psi

$(f_c/F'_c)^2+f_b/(F_b'^*[1-f_c/F_{cE}])$ = 0.770 *OK*

Plate Bearing D+(Lr or S or R)
Perpendicular to grain (DF #2 top and bottom plates)

$Fc_\perp' = Fc_\perp^*(C_MC_tC_iC_b)$
Top and bottom plate grade = DF #2
C_M = 1.00 for MC < 19%
C_t = 1.00 Insulated against 100+
C_i = 1.00 No incising
C_b = 1.00 No increase for bearin
$Fc_\perp$ = 625 psi
$Fc_\perp' = Fc_\perp^*(C_MC_tC_iC_b)$ = 625 psi
$fc_\perp$ = 98 lbs

Calculation summary								
	Allowable and actual stresses							
Load combination	C_D	fc	Fc'	fc/Fc'	fb	Fb'	fb/Fb'	fb+fc
D	0.90	68	386	0.177	0	1204	0.000	0.03
D+L	1.00	68	389	0.175	0	1337	0.000	0.03
D+(Lr or S or R)	1.25	98	395	*0.249*	0	1668	0.000	0.06
D+0.75*LL+0.75*(Lr or S or R)	1.25	91	395	0.230	0	1668	0.000	0.05
D+(0.6W or 0.7E)	1.60	68	401	0.170	875	2129	0.411	0.52
D+0.75*(0.6W)+0.75*L+0.75*(Lr or S or R)	1.60	91	401	0.227	657	2129	0.308	0.45
D+0.75*(0.7E)+0.75*L+0.75*S	1.60	85	401	0.213	179	2129	0.084	0.15
0.6D+0.6W	1.60	41	401	0.102	1459	2129	*0.685*	*0.77*
0.6D+0.7E	1.60	41	401	0.102	238	2129	0.112	0.13

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PROJECT MILAN LOT 2

Design calculations for 2X6 9.08 foot tall DF #2 wood stud wall

Gravity loads (Compression only) D+(Lr or S or R)

Compression allowable stress calculations

Stud grade = DF #2
Fc = 1350 psi NDS 2012 Table 4A
E = 1600000 psi NDS 2012 Table 4A
Actual stud width = 1.50 in
Actual stud depth = 5.50 in
Fc' = Fc*($C_DC_MC_tC_FC_iC_P$)
C_D = 1.25 For roof live load
C_M = 1.00 For MC < 19%
C_t = 1.00 Insulated against 100+ F
C_F = 1.10 Size increase
C_i = 1.00 No incising
C_T = 1.00

Fc* = Fc*($C_DC_MC_tC_FC_i$) = 1856 psi
Emin = 580000 psi
E' = E*($C_MC_tC_iC_T$) = 580000 psi
k = 1.0
L = 9.08 ft
le = kL = 9.08 ft
le/d = 19.81 *≤ 50, OK*
$F_{cE} = 0.822$*E'min/$(le/d)^2$ = 1215 psi
c = 0.8 For solid sawn lumber
C_P = 0.533
Fc' = Fc*($C_DC_MC_tC_FC_iC_P$) = 989 psi

Compression actual stress calculations

Gravity unit loads

Roof DL = 25 psf
Lr = 20 psf
S = 15 psf
R = 5 psf
Floor DL = 15 psf
LL = 40 psf
Wall weight = 15 psf

Roof slope = 4 :12
Roof Tributary width = 25.00 ft
Floor Tributary width = 0.00 ft
Wall height on top of wall = 0.00 ft
Stud spacing = 16 in o.c.
Roof Tributary Area = 33.33 ft^2
Floor Tributary Area = 0.00 ft^2
width, b = 1.50
depth, d = 5.50
P = 1727 lbs
A = b*d = 8.25 in^2
fc = P/A = 209 *psi OK*

Lateral Loads (Bending only) 0.6D+0.6W

Bending allowable stress calculations

Fb = 900 psi NDS 2012 Table 4A
E = 1600000 psi NDS 2012 Table 4A
Actual stud width = 1.5 in
Actual stud depth = 5.5 in

Fb' = Fb*($C_DC_MC_tC_LC_FC_{fu}C_iC_rC_f$)
C_D = 1.60 For wind or seismic loads
C_M = 1.00 For MC < 19%
C_t = 1.00 Insulated against 100+ F
C_F = 1.30 Size increase
C_{fu} = 1.00 No flat side
C_i = 1.00 No incising
C_r = 1.15 Repetitive member
C_f = 1.00 Rectangular shaped
C_T = 1.00

Fb* = Fb*($C_DC_MC_tC_FC_{fu}C_iC_rC_f$) = 2153 psi
Emin = 580000 psi
E' = E*($C_MC_tC_iC_T$) = 580000 psi
unbraced length, lu = 1.00 ft
lu/d = 2.18
le = 24.72 in
$R_B = (le*d/b^2)^{1/2}$ = 7.77 *≤ 50, OK*
$F_{bE} = 1.20$*E'min/$(R_B)^2$ = 11518 psi
C_L = 0.989
Fb' = Fb*($C_DC_MC_tC_LC_FC_{fu}C_iC_rC_f$) = 2129 psi

Bending actual stress calculations

Stud height = 9.1 ft
Stud spacing = 16 in o.c.
p_{net30} = 23.57 psf
λ = 1.32
Wind load, W = 41.48 plf

I_E = 1.00
S_{DS} = 0.464
W_W = 15 psf
Stud spacing = 16 in o.c.
Seismic load, E = 9.52 plf
Design Load 0.6W = 24.89 plf
Design Moment = $wl^2/8$ = 256 ft-lb
S = $bd^2/6 = b*d^2/6$ = 7.56 in^3
fb = M/S = 407 *psi OK*

Deflection

Allowable deflection, L/360 0.30 in
Actual deflection = $5wL^4/384EI$ = 0.08 *in OK*
L/1361

Shear

Fv = 180 psi NDS 2012 Table
Fv' = Fv*($C_DC_MC_tC_i$) = 288 psi
Max V = W*L/2 = 113 psi
A = b*d = 8.25 in^2
fv = 1.5*V/A = 21 *psi OK*

Bending + Compression 0.6D+0.6W

fc = P/A = 77 psi
F'_c = 1049 psi
F_{cE} = 1215 psi *> fc, OK*
fb = 678 psi
F'_b = 2129 psi

$(f_c/F'_c)^2+f_b/(F_b'*[1-f_c/F_{cE}])$ = 0.346 *OK*

Plate Bearing D+(Lr or S or R)

Perpendicular to grain (DF #2 top and bottom plates)

$Fc_{\perp}' = Fc_{\perp}*(C_MC_tC_iC_b)$

Top and bottom plate grade = DF #2
C_M = 1.00 for MC < 19%
C_t = 1.00 Insulated against 100+
C_i = 1.00 No incising
C_b = 1.00 No increase for bearin
$Fc_{\perp}$ = 625 psi
$Fc_{\perp}' = Fc_{\perp}*(C_MC_tC_iC_b)$ = 625 psi
$fc_{\perp}$ = 209 lbs

Calculation summary

Load combination	Allowable and actual stresses							
	C_D	fc	Fc'	fc/Fc'	fb	Fb'	fb/Fb'	fb+fc
D	0.90	128	878	0.146	0	1204	0.000	0.02
D+L	1.00	128	918	0.140	0	1337	0.000	0.02
D+(Lr or S or R)	1.25	209	989	***0.212***	0	1668	0.000	0.04
D+0.75*LL+0.75*(Lr or S or R)	1.25	189	989	0.191	0	1668	0.000	0.04
D+(0.6W or 0.7E)	1.60	128	1049	0.122	407	2129	0.191	0.23
D+0.75*(0.6W)+0.75*L+0.75*(Lr or S or R)	1.60	189	1049	0.180	305	2129	0.143	0.20
D+0.75*(0.7E)+0.75*L+0.75*S	1.60	174	1049	0.166	82	2129	0.038	0.07
0.6D+0.6W	1.60	77	1049	0.073	678	2129	***0.319***	***0.35***
0.6D+0.7E	1.60	77	1049	0.073	109	2129	0.051	0.06

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PROJECT MILAN LOT 2 (WORST CASE TWO STORY)

Design calculations for 2X6 9.08 foot tall DF #2 wood stud wall

Gravity loads (Compression only) D+0.75*LL+0.75*(Lr or S or R)

