Renovate and Modernize HVAC Systems
Building 6
Wichita, KS

Structural Design Calculations

Prepared For:
M+HFG Architecture
Wichita, KS

PEC Project No.:
180248-001

Prepared by:
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License No. 23587

Date:
July 26, 2018

PEC
PROFESSIONAL ENGINEERING CONSULTANTS PA
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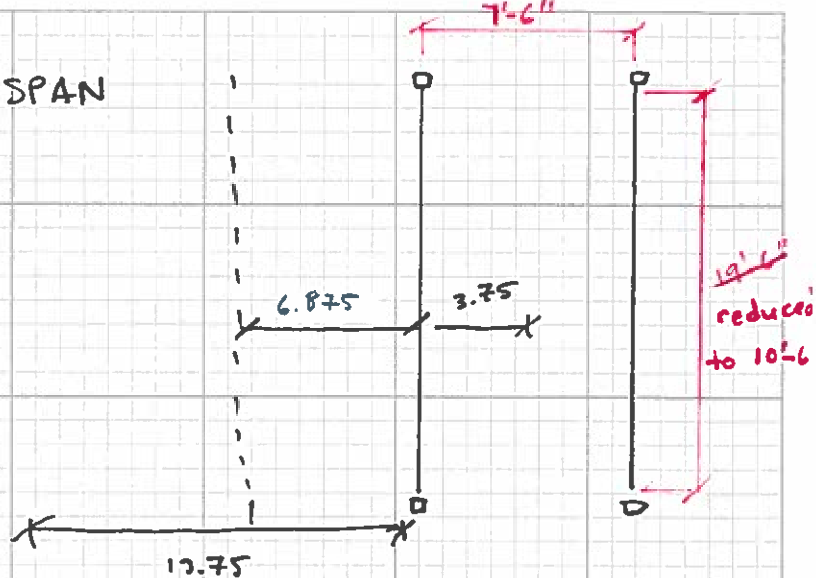
Project 180248-001 : Dole VA   Date ________
Item PSL Beam (Attic Floor Framing)   By ________

26'-6" - 7'-0" = 19'-6" SPAN

SPACE IN BETWEEN — 7'-6"



Trib = 10.625'

Load from Attic

DL : Plaster + wood lathe : 10 psf

1" Floor : 2.5 psf

Carpet + pad : 2 psf

2x10 @ 16" : 2.6 psf

Mech/Elec : 3 psf

20.1 psf

Convert to PLF

Total Load : 20.1 × 10.625 = 213.6 PLF

40 × 10.625 = 425 PLF

638.6 PLF

Live Load : 425 PLF

Beam Analysis

Reactions :	#1	#2
DL :	1.25	1.6
L :	2.23	2.23

PSL ~~3½ x 18" deep~~ → 3½ × 9¼ (after reduction)

Max Shear = ~~6.736^k~~ 3.781^k

Max Moment = ~~35.43~~ 9.179^k-ft

Wood Beam

File = U:\Wichita-Facil\2018\180248\001\Struct\Calcs\180248 dole va building 6.ec6
ENERCALC, INC. 1983-2017, Build:6.17.3.29, Ver:6.17.3.29

Lic. # : KW-06005112 **Licensee : PEC**

Description : 3.5" x 9.25" WOOD BEAM (W/ AHU)

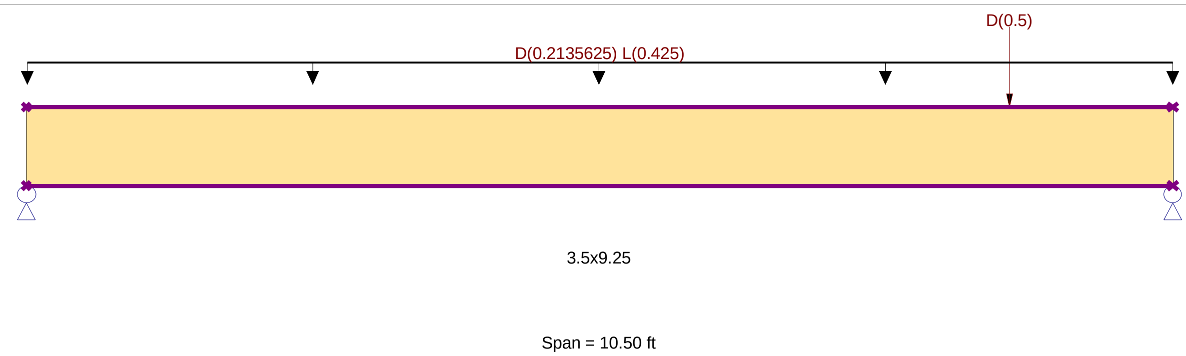
CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : IBC 2015