Compression allowable stress calculations

Stud grade = DF #2
Fc = 1350 psi NDS 2012 Table 4A
E = 1600000 psi NDS 2012 Table 4A
Actual stud width = 1.50 in
Actual stud depth = 5.50 in
$Fc' = Fc*(C_DC_MC_tC_FC_iC_P)$
C_D = 1.25 For roof live load
C_M = 1.00 For MC < 19%
C_t = 1.00 Insulated against 100+ F
C_F = 1.10 Size increase
C_i = 1.00 No incising
C_T = 1.00

$Fc^* = Fc*(C_DC_MC_tC_FC_i)$ = 1856 psi
Emin = 580000 psi
$E' = E*(C_MC_tC_iC_T)$ = 580000 psi
k = 1.0
L = 9.08 ft
le = kL = 9.08 ft
le/d = 19.81 *≤ 50, OK*
$F_{cE} = 0.822*E'min/(le/d)^2$ = 1215 psi
c = 0.8 For solid sawn lumber
C_P = 0.533
$Fc' = Fc*(C_DC_MC_tC_FC_iC_P)$ = 989 psi

Compression actual stress calculations

Gravity unit loads

Roof DL = 25 psf
Lr = 20 psf
S = 15 psf
R = 5 psf
Floor DL = 15 psf
LL = 40 psf
Wall weight = 15 psf

Roof slope = 4 :12
Roof Tributary width = 17.00 ft
Floor Tributary width = 12.00 ft
Wall height on top of wall = 9.00 ft
Stud spacing = 16 in o.c.
Roof Tributary Area = 22.67 ft^2
Floor Tributary Area = 16.00 ft^2
width, b = 1.50
depth, d = 5.50
P = 2019 lbs
A = b*d = 8.25 in^2
fc = P/A = 245 *psi OK*

Lateral Loads (Bending only) 0.6D+0.6W

Bending allowable stress calculations

Fb = 900 psi NDS 2012 Table 4A
E = 1600000 psi NDS 2012 Table 4A
Actual stud width = 1.5 in
Actual stud depth = 5.5 in

$Fb' = Fb*(C_DC_MC_tC_LC_FC_{fu}C_iC_rC_f)$
C_D = 1.60 For wind or seismic loads
C_M = 1.00 For MC < 19%
C_t = 1.00 Insulated against 100+ F
C_F = 1.30 Size increase
C_{fu} = 1.00 No flat side
C_i = 1.00 No incising
C_r = 1.15 Repetitive member
C_f = 1.00 Rectangular shaped
C_T = 1.00

$Fb^* = Fb*(C_DC_MC_tC_FC_{fu}C_iC_rC_f)$ = 2153 psi
Emin = 580000 psi
$E' = E*(C_MC_tC_iC_T)$ = 580000 psi
unbraced length, lu = 1.00 ft
lu/d = 2.18
le = 24.72 in
$R_B = (le*d/b^2)^{1/2}$ = 7.77 *≤ 50, OK*
$F_{bE} = 1.20*E'min/(R_B)^2$ = 11518 psi
C_L = 0.989
$Fb' = Fb*(C_DC_MC_tC_LC_FC_{fu}C_iC_rC_f)$ = 2129 psi

Bending actual stress calculations

Stud height = 9.1 ft
Stud spacing = 16 in o.c.
p_{net30} = 23.57 psf
λ = 1.32
Wind load, W = 41.48 plf

I_E = 1.00
S_{DS} = 0.464
W_W = 15 psf
Stud spacing = 16 in o.c.
Seismic load, E = 9.52 plf
Design Load 0.6W = 24.89 plf
Design Moment = $wl^2/8$ = 256 ft-lb
$S = bd^2/6 = b*d^2/6$ = 7.56 in^3
fb = M/S = 407 *psi OK*

Deflection

Allowable deflection, L/360 0.30 in
Actual deflection = $5wL^4/384EI$ = 0.08 *in OK*
L/1361

Shear

Fv = 180 psi NDS 2012 Table
$Fv' = Fv*(C_DC_MC_tC_i)$ = 288 psi
Max V = W*L/2 = 113 psi
A = b*d = 8.25 in^2
fv = 1.5*V/A = 21 *psi OK*

Bending + Compression 0.6D+0.6W

fc = P/A = 87 psi
F'_c = 1049 psi
F_{cE} = 1215 psi *> fc, OK*
fb = 678 psi
F'_b = 2129 psi

$(f_c/F'_c)^2+f_b/(F_b'*[1-f_c/F_{cE}])$ = 0.350 *OK*

Plate Bearing D+0.75*LL+0.75*(Lr or S or R)

Perpendicular to grain (DF #2 top and bottom plates)

$Fc_{\perp}' = Fc_{\perp}*(C_MC_tC_iC_b)$
Top and bottom plate grade = DF #2
C_M = 1.00 for MC < 19%
C_t = 1.00 Insulated against 100+
C_i = 1.00 No incising
C_b = 1.00 No increase for bearin
$Fc_{\perp}$ = 625 psi
$Fc_{\perp}' = Fc_{\perp}*(C_MC_tC_iC_b)$ = 625 psi
$fc_{\perp}$ = 245 lbs

Calculation summary								
	Allowable and actual stresses							
Load combination	C_D	fc	Fc'	fc/Fc'	fb	Fb'	fb/Fb'	fb+fc
D	0.90	145	878	0.165	0	1204	0.000	0.03
D+L	1.00	223	918	0.243	0	1337	0.000	0.06
D+(Lr or S or R)	1.25	200	989	0.202	0	1668	0.000	0.04
D+0.75*LL+0.75*(Lr or S or R)	1.25	245	989	***0.247***	0	1668	0.000	0.06
D+(0.6W or 0.7E)	1.60	145	1049	0.139	407	2129	0.191	0.24
D+0.75*(0.6W)+0.75*L+0.75*(Lr or S or R)	1.60	245	1049	0.233	305	2129	0.143	0.23
D+0.75*(0.7E)+0.75*L+0.75*S	1.60	234	1049	0.223	82	2129	0.038	0.10
0.6D+0.6W	1.60	87	1049	0.083	678	2129	***0.319***	***0.35***
0.6D+0.7E	1.60	87	1049	0.083	109	2129	0.051	0.06

6/1/2020 17:34

Wood Connection - Bolted V6.2.0 (7/19/16)

Bolted connection calculations for: SILL PLATE ANCHORAGE FOR WORST CASE OUT OF PLANE LOADS-48" O.C.

Project: MILAN LOT 2 **Date:** 6/1/2020 17:37

CONNECTION DETAILS

Single or double shear connection = SINGLE
Bolt Diameter, D = 0.5 in
Dr = 0.5 in
Bending yield strength, F_{yb} = 45000 psi
Side member dowel bearing strength, $F_{em||}$ = 6000 psi
Side member dowel bearing strength, $F_{em\perp}$ = 6000 psi
Steel Fy = 33000 psi
Steel Fu = 41233 psi
G_m = 0.50
Side member dowel bearing strength, $F_{es||}$ = 5600 psi
Side member dowel bearing strength, $F_{es\perp}$ = 3150 psi
$R_e = F_{em}/F_{es}$ = 1.905
Main member bearing length, ℓ_m = 12.00 in
Side member bearing length, ℓ_s = 1.50 in
$R_t = \ell_m/\ell_s$ = 8.00
θ_m = 0 DEGREES
θ_s = 90 DEGREES
K_θ = 1.25
k_1 = 4.842
k_2 = 1.419
k_3 = 1.285
$F_{em\theta}$ = 6000 psi
$F_{es\theta}$ = 3150 psi

Main member
Width = 12 in
Depth = 12 in
Material = Concrete
G_m =

Side member
Width = 1.5 in
Depth = 5.5 in
Material = Wood
G_s = 0.50

Coefficients for k values
1+Re = 2.90
2+Re = 3.90
1+2Re = 4.81
1+Rt = 9.00
$1+Rt+Rt^2$ = 73.00