Material Properties

Analysis Method :	Allowable Stress Design	Fb - Tension	2,900.0 psi	*E : Modulus of Elasticity*	
Load Combination :	IBC 2015	Fb - Compr	2,900.0 psi	Ebend- xx	2,000.0 ksi
		Fc - Prll	2,900.0 psi	Eminbend - xx	1,016.54 ksi
Wood Species	: Trus Joist	Fc - Perp	750.0 psi		
Wood Grade	: Parallam PSL 2.0E	Fv	290.0 psi		
		Ft	2,025.0 psi	Density	45.050 pcf

Beam Bracing : Beam is Fully Braced against lateral-torsional buckling



Applied Loads

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

Beam self weight calculated and added to loads
Uniform Load : D = 0.02010, L = 0.040 ksf, Tributary Width = 10.625 ft
Point Load : D = 0.50 k @ 9.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	0.773 : 1	Maximum Shear Stress Ratio	=	0.533 : 1
Section used for this span		**3.5x9.25**	Section used for this span		**3.5x9.25**
fb : Actual	=	2,240.45 psi	fv : Actual	=	154.61 psi
FB : Allowable	=	2,900.00 psi	Fv : Allowable	=	290.00 psi
Load Combination		+D+L+H	Load Combination		+D+L+H
Location of maximum on span	=	5.365 ft	Location of maximum on span	=	9.734 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.253 in	Ratio =	497 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.405 in	Ratio =	310 >=240
Max Upward Total Deflection	0.000 in	Ratio =	0 <240

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
+D+H													0.00	0.00	0.00	0.00
Length = 10.50 ft	1	0.320	0.254	0.90	1.000	1.00	1.00	1.00	1.00	1.00	3.47	834.07	2610.00	1.43	66.32	261.00
+D+L+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 10.50 ft	1	0.773	0.533	1.00	1.000	1.00	1.00	1.00	1.00	1.00	9.32	2,240.45	2900.00	3.34	154.61	290.00
+D+Lr+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 10.50 ft	1	0.230	0.183	1.25	1.000	1.00	1.00	1.00	1.00	1.00	3.47	834.07	3625.00	1.43	66.32	362.50
+D+S+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 10.50 ft	1	0.250	0.199	1.15	1.000	1.00	1.00	1.00	1.00	1.00	3.47	834.07	3335.00	1.43	66.32	333.50
+D+0.750Lr+0.750L+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 10.50 ft	1	0.521	0.366	1.25	1.000	1.00	1.00	1.00	1.00	1.00	7.86	1,888.58	3625.00	2.86	132.54	362.50

Description : 3.5" x 9.25" WOOD BEAM (W/ AHU)