YIELD LIMIT EQUATIONS

Single shear

$$I_m = Z = \frac{D\ell_m F_{em}}{4K\theta}$$ 7200

$$I_s = Z = \frac{D\ell_s F_{es}}{4K\theta}$$ 473

$$II = Z = \frac{k_1 D\ell_s F_{es}}{3.6K\theta}$$ 2542

$$III_m = Z = \frac{k_2 D\ell_m F_{em}}{3.2(1+2Re)K\theta}$$ 2655

$$III_s = Z = \frac{k_3 D\ell_s F_{em}}{3.2(2+Re)K\theta}$$ 370

$$IV = Z = \frac{D^2}{3.2K\theta}\sqrt{\frac{2F_{em}F_{yb}}{3(1+R_e)}}$$ 492

Z = 370 lbs/bolt

Double shear

$$I_m = Z = \frac{D\ell_m F_{em}}{4K\theta}$$ 7200

$$I_s = Z = \frac{2D\ell_s F_{es}}{4K\theta}$$ 945

$$III_s = Z = \frac{2k_3 D\ell_s F_{em}}{3.2(2+Re)K\theta}$$ 740

$$IV = Z = \frac{2D^2}{3.2K\theta}\sqrt{\frac{2F_{em}F_{yb}}{3(1+R_e)}}$$ 984

BOLT DESIGN CALCULATIONS

Design Force, P = 590 lbs
Bolt diameter, D = 0.500 in
Z from governing yield limit equations = 370 lbs
Main member width, ℓ_m = 12.00
Main member depth, d_m = 12.00
Side member width, ℓ_s = 1.5 in
Connected member depth, d_s = 5.50 in
n = number of lag bolts per row = 1
Number of rows = 1
N = total number of lag bolts = 1
Bolts or screws staggered, Y or N? = N

lag screw guess = 1
C_D = 1.60
C_M = 1.00
C_t = 1.00
C_{eg} = 1.00
C_{Em} = 1.00

C_{Em} is for allowable stress increase of 1.2 for E_m level forces (ASCE 7-10 12.4.3.3)

Geometry factor, C_Δ = 1.0 if:

End distance = 7D = 3.5 in
Edge distance = 0.75 in
Center to center spacing, s = 4D = 2.00 in
Row spacing = 1.5D = 0.75 in

Group action factor, C_g :

Load slip modulus, $\gamma = 180000(D^{1.5})$ = 63640 lb/in

Member stiffness

E_m from NDS table 4A = 285000 psi
$A_m = d*w$ = 144.000 in^2
E_mA_m = 41040000 lb/in
E_s from NDS table 4A = 1600000 psi
A_s = 8.25 in
E_sA_s = 13200000 lb/in
$u = 1 + \gamma(s/2)[1/E_mA_m + 1/E_sA_s]$ = 1.006
$m = u - \mathrm{sqrt}(u^2-1)$ = 0.893

Coefficients for Cg

1+m = 1.8933
1-m = 0.1067
m^{2n} = 0.7980
$1+R_{EA}$ = 1.3216
$1+R_{EA}*m^n$ = 1.2873

Transformed section

E_mA_m/E_sA_s = 3.1091
E_sA_s/E_mA_m = 0.3216

R_{EA} = smaller of two ratios = 0.3216

$$C_g = \frac{m(1-m^{2n})}{n[(1 + R_{EA}m^n)*(1 + m) - 1 + m^{2n}]} \times \frac{1 + R_{EA}}{1 - m} = 1.0000$$

Connection Pallow = $Z' = N*ZC_DC_MC_tC_gC_\Delta C_{eg}C_{Em}$ = **592** lbs *OK*

CONTINUOUS FOOTING DESIGN V6.0.0 (7/1/16)

Project: MILAN LOT 2
Description: 0

Date: 6/1/2020 18:31
Design by: LRP

FOOTING LOADS AND BEARING CALCULATIONS

Footing Bearing Calculations (ASD)

Total vertical load, P = 1556 **plf**
Factored vertical load, Pu = 2063 **plf**
Allowable soil pressure, Qa = 1500 **psf**
footing length, l = 1 **ft**
Reqd footing width (Multiples of 4") = 12 **in.**
footing width, w = P/(Qa*l) = 14 **in.**
e = 0.00 **in.**
Use = 18 **in.**
Qmax = P/A+M/S = 1037 **psf**
Qmin = P/A-M/S = 1037 **psf**
OK

Uniform Loads

Framed wall = 15 psf * 15.5' ht = 233 plf
Concrete/CMU wall = 100 psf * 0' ht = 0 plf
Roof DL = 25 psf * 24.5' width : 646 plf
Roof LL = 20 psf * 24.5' width : 490 plf
Floor DL = 15 psf * 0' width = 0 plf
Floor LL = 40 psf * 0' width = 0 plf
Snow load = 20 psf * 24.5' width : 490 plf
Rain load = 5 psf * 24.5' width : 123 plf
Footing weight = 188 plf
Total service load, P = 1556 plf

FOOTING DESIGN CALCULATIONS

Footing Flexural Design (LRFD) - Plain Concrete

Governed by 1.2D+(f1*L or 0.5W)+1.6*(Lr or S or R)

f'c = 2500 **psi**
fy = 60000 **psi**
Factored load, Pu = 2063 **plf**
Qumax = 1375 **psf/ft of wall**
Factored moment, Mu = 251 **lb-ft/ft of wall**
h = 8 **in**
Sm = 12*h^2/6 = 128 **in^3**
ΦMn = 0.60*5*λ*f'c1/2*Sm = 1600 **lb-ft/ft of wall**
OK

Footing Longitudinal steel requirement

As(min) = 0.0018*b*d = 0.324 **in^2**
Number of rebar = 2
Size of rebar = 4
As actual = 0.4 **in^2**
OK

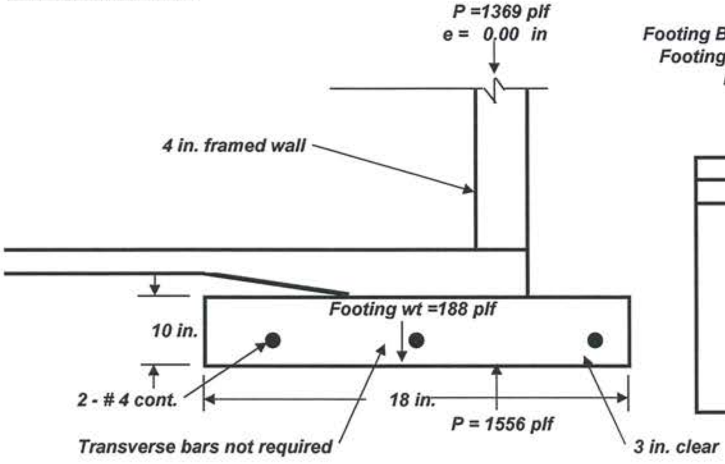
Footing Shear Design (LRFD) - Plain Concrete

Governed by 1.2D+(f1*L or 0.5W)+1.6*(Lr or S or R)

Vu = Qu*((w-wall thickness)/2-d) = 29 **plf**
ΦVc = 0.60*4/3*(f'c)^.5*b*h = 3840 **plf**
OK

FOOTING DESIGN AND LOAD COMBINATION SUMMARY AND SCHEMATIC FOOTING DETAIL

Footing design detail



	Actual	Allowable		
Footing Bearing pressure =	1037	1500	psf	OK
Footing One Way Shear =	29	3840	plf	OK
Footing Moment =	251	1600	lb-ft/ft	OK

Footing bearing calculation summary (ASD)

Load combination	Qmax	Qmin	e, ft
D	710	710	0.00
D+L	710	710	0.00
D+(Lr or S or R)	**1037**	**1037**	**0.00**
D+0.75L+0.75(Lr or S or R)	955	955	0.00
D+(0.6W or 0.7E)	710	710	0.00
D+0.75(0.6W)+0.75L+0.75(Lr or S or R)	955	955	0.00
D+0.75(0.7E)+0.75L+0.75(S)	955	955	0.00
0.6D+0.6W	426	426	0.00
0.6D+0.7E	426	426	0.00

SPOT FOOTING DESIGN V7.4.0 (4/21/15)
PROJECT: MILAN LOT 2

FOOTING LOADS AND BEARING CALCULATIONS

Footing Bearing Calculations (ASD)

Governed by D+(Lr or S or R)

Total vertical load, P =	8457	**lbs**
Max bearing capacity, Qa =	1500	**psf**
Footing w and l =	30	**in.**
Lateral Load =	0	**lbs**
Column Height =	9.00	**ft**
M =	0	**lb-ft**
e = M/P =	0.00	**ft**
w/6 =	0.417	**ft**
Qmax = P/A + Mx/Sx + My/Sy =	1354	**psf**
Qmin = P/A - Mx/Sx - My/Sy =	1354	**psf**
	OK	

Uniform Loads

Roof DL =	15 psf	**X 0 sq. ft. =**	0	lbs
Roof LL =	20 psf	**X 0 sq. ft. =**	0	lbs
Snow load =	20 psf	**X 0 sq. ft. =**	0	lbs
Rain load =	5 psf	**X 0 sq. ft. =**	0	lbs
Floor DL =	15 psf	**X sq. ft. =**	0	lbs
Floor LL =	70 psf	**X sq. ft. =**	0	lbs
Framed wall DL =	15 psf	**X sq. ft. =**	0	lbs
Concrete wall DL =	100 psf	**X sq. ft. =**	0	lbs

Lateral Loads

	X	**Y**	
Roof DL =	0	0	**lbs**
Lr =	0	0	**lbs**
S =	0	0	**lbs**
R =	0	0	**lbs**
FLOOR DL =	0	0	**lbs**
LL =	0	0	**lbs**
Wind =	0	0	**lbs**
Seismic =	0	0	**lbs**

Point Loads

Beam #	**LL**	**DL**	**LL Type**
RB-10	1902 lbs	2596 lbs	ROOF
RB-11	1360 lbs	1844 lbs	ROOF

Uplift =	0	lbs
Additional wt resisting uplift =	0	lbs
Footing wt =	755	lbs

FOOTING DESIGN CALCULATIONS - REINFORCED CONCRETE

Footing Flexural Design (LRFD)

Governed by 1.6*Q_{Max}

f'c =	2500	**psi**	
fy =	60000	**psi**	
Factored soil bearing pressure (1.6*Qa), Qu =	2166	**psf**	
Footing factored moment, Mu =	1083	**lb-ft/ft**	
Min. clear distance =	3.00	**in**	
Footing thickness =	10	**in**	
d =	6.75	**in**	
Starting guess for As = Mu/4d =	0.040	**in2**	
spacing =	12.00	**in o.c.**	
Rebar size =	4		
As =	0.200	**in^2**	
As(min) = 0.0018*b*d =	0.540	**in^2**	
As actual =	0.600	**in^2**	OK
a = (As*fy)/(0.85*f'c*b) =	0.471	**in**	
Footing factored moment, Mu =	1083	**lb-ft/ft**	
ΦMn = 0.9*As*fy*(d-a/2) =	5863	**lb-ft/ft**	OK
Conc. Ult. compressive strain, εcu =	0.003		
β1 =	0.85		
c =	0.554		
Strain in steel, εt = (εcu(d-a/β1))/(a/β1) =	0.034		
εt > 0.004 (ACI Requirement) =	OK		
εt > 0.005 (Tension controlled) =	OK		

Footing Two Way Shear Design (LRFD)

Governed by 1.6*Q_{Max}

Pu =	13540	**lbs**
Mu =	0	**lb-ft**
Column Embedment =	0.00	**in**
d =	6.75	**in**
Vu1 = Pu*(1-b1*b2/(B*L)) =	11094	**lbs**
γv = 1-1/(1+2/3*(b1/b2)^1/2) =	0.400	
Perimeter of shear failure = bo =	51	**in**
Vu = [Vu1/(d*bo)+γv*Mu*y/Jc]*(d*bo) =	11094	**lbs**
β = short side of column/long side of column =	1.00	
ΦVc = 0.75*(2+4/β)*(f'c)^0.5*bo*d =	77456	**lbs**
ΦVc = 0.75*(αsd/bo+2)*(f'c)^0.5*bo*d =	94163	**lbs**
ΦVc = 0.75*4*(f'c)^0.5*bo*d =	51638	**lbs**
ΦVc =	51638	**lbs**
	OK	

Footing One Way Shear Design (LRFD)

Governed by 1.2D+(f1*L or 0.5W)+1.6*(Lr or S or R)

Vu = Qu*((w-column thickness)/2-d) =	948	**plf**
ΦVc = 0.75*2*(f'c)^.5*b*d =	6075	**plf**
	OK	

Footing Uplift Design (ASD)

Governed by 0.6D+(0.6W or 0.7E)+0.6*footing wt+0.6*Bulk wt

Uplift force =	0	**lbs**
Concrete unit weight =	145	**pcf**
Total resisting dead load =	5195	**lbs**
0.6*D =	3117	**lbs**
	OK	

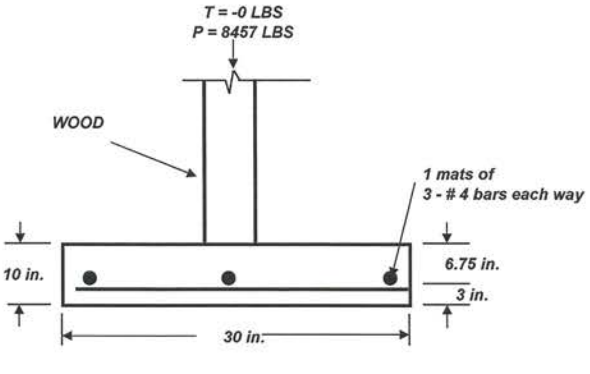
FOOTING DESIGN AND LOAD COMBINATION SUMMARY AND SCHEMATIC FOOTING DETAIL

	Actual	**Allowable**		
Footing Bearing pressure =	1354	1500	**psf**	OK
Footing One Way Shear =	948	6075	**lbs/ft**	OK
Footing Two Way Shear =	11094	51638	**lbs**	OK
Footing Moment =	1083	5863	**lb-ft/ft**	OK

Footing Bearing Pressure Summary by Load Combination

Load combination	Q_{max}	Q_{min}	Qu_{max}	Qu_{min}
D	832	832	1165	1165
D+L	832	832	1331	1331
D+(Lr or S or R)	**1354**	**1354**	**2166**	**2166**
D+0.75L+0.75(Lr or S or R)	1223	1223	1957	1957
D+(0.6W or 0.7E)	832	832	1331	1331
D+0.75+0.75*0.6W+0.75L+0.75(Lr or S or R)	1223	1223	1957	1957
D+0.75+0.75*0.7E+0.75L+0.75S	832	832	1331	1331
0.6D+0.6W	499	499	798	798
0.6D+0.7E	499	499	798	798

Design by: LRP   Date: 6/1/2020 18:35



SPOT FOOTING DESIGN V7.4.0 (4/21/15)
PROJECT: MILAN LOT 2

FOOTING LOADS AND BEARING CALCULATIONS

Footing Bearing Calculations (ASD)

Governed by D+(Lr or S or R)

Total vertical load, P = 10808 **lbs**
Max bearing capacity, Qa = 1500 **psf**
Footing w and l = 36 **in.**
Lateral Load = 0 **lbs**
Column Height = 9.00 **ft**
M = 0 **lb-ft**
e = M/P = 0.00 **ft**
w/6 = 0.500 **ft**
Qmax = P/A + Mx/Sx + My/Sy = 1201 **psf**
Qmin = P/A - Mx/Sx - My/Sy = 1201 **psf**
OK

Uniform Loads

Roof DL =	15 psf	**X 0 sq. ft. =**	0	lbs
Roof LL =	20 psf	**X 0 sq. ft. =**	0	lbs
Snow load =	20 psf	**X 0 sq. ft. =**	0	lbs
Rain load =	5 psf	**X 0 sq. ft. =**	0	lbs
Floor DL =	15 psf	**X sq. ft. =**	0	lbs
Floor LL =	70 psf	**X sq. ft. =**	0	lbs
Framed wall DL =	15 psf	**X sq. ft. =**	0	lbs
Concrete wall DL =	100 psf	**X sq. ft. =**	0	lbs

Point Loads

Beam #	**LL**	**DL**	**LL Type**
RB-3	4140 lbs	5580 lbs	ROOF
			ROOF

Lateral Loads

	X	**Y**	
Roof DL =	0	0	**lbs**
Lr =	0	0	**lbs**
S =	0	0	**lbs**
R =	0	0	**lbs**
FLOOR DL =	0	0	**lbs**
LL =	0	0	**lbs**
Wind =	0	0	**lbs**
Seismic =	0	0	**lbs**

Uplift = 0 lbs
Additional wt resisting uplift = 0 lbs
Footing wt = 1088 lbs

FOOTING DESIGN CALCULATIONS - REINFORCED CONCRETE

Footing Flexural Design (LRFD)