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
+D+0.750L+0.750S+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 10.50 ft	1	0.566	0.397	1.15	1.000	1.00	1.00	1.00	1.00	1.00	7.86	1,888.58	3335.00	2.86	132.54	333.50
+D+0.60W+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 10.50 ft	1	0.180	0.143	1.60	1.000	1.00	1.00	1.00	1.00	1.00	3.47	834.07	4640.00	1.43	66.32	464.00
+D+0.70E+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 10.50 ft	1	0.180	0.143	1.60	1.000	1.00	1.00	1.00	1.00	1.00	3.47	834.07	4640.00	1.43	66.32	464.00
+D+0.750Lr+0.750L+0.450W+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 10.50 ft	1	0.407	0.286	1.60	1.000	1.00	1.00	1.00	1.00	1.00	7.86	1,888.58	4640.00	2.86	132.54	464.00
+D+0.750L+0.750S+0.450W+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 10.50 ft	1	0.407	0.286	1.60	1.000	1.00	1.00	1.00	1.00	1.00	7.86	1,888.58	4640.00	2.86	132.54	464.00
+D+0.750L+0.750S+0.5250E+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 10.50 ft	1	0.407	0.286	1.60	1.000	1.00	1.00	1.00	1.00	1.00	7.86	1,888.58	4640.00	2.86	132.54	464.00
+0.60D+0.60W+0.60H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 10.50 ft	1	0.108	0.086	1.60	1.000	1.00	1.00	1.00	1.00	1.00	2.08	500.44	4640.00	0.86	39.79	464.00
+0.60D+0.70E+0.60H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 10.50 ft	1	0.108	0.086	1.60	1.000	1.00	1.00	1.00	1.00	1.00	2.08	500.44	4640.00	0.86	39.79	464.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+L+H	1	0.4054	5.288		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	3.477	3.834
Overall MINimum	0.747	0.962
+D+H	1.246	1.603
+D+L+H	3.477	3.834
+D+Lr+H	1.246	1.603
+D+S+H	1.246	1.603
+D+0.750Lr+0.750L+H	2.919	3.276
+D+0.750L+0.750S+H	2.919	3.276
+D+0.60W+H	1.246	1.603
+D+0.70E+H	1.246	1.603
+D+0.750Lr+0.750L+0.450W+H	2.919	3.276
+D+0.750L+0.750S+0.450W+H	2.919	3.276
+D+0.750L+0.750S+0.5250E+H	2.919	3.276
+0.60D+0.60W+0.60H	0.747	0.962
+0.60D+0.70E+0.60H	0.747	0.962
D Only	1.246	1.603
Lr Only		
L Only	2.231	2.231
S Only		
W Only		
E Only		
H Only		

How to Use This Table

1. Calculate total and live load (neglect beam weight) on the beam or header in pounds per linear foot (plf).
2. Select appropriate **Span** (center-to-center of bearing).
3. Scan horizontally to find the proper width, and a depth with a capacity that exceeds actual total and live loads.
4. Review bearing length requirements to ensure adequacy.

*Also see **General Notes** on page 11.*

↓

2.0E Parallam® PSL: Floor—100% (PLF)