Governed by 1.6*Q_{Max}

f'c = 2500 **psi**
fy = 60000 **psi**
Factored soil bearing pressure (1.6*Qa), Qu = 1922 **psf**
Footing factored moment, Mu = 1501 **lb-ft/ft**
Min. clear distance = 3.00 **in**
Footing thickness = 10 **in**
d = 6.75 **in**
Starting guess for As = Mu/4d = 0.056 **in2**
spacing = 10.00 **in o.c.**
Rebar size = 4
As = 0.240 **in^2**
As(min) = 0.0018*b*d = 0.648 **in^2**
As actual = 0.800 **in^2** OK
a = (As*fy)/(0.85*f'c*b) = 0.565 **in**
Footing factored moment, Mu = 1501 **lb-ft/ft**
ΦMn = 0.9*As*fy*(d-a/2) = 6985 **lb-ft/ft** OK
Conc. Ult. compressive strain, εcu = 0.003
β1 = 0.85
c = 0.664
Strain in steel, εt = (εcu(d-a/β1))/(a/β1) = 0.027
εt > 0.004 (ACI Requirement) = OK
εt > 0.005 (Tension controlled) = OK

Footing Two Way Shear Design (LRFD)

Governed by 1.6*Q_{Max}

Pu = 17294 **lbs**
Mu = 0 **lb-ft**
Column Embedment = 0.00 **in**
d = 6.75 **in**
Vu1 = Pu*(1-b1*b2/(B*L)) = 15125 **lbs**
γv = 1-1/(1+2/3*(b1/b2)^1/2) = 0.400
Perimeter of shear failure = bo = 51 **in**
Vu = [Vu1/(d*bo)+γv*Mu*y/Jc]*(d*bo) = 15125 **lbs**
β = short side of column/long side of column = 1.00
ΦVc = 0.75*(2+4/β)*(f'c)^0.5*bo*d = 77456 **lbs**
ΦVc = 0.75*(αsd/bo+2)*(f'c)^0.5*bo*d = 94163 **lbs**
ΦVc = 0.75*4*(f'c)^0.5*bo*d = 51638 **lbs**
ΦVc = 51638 **lbs**
OK

Footing One Way Shear Design (LRFD)

Governed by 1.2D+(f1*L or 0.5W)+1.6*(Lr or S or R)

Vu = Qu*((w-column thickness)/2-d) = 1321 **plf**
ΦVc = 0.75*2*(f'c)^.5*b*d = 6075 **plf**
OK

Footing Uplift Design (ASD)

Governed by 0.6D+(0.6W or 0.7E)+0.6*footing wt+0.6*Bulk wt

Uplift force = 0 **lbs**
Concrete unit weight = 145 **pcf**
Total resisting dead load = 6668 **lbs**
0.6*D = 4001 **lbs**
OK

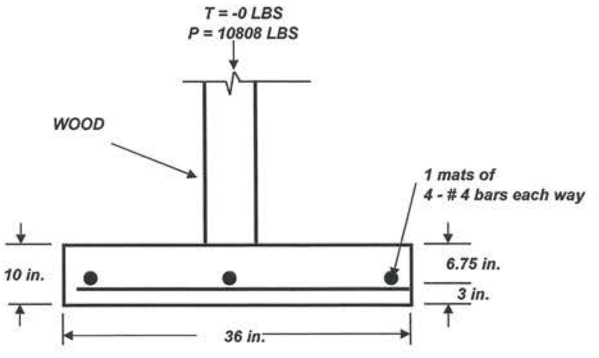
FOOTING DESIGN AND LOAD COMBINATION SUMMARY AND SCHEMATIC FOOTING DETAIL

	Actual	**Allowable**		
Footing Bearing pressure =	1201	1500	**psf**	OK
Footing One Way Shear =	1321	6075	**lbs/ft**	OK
Footing Two Way Shear =	15125	51638	**lbs**	OK
Footing Moment =	1501	6985	**lb-ft/ft**	OK

Footing Bearing Pressure Summary by Load Combination

Load combination	Q_{max}	Q_{min}	Qu_{max}	Qu_{min}
D	741	741	1037	1037
D+L	741	741	1186	1186
D+(Lr or S or R)	***1201***	***1201***	***1922***	***1922***
D+0.75L+0.75(Lr or S or R)	1086	1086	1738	1738
D+(0.6W or 0.7E)	741	741	1186	1186
D+0.75+0.75*0.6W+0.75L+0.75(Lr or S or R)	1086	1086	1738	1738
D+0.75+0.75*0.7E+0.75L+0.75S	741	741	1186	1186
0.6D+0.6W	445	445	712	712
0.6D+0.7E	445	445	712	712

Design by: LRP　　Date: 6/1/2020 18:36



HSS 8×6×1/4

Simply Supported Steel Beam Design (LRFD) v7.3.2 (4-27-17)
PROJECT MILAN LOT 2 (SHEAR TRANSFER COLUMN)

Material Properties and Loading

	Cantilever	Span	Cantilever	Total Length
Beam span =		14.00	2.00	16.00
Braced Length, Lb =		2.00	ft	
Steel Shape =		HSS		
Fy =		46000	psi	
E =		29000000	psi	

Section req'd for M = HSS10X2X1/8
Section to use = HSS6X6X3/16
Section Orientation = Strong
Bearing Connection

Local Buckling Criteria
Flange = Non-Compact
Web = Compact
Non-compact section

ΔTL Governs Design 94%

Unfactored Load Reactions			
Load type	R_A	R_B	
D =	-389	4039	*lbs*
L =	0	0	*lbs*
Lr =	0	0	*lbs*
S =	0	0	*lbs*
R =	0	0	*lbs*
W =	0	0	*lbs*
E =	0	0	*lbs*

Point Loads

	P_L	P_D	P_T	a	LL Type	Load description
P_1 =		3418	3418	16.00	Roof	
P_2 =			0			
P_3 =			0			
P_4 =			0			
P_5 =			0			
P_6 =			0			
P_7 =			0			

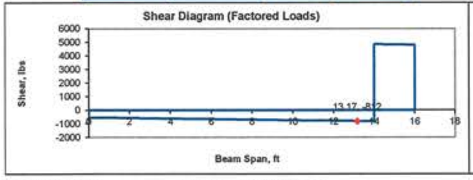
Roof slope: 4 :12

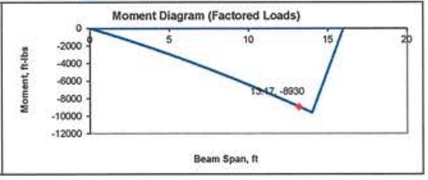
Uniform Loads

Live, psf	Dead, psf	Trib. Width		W_L	W_D	W_T	Start @	End @	LL Type	Load description
20	15	0.00	W_1 =	0	0	0	0.00	14.00	Roof	
			W_2 =			0				
			W_3 =			0				
			W_4 =			0				
			W_5 =			0				
			W_6 =			0				

Triangular Loads (Starting or ending load must be 0)

	Start W_L	Start W_D	End W_L	End W_D	Start W_T	End W_T	Start @	End @	LL Type	Load description
T_1 =	0	0	0	0	0	0	0.00	16.00	Roof	
T_2 =					0	0				
T_3 =					0	0				
T_4 =					0	0				





Bending Design Design controlled by 1.4D

F2. Doubly Symmetric Compact I-Shaped Members and channels bent about their major axis

1. Yielding

*ΦMn = 0.9*FyZx*
Fy = 46000 *psi*
Zx = 8.63 *in^3*
Mp = FyZx = 396980 *lb-in*
*ΦMn = 0.9*Mp* = 29774 *lb-ft*

2. Lateral Torsional Buckling

M_p = 396980 *lb-in*
L_b = 2.000 *ft*
M_{max} = -9611 *lb-ft*
M_A = 0 *lb-ft*
M_B = 0 *lb-ft*
M_C = 0 *lb-ft*
F_L = 36000 *psi*
Mr = FLSx = 22260 *lb-ft*
R_m = 1.00
h_o = 6
c = 0.00
$r_{ts} = (I_y{*}C_w)^{1/2}/S_x$ = 0.000
$F_{cr} = (C_b\pi^{\wedge 2}E)/(L_b/r_{ts})^2{*}(1{+}0.078{*}Jc/(S_xh_o){*}(L_b/r_{ts})^{\wedge 2})^{\wedge 1/2}$ = #DIV/0!
L_r = 0.00 *ft*
L_p = 0.000 *ft*
C_b = 1.000
Lr < Lb, Mn = FcrSx ≤ Mp, Mp = #DIV/0! *lb-in*
*ΦMn = 0.9*Mp* = #DIV/0! *lb-ft*

Beam Section Properties

A_g =	3.98	in^2
depth =	6.0	in
b_f =	6.0	in
t_f =	0.19	in
t_w =	0.1875	in
Ix =	22.30	in^4
Iy =	0	in4
Zx =	8.63	in^3
Sx =	7.42	in^3
r_x =	2.370	in
r_y =	0.00	in
C_w =	0	in6
J =	35	in4
G =	11200	ksi
b/t =	31.50	
h/t =	31.50	
	Flange	Web
λp =	28.12	60.76
λr =	35.15	143.12

Mu = 9611 *lb-ft*
ΦMn = ***#DIV/0!*** *lb-ft*
#DIV/0!