Span	Condition	3½" Width							5¼" Width						
		9¼"	9½"	11¼"	11⅞"	14"	16"	18"	9¼"	9½"	11¼"	11⅞"	14"	16"	18"
8'	Total Load	1,469	1,517	1,861	1,990	2,441	2,441	2,441	2,204	2,275	2,792	2,985	3,661	3,661	3,661
	Live Load L/360	1,169	1,257	*	*	*	*	*	1,753	1,886	*	*	*	*	*
	Min. End/Int. Bearing (in.)	2.7/6.8	2.8/7	3.4/8.6	3.7/9.2	4.5/11.3	4.5/11.3	4.5/11.3	2.7/6.8	2.8/7	3.4/8.6	3.7/9.2	4.5/11.3	4.5/11.3	4.5/11.3
9'-6"	Total Load	1,076	1,147	1,510	1,611	1,970	2,052	2,052	1,614	1,720	2,265	2,416	2,955	3,079	3,079
	Live Load L/360	724	780	1,248	1,446	*	*	*	1,086	1,171	1,872	2,170	*	*	*
	Min. End/Int. Bearing (in.)	2.4/5.9	2.5/6.3	3.3/8.3	3.5/8.8	4.3/10.8	4.5/11.3	4.5/11.3	2.4/5.9	2.5/6.3	3.3/8.3	3.5/8.8	4.3/10.8	4.5/11.3	4.5/11.3
10'	Total Load	930	1,003	1,420	1,514	1,848	1,949	1,949	1,395	1,505	2,130	2,271	2,772	2,923	2,923
	Live Load L/360	626	675	1,084	1,257	*	*	*	940	1,013	1,626	1,886	*	*	*
	Min. End/Int. Bearing (in.)	2.1/5.4	2.3/5.8	3.3/8.2	3.5/8.7	4.3/10.6	4.5/11.3	4.5/11.3	2.1/5.4	2.3/5.8	3.3/8.2	3.5/8.7	4.3/10.6	4.5/11.3	4.5/11.3
12'	Total Load	548	592	964	1,092	1,480	1,620	1,620	822	888	1,446	1,639	2,220	2,431	2,431
	Live Load L/360	372	401	651	758	1,198	*	*	558	602	976	1,137	1,797	*	*
	Min. End/Int. Bearing (in.)	1.5/3.8	1.7/4.1	2.7/6.7	3/7.6	4.1/10.3	4.5/11.3	4.5/11.3	1.5/3.8	1.7/4.1	2.7/6.7	3/7.6	4.1/10.3	4.5/11.3	4.5/11.3
14'	Total Load	347	375	616	721	1,093	1,386	1,386	520	563	925	1,082	1,639	2,079	2,079
	Live Load L/360	238	257	419	489	780	1,132	*	357	386	629	734	1,171	1,698	*
	Min. End/Int. Bearing (in.)	1.5/3.5	1.5/3.5	2/5	2.4/5.9	3.5/8.9	4.5/11.3	4.5/11.3	1.5/3.5	1.5/3.5	2/5	2.4/5.9	3.5/8.9	4.5/11.3	4.5/11.3
16'-6"	Total Load	210	228	379	444	720	1,009	1,173	316	342	568	667	1,080	1,514	1,760
	Live Load L/360	147	159	260	305	490	716	995	220	238	391	457	735	1,074	1,493
	Min. End/Int. Bearing (in.)	1.5/3.5	1.5/3.5	1.5/3.7	1.7/4.3	2.8/6.9	3.9/9.7	4.5/11.3	1.5/3.5	1.5/3.5	1.5/3.7	1.7/4.3	2.8/6.9	3.9/9.7	4.5/11.3
18'-6"	Total Load	147	160	268	315	514	759	1,000	221	240	402	473	771	1,138	1,501
	Live Load L/360	105	113	186	218	352	517	722	157	170	280	328	529	776	1,084
	Min. End/Int. Bearing (in.)	1.5/3.5	1.5/3.5	1.5/3.5	1.5/3.5	2.2/5.6	3.3/8.2	4.3/10.8	1.5/3.5	1.5/3.5	1.5/3.5	1.5/3.5	2.2/5.6	3.3/8.2	4.3/10.8
20'	Total Load	115	125	210	248	407	603	850	172	187	316	372	610	905	1,275
	Live Load L/360	83	90	148	174	281	414	579	125	135	223	261	422	621	869
	Min. End/Int. Bearing (in.)	1.5/3.5	1.5/3.5	1.5/3.5	1.5/3.5	1.9/4.8	2.8/7.1	4/9.9	1.5/3.5	1.5/3.5	1.5/3.5	1.5/3.5	1.9/4.8	2.8/7.1	4/9.9
22'	Total Load	84	91	156	184	304	454	642	126	137	234	277	457	681	964
	Live Load L/360	63	68	112	131	213	314	441	94	102	168	197	320	472	662
	Min. End/Int. Bearing (in.)	1.5/3.5	1.5/3.5	1.5/3.5	1.5/3.5	1.6/4	2.4/5.9	3.3/8.3	1.5/3.5	1.5/3.5	1.5/3.5	1.5/3.5	1.6/4	2.4/5.9	3.3/8.3
24'	Total Load	62	68	118	140	232	349	496	94	103	177	210	349	523	744
	Live Load L/360	48	52	86	102	165	244	343	73	79	130	153	248	366	515
	Min. End/Int. Bearing (in.)	1.5/3.5	1.5/3.5	1.5/3.5	1.5/3.5	1.5/3.5	2/5	2.8/7.1	1.5/3.5	1.5/3.5	1.5/3.5	1.5/3.5	1.5/3.5	2/5	2.8/7.1
26'	Total Load		51	90	107	180	272	389	71	77	135	161	271	409	584
	Live Load L/360		41	68	80	130	193	272	57	62	102	120	196	290	409
	Min. End/Int. Bearing (in.)		1.5/3.5	1.5/3.5	1.5/3.5	1.5/3.5	1.7/4.3	2.4/6.1	1.5/3.5	1.5/3.5	1.5/3.5	1.5/3.5	1.5/3.5	1.7/4.3	2.4/6.1
28'	Total Load			70	84	142	216	310	54	59	105	126	213	324	465
	Live Load L/360			55	64	105	155	219	46	50	82	97	157	233	329
	Min. End/Int. Bearing (in.)			1.5/3.5	1.5/3.5	1.5/3.5	1.5/3.7	2.1/5.3	1.5/3.5	1.5/3.5	1.5/3.5	1.5/3.5	1.5/3.5	1.5/3.7	2.1/5.3
30'	Total Load			55	66	113	173	249			82	99	170	260	374
	Live Load L/360			44	52	85	127	179			67	79	128	190	269
	Min. End/Int. Bearing (in.)			1.5/3.5	1.5/3.5	1.5/3.5	1.5/3.5	1.8/4.6			1.5/3.5	1.5/3.5	1.5/3.5	1.5/3.5	1.8/4.6
32'	Total Load				52	91	140	203			64	78	136	210	305
	Live Load L/360				43	70	105	148			55	65	106	157	223
	Min. End/Int. Bearing (in.)				1.5/3.5	1.5/3.5	1.5/3.5	1.6/4.1			1.5/3.5	1.5/3.5	1.5/3.5	1.5/3.5	1.6/4.1