G4. Shear Design

Aw = 2.215 *in^2*
h/tw = 31.50
kv = 5.00
Cv = 1.00
Φ = 0.90
*ΦVn = 0.9(0.6)Fy*Aw*Cv* = 55017 *lbs*
V_u = 4826 *lbs*
OK

For webs of rolled I-shaped members with h/tw ≤ 260, Φ = 1.0 and Cv = 1.0
For webs of all other doubly and singly symmetric shapes and channels, Cv equals:
For h/tw ≤ 1.10*sqrt(kv*E/Fy), Cv = 1.00
For 1.10*sqrt(kv*E/Fy) < h/tw ≤ 1.37*sqrt(kv*E/Fy), Cv = 1.10*sqrt(kv*E/Fy)/(h/tw)
For h/tw > 1.37*sqrt(kv*E/Fy), Cv = 1.51*E*kv/((h/tw)2Fy)
1.10(kv*E/Fy)1/2* = 61.76
1.37(kv*E/Fy)1/2* = 76.92

Web local crippling

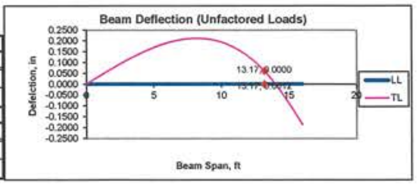
if lb/d ≤ 0.2, $\Phi Rn = 0.75{*}(0.40{*}tw^2[1{+}3(lb/d)(t_w/t_f)^{1.5}]{*}sqrt(E{*}Fywt_f/t_w))$
if lb/d > 0.2, $\Phi Rn = 0.75{*}(0.40{*}tw^2[1{+}(4lb/d{-}0.2)(t_w/t_f)^{1.5}]{*}sqrt(E{*}Fywt_f/t_w))$
lb/d = 0.167
ΦRn = 18272 *lbs*
OK

Web local yielding

Bearing length, lb = 1.00 *in*
ΦRn = 1.0(5k+lb)*Fyw*tw* = 8625 *lbs*
OK

Deflection Camber = 0.00 *in*

Mid Span Deflection					
Loading	$Ratio_{allow}$	$\Delta_{allowed}$	Δ_{actual}	$Ratio_{actual}$	Check
Δ_{LL}	360	0.467	0.000	#DIV/0!	OK
Δ_{TL}	240	0.700	0.212	L/792	OK
Cantilever Deflection					
Loading	$Ratio_{allow}$	$\Delta_{allowed}$	Δ_{actual}	$Ratio_{actual}$	Check
Δ_{LL}	180	0.133	0.000	N/A	OK
Δ_{TL}	120	0.200	0.187	L/128	OK



HSS 6x6x3/16

Simply Supported Steel Beam Design (LRFD) v7.3.2 (4-27-17)
PROJECT MILAN LOT 2 (SHEAR TRANSFER COLUMN)

Material Properties and Loading

	Cantilever	Span	Cantilever	Total Length
Beam span =		9.00	7.00	16.00

Braced Length, Lb = 2.00 ft
Steel Shape = HSS
Fy = 46000 psi
E = 29000000 psi

Section req'd for M = HSS9X9X1/8
Section to use = HSS8X6X1/4
Section Orientation = Strong
Bearing Connection

Local Buckling Criteria
Flange = Compact
Web = Compact
Compact Section

ΔTL Governs Design 86%

Unfactored Load Reactions

Load type	R_A	R_B	
D =	-1603	4073	*lbs*
L =	0	0	*lbs*
Lr =	0	0	*lbs*
S =	0	0	*lbs*
R =	0	0	*lbs*
W =	0	0	*lbs*
E =	0	0	*lbs*

Point Loads

	P_L	P_D	P_T	a	LL Type	Load description
P_1 =		2112	2112	16.00	Roof	
P_2 =			0			
P_3 =			0			
P_4 =			0			
P_5 =			0			
P_6 =			0			
P_7 =			0			

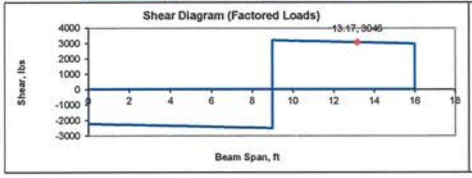
Roof slope: 4 :12

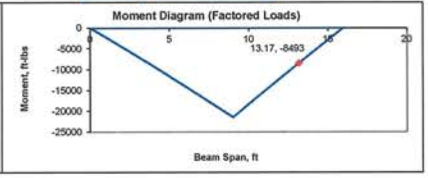
Uniform Loads

Live, psf	Dead, psf	Trib. Width		W_L	W_D	W_T	Start @	End @	LL Type	Load description
20	15	0.00	W_1 =	0	0	0	0.00	14.00	Roof	
			W_2 =			0				
			W_3 =			0				
			W_4 =			0				
			W_5 =			0				
			W_6 =			0				

Triangular Loads (Starting or ending load must be 0)

	Start W_L	Start W_D	End W_L	End W_D	Start W_T	End W_T	Start @	End @	LL Type	Load description
T_1 =	0	0	0	0	0	0	0.00	16.00	Roof	
T_2 =					0	0				
T_3 =					0	0				
T_4 =					0	0				


Shear Diagram (Factored Loads)


Moment Diagram (Factored Loads)

Bending Design **Design controlled by 1.4D**

F7. Square and rectangular HSS and box shaped members

1. Yielding *ΦMn = 0.9*FyZx*

Fy = 46000 *psi*
Zx = 16.90 *in³*
Mp = FyZx = 777400 *lb-in*
*ΦMn = 0.9*Mp =* 58305 *lb-ft*

2b. Flange local buckling (Non compact flanges)
ΦMn = 0.9(Mp - (Mp-FySx)*(3.57b/t*sqrt(Fy/E)-4.0) <= Mp)*
Mp = FyZx = 777400 *lb-in*
ΦMn = 58305 *lb-ft*

2c. Flange local buckling (Slender flanges)
be = 1.92tf*sqrt(E/Fy)*(1-0.38/(b/tf)*sqrt(E/Fy))<=b
be = 6.00 *in*
Se = 14.200 *in³*
*ΦMn = 0.9*Fy*Se =* 48990 *lb-ft*

3. Web local buckling (Non compact and slender webs)
ΦMn = 0.9(Mp - (Mp-FySx)*(0.305*h/tw*sqrt(Fy/E)-0.738) <= Mp)*
ΦMn = 58305 *lb-ft*

Mu = 21466 *lb-ft*
ΦMn = *58305* *lb-ft*
OK

Beam Section Properties
A_g = 6.17 in^2
depth = 8.0 in
b_f = 6.0 in
t_f = 0.25 in
t_w = 0.25 in
Ix = 56.60 in^4
Iy = 36.4 in4
Zx = 16.90 in^3
Sx = 14.20 in^3
r_x = 3.030 in
r_y = 2.4 in
C_w = 0.25 in^6
J = 70.3 in^4
G = 11200 ksi
b/t = 22.80
h/t = 31.30

	Flange	Web
λp =	28.12	60.76
λr =	35.15	143.12

G4. Shear Design

Aw = 3.913 *in²*
h/tw = 31.30
kv = 5.00
Cv = 1.00
Φ = 0.90
*ΦVn = 0.9(0.6)Fy*Aw*Cv =* 97187 *lbs*
V_u = 3176 *lbs*
OK

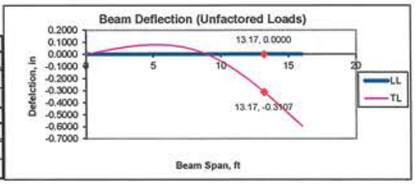
For webs of rolled I-shaped members with h/tw ≤ 260, Φ = 1.0 and Cv = 1.0
For webs of all other doubly and singly symmetric shapes and channels, Cv equals:
For h/tw ≤ 1.10*sqrt(kv*E/Fy), Cv = 1.00
For 1.10*sqrt(kv*E/Fy) < h/tw ≤ 1.37*sqrt(kv*E/Fy), Cv = 1.10*sqrt(kv*E/Fy)/(h/tw)
For h/tw > 1.37*sqrt(kv*E/Fy), Cv = 1.51*E*kv/((h/tw)2Fy)
1.10(kv*E/Fy)1/2 =* 61.76
1.37(kv*E/Fy)1/2 =* 76.92