* Indicates **Total Load** value controls.

Allowable Axial Loads (lbs) for 1.3E TimberStrand® LSL

Column Bearing Type	Effective Column Length	Column Size											
		3½" x 3½"			3½" x 4⅜"			3½" x 5½"			3½" x 7¼"		
		100%	115%	125%	100%	115%	125%	100%	115%	125%	100%	115%	125%
On Column Base	3'	12,165	13,665	14,625	15,210	17,085	18,280	19,120	21,475	22,980	25,205	28,310	30,290
	4'	10,745	11,830	12,490	13,435	14,790	15,610	16,885	18,590	19,625	22,260	24,505	25,870
	5'	9,120	9,810	10,215	11,400	12,265	12,765	14,335	15,420	16,050	18,895	20,325	21,155
	6'	7,550	7,985	8,235	9,440	9,980	10,295	11,865	12,550	12,945	15,640	16,540	17,060
	7'	6,235	6,525	6,695	7,795	8,160	8,370	9,800	10,255	10,520	12,915	13,520	13,870
	8'	5,195	5,400	5,515	6,490	6,750	6,895	8,160	8,485	8,670	10,755	11,185	11,430
	9'	4,375	4,525	4,610	5,465	5,655	5,765	6,870	7,110	7,245	9,060	9,370	9,550
	10'	3,725	3,840	3,905	4,655	4,795	4,880	5,850	6,030	6,135	7,715	7,950	8,085
	12'	2,785	2,855	2,895	3,480	3,565	3,615	4,375	4,485	4,545	5,770	5,910	5,995
	14'	2,155	2,200	2,225	2,695	2,750	2,780	3,385	3,455	3,495	4,465	4,555	4,610
On Wood Plate(1)(2)	3'–7'	5,765	5,765	5,765	7,065	7,065	7,065	8,740	8,740	8,740	10,785	10,785	10,785
	8'	5,195	5,400	5,515	6,490	6,750	6,895	8,160	8,485	8,670	10,755	10,785	10,785
	9'	4,375	4,525	4,610	5,465	5,655	5,765	6,870	7,110	7,245	9,060	9,370	9,550
	10'	3,725	3,840	3,905	4,655	4,795	4,880	5,850	6,030	6,135	7,715	7,950	8,085
	12'	2,785	2,855	2,895	3,480	3,565	3,615	4,375	4,485	4,545	5,770	5,910	5,995
	14'	2,155	2,200	2,225	2,695	2,750	2,780	3,385	3,455	3,495	4,465	4,555	4,610

(1) Wood plate bearing is based on compression perpendicular-to-grain stress of 425 psi adjusted per the NDS®, 3.10.4.

(2) See connection details below.

Allowable Axial Loads (lbs) for 1.8E Parallam® PSL

Column Bearing Type	Effective Column Length	Column Size																	
		3½" x 3½"			3½" x 5¼"			3½" x 7"			5¼" x 5¼"			5¼" x 7"			7" x 7"		
		100%	115%	125%	100%	115%	125%	100%	115%	125%	100%	115%	125%	100%	115%	125%	100%	115%	125%
On Column Base	6'	10,595	11,200	11,545	15,890	16,800	17,320	21,190	22,395	23,095	33,295	36,675	38,735	40,000	40,000	40,000	40,000	40,000	40,000
	7'	8,735	9,140	9,370	13,105	13,710	14,060	17,475	18,280	18,745	30,010	32,545	34,030	40,000	40,000	40,000	40,000	40,000	40,000
	8'	7,265	7,550	7,715	10,900	11,325	11,570	14,535	15,100	15,425	26,650	28,490	29,555	35,530	37,985	39,410	40,000	40,000	40,000
	9'	6,115	6,320	6,440	9,170	9,480	9,660	12,225	12,640	12,880	23,475	24,835	25,620	31,300	33,115	34,165	40,000	40,000	40,000
	10'	5,200	5,355	5,445	7,800	8,035	8,170	10,400	10,715	10,895	20,660	21,695	22,290	27,545	28,925	29,725	40,000	40,000	40,000
	12'	3,885	3,980	4,030	5,825	5,965	6,050	7,765	7,955	8,065	16,160	16,805	17,175	21,545	22,405	22,900	40,000	40,000	40,000
	14'	3,000	3,065	3,100	4,500	4,595	4,645	6,005	6,125	6,195	12,890	13,315	13,560	17,185	17,755	18,080	34,155	35,785	36,720
	16'	Slenderness ratio exceeds 50									10,480	10,775	10,950	13,970	14,370	14,595	28,485	29,640	30,300
	18'										8,670	8,885	9,010	11,560	11,850	12,010	24,020	24,860	25,345
	20'										7,285	7,445	7,535	9,710	9,925	10,050	20,475	21,110	21,475
	22'																17,630	18,125	18,405
	24'																15,325	15,715	15,935

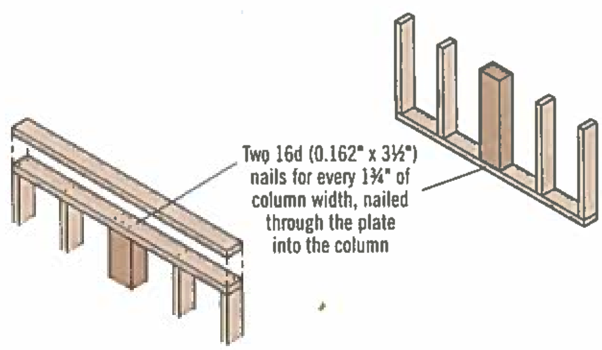
General Notes

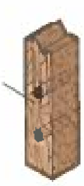
- Tables are based on:
  - Solid, one-piece column members used in dry-service conditions.
  - Bracing in both directions at column ends.
  - NDS®.
  - Simple columns with axial loads only. For side loads or other combined bending and axial loads, see the NDS®.
- Allowable loads have been adjusted to accommodate the worst case of the following eccentric conditions: ⅙ of column thickness (first dimension) or ⅙ of column width.
- Beams and columns must remain straight to within $^{L^2}/_{4608}$ (in.) of true alignment. L is the unrestrained length of the member in feet.

For column allowable design stresses see page 5.

The column and connector values listed are for dry-service conditions ONLY. When wet-service conditions exist, contact your Weyerhaeuser representative for other product solutions.

Top or Bottom Plate Connection





In order to use the manufacturer's published capacities when designing column caps, bases, or holdowns for uplift, the bolts or screws must be installed perpendicular to the wide face of strands as shown at left.

DO NOT install bolts or screws into the narrow face of strands

SIMPSON Strong-Tie®

Face-Mount Hangers – Solid Sawn Lumber (DF/SP)

The Joist Hanger Selector software enables you the most optimum product for your project. The software takes into consideration all the characteristics seen in this catalog. Visit **strongtie.com/jhs**.

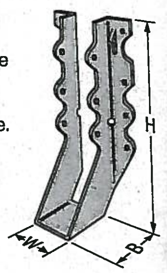
These products are available with additional corrosion protection. For more information, see p. 18.

These products are approved for installation with the Strong-Drive® SD Connector screw. See pp. 39–40 for more information.