Web local crippling
if lb/d ≤ 0.2, ΦRn = 0.75*(0.40*tw^2[1+3(lb/d)(t_w/t_f)$^{1.5}$]*sqrt(E*Fywt$_f$/t$_w$))
if lb/d > 0.2, ΦRn = 0.75*(0.40*tw^2[1+(4lb/d-0.2)(t_w/t_f)$^{1.5}$]*sqrt(E*Fywt$_f$/t$_w$))
lb/d = 0.125
ΦRn = 29777 *lbs*
OK

Web local yielding
Bearing length, lb = 1.00 *in*
ΦRn = 1.0(5k+lb)*Fyw*tw =* 11500 *lbs*
OK

Deflection Camber = 0.00 *in*

Mid Span Deflection					
Loading	$Ratio_{allow}$	$\Delta_{allowed}$	Δ_{actual}	$Ratio_{actual}$	Check
Δ_{LL}	360	0.300	0.000	#DIV/0!	*OK*
Δ_{TL}	240	0.450	0.082	L/1318	*OK*
Cantilever Deflection					
Loading	$Ratio_{allow}$	$\Delta_{allowed}$	Δ_{actual}	$Ratio_{actual}$	Check
Δ_{LL}	180	0.467	0.000	N/A	*OK*
Δ_{TL}	120	0.700	0.595	L/141	*OK*


Beam Deflection (Unfactored Loads)

SPOT FOOTING DESIGN V7.4.0 (4/21/15)
PROJECT: MILAN LOT 2 **DESCRIPTION:**

FOOTING LOADS AND BEARING CALCULATIONS

Footing Bearing Calculations (ASD)
Governed by D+L

Total vertical load, P = 7879 **lbs**
Max bearing capacity, Qa = 1500 **psf**
Footing w and l = 30 **in.**
Lateral Load = 0 **lbs**
Column Height = 10.00 **ft**
M = 0 **lb-ft**
e = M/P = 0.00 **ft**
w/6 = 0.417 **ft**
Qmax = P/A + Mx/Sx + My/Sy = 1261 **psf**
Qmin = P/A - Mx/Sx - My/Sy = 1261 **psf**
OK

Point Loads

Beam #	**LL**	**DL**	**LL Type**
FB-3	5281 lbs	1843 lbs	FLOOR

Uniform Loads

Roof DL =	15 psf	**X 0 sq. ft. =**	0	lbs
Roof LL =	20 psf	**X 0 sq. ft. =**	0	lbs
Snow load =	20 psf	**X 0 sq. ft. =**	0	lbs
Rain load =	5 psf	**X 0 sq. ft. =**	0	lbs
Floor DL =	15 psf	**X sq. ft. =**	0	lbs
Floor LL =	70 psf	**X sq. ft. =**	0	lbs
Framed wall DL =	15 psf	**X sq. ft. =**	0	lbs
Concrete wall DL =	100 psf	**X sq. ft. =**	0	lbs

Lateral Loads

	X	**Y**	
Roof DL =	0	0	**lbs**
Lr =	0	0	**lbs**
S =	0	0	**lbs**
R =	0	0	**lbs**
FLOOR DL =	0	0	**lbs**
LL =	0	0	**lbs**
Wind =	0	0	**lbs**
Seismic =	0	0	**lbs**

Uplift = 0 lbs
Additional wt resisting uplift = 0 lbs
Footing wt = 755 lbs

FOOTING DESIGN CALCULATIONS - REINFORCED CONCRETE

Footing Flexural Design (LRFD)
Governed by 1.6*Q $_{Max}$

f'c = 2500 **psi**
fy = 60000 **psi**
Factored soil bearing pressure (1.6*Qa), Qu = 2018 **psf**
Footing factored moment, Mu = 1184 **lb-ft/ft**
Min. clear distance = 3.00 **in**
Footing thickness = 10 **in**
d = 6.75 **in**
Starting guess for As = Mu/4d = 0.044 **in2**
spacing = 12.00 **in o.c.**
Rebar size = 4
As = 0.200 **in^2**
As(min) = 0.0018*b*d = 0.540 **in^2**
As actual = 0.600 **in^2** **OK**
a = (As*fy)/(0.85*f'c*b) = 0.471 **in**
Footing factored moment, Mu = 1184 **lb-ft/ft**
ΦMn = 0.9*As*fy*(d-a/2) = 5863 **lb-ft/ft** **OK**
Conc. Ult. compressive strain, εcu = 0.003
β1 = 0.85
c = 0.554
Strain in steel, εt = (εcu(d-a/β1))/(a/β1) = 0.034
εt > 0.004 (ACI Requirement) = **OK**
εt > 0.005 (Tension controlled) = **OK**

Footing Two Way Shear Design (LRFD)
Governed by 1.6*Q $_{Max}$

Pu = 12610 **lbs**
Mu = 0 **lb-ft**
Column Embedment = 0.00 **in**
d = 6.75 **in**
Vu1 = Pu*(1-b1*b2/(B*L)) = 10991 **lbs**
γv = 1-1/(1+2/3*(b1/b2)^1/2) = 0.400
Perimeter of shear failure = bo = 43 **in**
Vu = [Vu1/(d*bo)+γv*Mu*y/Jc]*(d*bo) = 10991 **lbs**
β = short side of column/long side of column = 1.00
ΦVc = 0.75*(2+4/β)*(f'c)^0.5*bo*d = 65306 **lbs**
ΦVc = 0.75*(αsd/bo+2)*(f'c)^0.5*bo*d = 90113 **lbs**
ΦVc = 0.75*4*(f'c)^0.5*bo*d = 43538 **lbs**
ΦVc = 43538 **lbs**
OK

Footing One Way Shear Design (LRFD)
Governed by 1.2D+1.6*L+0.5*(Lr or S or R)

Vu = Qu*((w-column thickness)/2-d) = 1051 **plf**
ΦVc = 0.75*2*(f'c)^.5*b*d = 6075 **plf**
OK

Footing Uplift Design (ASD)
Governed by 0.6D+(0.6W or 0.7E)+0.6*footing wt+0.6*Bulk wt

Uplift force = 0 **lbs**
Concrete unit weight = 145 **pcf**
Total resisting dead load = 2598 **lbs**
0.6*D = 1559 **lbs**
OK

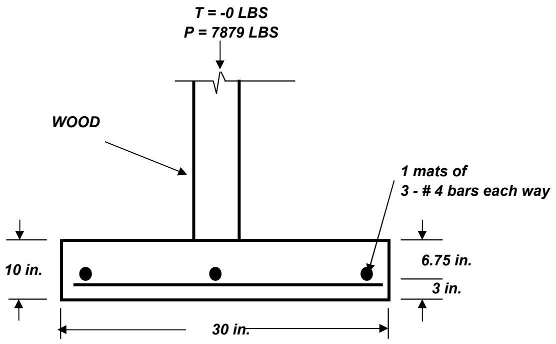
FOOTING DESIGN AND LOAD COMBINATION SUMMARY AND SCHEMATIC FOOTING DETAIL

	Actual	**Allowable**		
Footing Bearing pressure =	1261	1500	**psf**	**OK**
Footing One Way Shear =	1051	6075	**lbs/ft**	**OK**
Footing Two Way Shear =	10991	43538	**lbs**	**OK**
Footing Moment =	1184	5863	**lb-ft/ft**	**OK**

Footing Bearing Pressure Summary by Load Combination

Load combination	Q_{max}	Q_{min}	Qu_{max}	Qu_{min}
D	416	416	582	582
D+L	**1261**	**1261**	**2018**	**2018**
D+(Lr or S or R)	416	416	666	666
D+0.75L+0.75(Lr or S or R)	1050	1050	1680	1680
D+(0.6W or 0.7E)	416	416	666	666
D+0.75+0.75*0.6W+0.75L+0.75(Lr or S or R)	1050	1050	1680	1680
D+0.75+0.75*0.7E+0.75L+0.75S	1050	1050	1680	1680
0.6D+0.6W	250	250	400	400
0.6D+0.7E	250	250	400	400