Joist Size	Model No.	Ga.	Dimensions (in.) W	H	B	Min./Max.	Fasteners Header	Joist	DF/SP Allowable Loads Uplift (160)	Floor (100)	Snow (115)	Roof (125)	Installed Cost Index (ICI)	Code Ref.
Sawn Lumber Sizes														
2X4	LU24	20	1 9/16	3⅛	1½	—	(4) 16d	(2) 10d x 1½"	265	555	635	685	Lowest	I7, I27, FL, L5, L17
	LUS24	18	1 9/16	3⅛	1¾	—	(4) 10d	(2) 10d	490	670	765	825	+3%	
	U24	16	1 9/16	3⅛	1½	—	(4) 16d	(2) 10d x 1½"	265	575	655	705	+67%	I7, FL, L17
	HU26	14	1 9/16	3 1/16	2¼	—	(4) 16d	(2) 10d x 1½"	335	595	670	720	+295%	
DBL 2X4	LUS24-2	18	3⅛	3⅛	2	—	(4) 16d	(2) 16d	440	800	910	985	Lowest	I7, I27, FL, L5, L17
	U24-2	16	3⅛	3	2	—	(4) 16d	(2) 10d	370	575	655	705	+33%	I7, FL, L17
	HU24-2 / HUC24-2	14	3⅛	3 1/16	2½	—	(4) 16d	(2) 10d	380	380	595	720	+240%	
2x6	LUS26	18	1 9/16	4¾	1¾	—	(4) 10d	(4) 10d	1,165	865	990	1,070	Lowest	I7, I27, FL, L5, L17
	LU26	20	1 9/16	4¾	1½	—	(6) 16d	(4) 10d x 1½"	565	835	950	1,030	+6%	
	U26	16	1 9/16	4¾	2	—	(6) 16d	(4) 10d x 1½"	585	865	980	1,055	+43%	I7, FL, L17
	LUC26Z	18	1 9/16	4¾	1¾	—	(6) 16d	(4) 10d x 1½"	730	845	965	1,040	+160%	
	HU26	14	1 9/16	3 1/16	2¼	—	(4) 16d	(2) 10d x 1½"	335	595	670	720	+179%	
	HUS26	16	1⅝	5⅜	3	—	(14) 16d	(6) 16d	1,550	2,720	3,095	3,335	+276%	I7, I27, FL, L5, L17
DBL 2X6	LUS26-2	18	3⅛	4⅞	2	—	(4) 16d	(4) 16d	1,165	1,030	1,180	1,275	Lowest	
	U26-2	16	3⅛	5	2	—	(8) 16d	(4) 10d	740	1,150	1,305	1,410	+65%	I7, FL, L17
	HUS26-2 / HUSC26-2	14	3⅛	5 3/16	2	—	(4) 16d	(4) 16d	1,235	1,065	1,210	1,305	+172%	
	HU26-2 / HUC26-2	14	3⅛	5⅜	2½	Min.	(8) 16d	(4) 10d	760	1,190	1,345	1,445	+233%	I7, I27, FL, L5, L17
		14	3⅛	5⅜	2½	Max.	(12) 16d	(6) 10d	1,135	1,785	2,015	2,165	+254%	I7, FL, L17
TPL 2x6	LUS26-3	18	4⅝	4⅛	2		(4) 16d	(4) 16d	1,165	1,030	1,180	1,280	*	I7, FL
	U26-3	16	4⅝	4¼	2		(8) 16d	(4) 10d	740	1,150	1,305	1,410	*	
	HU26-3 / HUC26-3	14	4 11/16	4⅝	2½	Min.	(8) 16d	(4) 10d	760	1,190	1,345	1,445	*	
		14	4 11/16	4⅝	2½	Max.	(12) 16d	(6) 10d	1,135	1,785	2,015	2,165	*	
2x8	LUS26	18	1 9/16	4¾	1¾	—	(4) 10d	(4) 10d	1,165	865	990	1,070	Lowest	I7, I27, FL, L5, L17
	LU26	20	1 9/16	4¾	1½	—	(6) 16d	(4) 10d x 1½"	565	835	950	1,030	+6%	
	LUS28	18	1 9/16	6⅝	1¾	—	(6) 10d	(4) 10d	1,165	1,105	1,260	1,365	+23%	
	LU28	20	1 9/16	6⅝	1½	—	(8) 16d	(6) 10d x 1½"	850	1,110	1,270	1,335	+39%	
	U26	16	1 9/16	4¾	2	—	(6) 16d	(4) 10d x 1½"	585	865	980	1,055	+43%	I7, FL, L17
	LUC26Z	18	1 9/16	4¾	1¾	—	(6) 16d	(4) 10d x 1½"	730	845	965	1,040	+160%	
	HU28	14	1 9/16	5¼	2¼	—	(6) 16d	(4) 10d x 1½"	610	895	1,005	1,085	+251%	
	HUS26	16	1⅝	5⅜	3	—	(14) 16d	(6) 16d	1,550	2,720	3,095	3,335	+276%	I7, I27, FL, L5, L17
	HUS28	16	1⅝	7	3	—	(22) 16d	(8) 16d	2,000	3,965	4,120	4,220	+409%	
DBL 2x8	LUS26-2	18	3⅛	4⅞	2	—	(4) 16d	(4) 16d	1,165	1,030	1,180	1,280	Lowest	
	LUS28-2	18	3⅛	7	2	—	(6) 16d	(4) 16d	1,165	1,315	1,500	1,625	+8%	
	U26-2	16	3⅛	5	2	—	(8) 16d	(4) 10d	740	1,150	1,305	1,410	+65%	I7, FL, L17
	HUS28-2	14	3⅛	7 3/16	2	—	(6) 16d	(6) 16d	1,550	1,595	1,815	1,960	+188%	
	HU28-2 / HUC28-2	14	3⅛	7	2½	Min.	(10) 16d	(4) 10d	760	1,490	1,680	1,805	+397%	
		14	3⅛	7	2½	Max.	(14) 16d	(6) 10d	1,135	2,085	2,350	2,530	+418%	