Design by: LRP Date: 3/28/2021 23:44



SPOT FOOTING DESIGN V7.4.0 (4/21/15)

PROJECT: MILAN LOT 2 ***DESCRIPTION:***

FOOTING LOADS AND BEARING CALCULATIONS

Footing Bearing Calculations (ASD)

Governed by D+(Lr or S or R)

Total vertical load, P = *17434* ***lbs***
Max bearing capacity, Qa = *1500* ***psf***
Footing w and l = *42* ***in.***
Lateral Load = *0* ***lbs***
Column Height = *10.00* ***ft***
M = *0* ***lb-ft***
e = M/P = *0.00* ***ft***
w/6 = *0.583* ***ft***
Qmax = P/A + Mx/Sx + My/Sy = *1424* ***psf***
Qmin = P/A - Mx/Sx - My/Sy = *1424* ***psf***
OK

Point Loads

Beam #	***LL***	***DL***	***LL Type***
RB-3	*6233 lbs*	*9721 lbs*	*ROOF*

Uplift = *0* *lbs*
Additional wt resisting uplift = *0* *lbs*
Footing wt = *1480* *lbs*

Uniform Loads

Roof DL = *15 psf* ***X 0 sq. ft. =*** 0 *lbs*
Roof LL = *20 psf* ***X 0 sq. ft. =*** 0 *lbs*
Snow load = *20 psf* ***X 0 sq. ft. =*** 0 *lbs*
Rain load = *5 psf* ***X 0 sq. ft. =*** 0 *lbs*
Floor DL = *15 psf* ***X sq. ft. =*** 0 *lbs*
Floor LL = *70 psf* ***X sq. ft. =*** 0 *lbs*
Framed wall DL = *15 psf* ***X sq. ft. =*** 0 *lbs*
Concrete wall DL = *100 psf* ***X sq. ft. =*** 0 *lbs*

Lateral Loads

	X	***Y***	
Roof DL =	*0*	*0*	***lbs***
Lr =	*0*	*0*	***lbs***
S =	*0*	*0*	***lbs***
R =	*0*	*0*	***lbs***
FLOOR DL =	*0*	*0*	***lbs***
LL =	*0*	*0*	***lbs***
Wind =	*0*	*0*	***lbs***
Seismic =	*0*	*0*	***lbs***

FOOTING DESIGN CALCULATIONS - REINFORCED CONCRETE

Footing Flexural Design (LRFD)

*Governed by 1.6*Q $_{Max}$*

f'c = *2500* ***psi***
fy = *60000* ***psi***
Factored soil bearing pressure (1.6*Qa), Qu = *2278* ***psf***
Footing factored moment, Mu = *2856* ***lb-ft/ft***
Min. clear distance = *3.00* ***in***
Footing thickness = *10* ***in***
d = *6.75* ***in***
Starting guess for As = Mu/4d = *0.106* ***in2***
spacing = *12.00* ***in o.c.***
Rebar size = *4*
As = *0.200* ***in^2***
As(min) = 0.0018*b*d = *0.756* ***in^2***
As actual = *0.800* ***in^2*** ***OK***
a = (As*fy)/(0.85*f'c*b) = *0.471* ***in***
Footing factored moment, Mu = *2856* ***lb-ft/ft***
ΦMn = 0.9*As*fy*(d-a/2) = *5863* ***lb-ft/ft*** ***OK***
Conc. Ult. compressive strain, εcu = *0.003*
β1 = *0.85*
c = *0.554*
Strain in steel, εt = (εcu(d-a/β1))/(a/β1) = *0.034*
εt > 0.004 (ACI Requirement) = ***OK***
εt > 0.005 (Tension controlled) = ***OK***

Footing Uplift Design (ASD)

*Governed by 0.6D+(0.6W or 0.7E)+0.6*footing wt+0.6*Bulk wt*

Uplift force = *0* ***lbs***
Concrete unit weight = *145* ***pcf***
Total resisting dead load = *11201* ***lbs***
0.6*D = *6721* ***lbs***
OK

Footing Two Way Shear Design (LRFD)

*Governed by 1.6*Q $_{Max}$*

Pu = *27910* ***lbs***
Mu = *0* ***lb-ft***
Column Embedment = 0.00 ***in***
d = 6.75 ***in***
Vu1 = Pu*(1-b1*b2/(B*L)) = 26082 ***lbs***
γv = 1-1/(1+2/3*(b1/b2)^1/2) = 0.400
Perimeter of shear failure = bo = *43* ***in***
Vu = [Vu1/(d*bo)+γv*Mu*y/Jc]*(d*bo) = *26082* ***lbs***
β = short side of column/long side of column = *1.00*
ΦVc = 0.75*(2+4/β)*(f'c)^0.5*bo*d = *65306* ***lbs***
ΦVc = 0.75*(αsd/bo+2)*(f'c)^0.5*bo*d = *90113* ***lbs***
ΦVc = 0.75*4*(f'c)^0.5*bo*d = *43538* ***lbs***
ΦVc = *43538* ***lbs***
OK

Footing One Way Shear Design (LRFD)

*Governed by 1.2D+(f1*L or 0.5W)+1.6*(Lr or S or R)*

Vu = Qu*((w-column thickness)/2-d) = *2326* ***plf***
ΦVc = 0.75*2*(f'c)^.5*b*d = *6075* ***plf***
OK

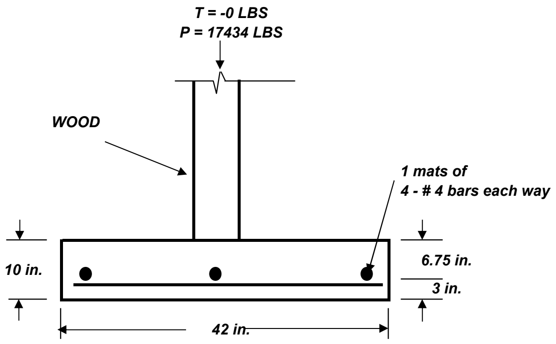
FOOTING DESIGN AND LOAD COMBINATION SUMMARY AND SCHEMATIC FOOTING DETAIL

	Actual	***Allowable***		
Footing Bearing pressure =	1424	*1500*	***psf***	***OK***
Footing One Way Shear =	*2326*	*6075*	***lbs/ft***	***OK***
Footing Two Way Shear =	*26082*	*43538*	***lbs***	***OK***
Footing Moment =	*2856*	*5863*	***lb-ft/ft***	***OK***

Footing Bearing Pressure Summary by Load Combination

Load combination	Q_{max}	Q_{min}	Qu_{max}	Qu_{min}
D	915	915	1281	1281
D+L	915	915	1464	1464
D+(Lr or S or R)	***1424***	***1424***	***2278***	***2278***
D+0.75L+0.75(Lr or S or R)	1296	1296	2074	2074
D+(0.6W or 0.7E)	915	915	1464	1464
D+0.75+0.75*0.6W+0.75L+0.75(Lr or S or R)	1296	1296	2074	2074
D+0.75+0.75*0.7E+0.75L+0.75S	915	915	1464	1464
0.6D+0.6W	549	549	878	878
0.6D+0.7E	549	549	878	878

Design by: LRP Date: 3/28/2021 23:42

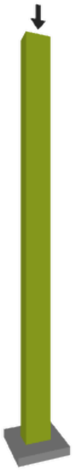


MEMBER REPORT

PASSED

Level, COL RB-3

1 piece(s) 6 x 6 Douglas Fir-Larch No. 2



Drawing is Conceptual

Post Height: 9'

Design Results	Actual	Allowed	Result	LDF	Load: Combination
Slenderness	20	50	Passed (39%)	--	--
Compression (lbs)	15954	15693	Passed (102%)	0.90	1.0 D
Base Bearing (lbs)	15954	980100	Passed (2%)	--	1.0 D
Bending/Compression	N/A	1	Passed (N/A)	--	N/A

- Input axial load eccentricity for the design is zero
- Applicable calculations are based on NDS.

Supports	Type	Material
Base	Beam	Steel

Member Type : Free Standing Post
Building Code : IBC 2018
Design Methodology : ASD

Max Unbraced Length	Comments
Full Member Length	No bracing assumed.

Vertical Load	Dead (0.90)	Comments
1 - Point (lb)	15954	Default Load

Weyerhaeuser Notes

ForteWEB Software Operator	Job Notes
L RIED POPE PE, PLS LR POPE ENGINEERING INC (435) 628-1676 lrpope@lrpope.com	