1. Uplift loads apply to 10d and 16d header fasteners. Uplift loads have been increased for wind or earthquake loading with no further increase allowed. Reduce where other loads govern.
2. 10d commons or 16d sinkers may be used instead of the specified 16d at 0.84 of the table load value.
3. 16d sinkers may be used instead of the specified 10d commons with no load reduction. (16d sinkers are not acceptable for HDG applications.)
4. Min. nailing quantity and load values — fill all round holes; Max. nailing quantity and load values — fill all round and triangle holes.
5. DF/SP loads can be used for SCL that has fastener holding capacity of Doug Fir.
6. Truss chord cross-grain tension may limit allowable loads in accordance with ANSI/TPI 1-2014. Simpson Strong-Tie® Connector Selector™ software includes the evaluation of cross-grain tension in its hanger software includes the evaluation of cross-grain tension in its hanger allowable loads. For additional information, contact Simpson Strong-Tie.
7. **Nails:** 16d = 0.162" dia. x 3½" long, 10d = 0.148" dia. x 3" long, 10d x 1½" = 0.148" dia. x 1½" long. See pp. 26–27 for other nail sizes and information.

Hangers do not have an Installed Cost Index.



Codes: See p. 14 for Code Reference Key Chart

SIMPSON Strong-Tie®

Face-Mount Hangers – Solid Sawn Lumber (DF/SP)

These products are available with additional corrosion protection. For more information, see p. 18.

These products are approved for installation with the Strong-Drive® SD Connector screw. See pp. 39–40 for more information.

Joist Size	Model No.	Ga.	Dimensions (in.) W	H	B	Min./Max.	Fasteners Header	Joist	DF/SP Allowable Loads Uplift (160)	Floor (100)	Snow (115)	Roof (125)	Installed Cost Index (ICI)	Code Ref.
Sawn Lumber Sizes														
TPL 2X8	LUS28-3	18	4⅝	6¼	2	—	(6) 16d	(4) 16d	1,165	1,315	1,500	1,625	*	I7, FL
	U26-3	16	4⅝	4¼	2	—	(8) 16d	(4) 10d	740	1,150	1,305	1,410	*	
	HU26-3 / HUC26-3	14	4 11/16	4⅝	2½	Min.	(8) 16d	(4) 10d	760	1,190	1,345	1,445	*	
		14	4 11/16	4⅝	2½	Max.	(12) 16d	(6) 10d	1,135	1,785	2,015	2,165	*	
QUAD 2X8	HU28-4 / HUC28-4	14	6⅛	7	2½	Min.	(10) 16d	(4) 16d	900	1,490	1,680	1,805	*	160
		14	6⅛	7	2½	Max.	(14) 16d	(6) 16d	1,345	2,085	2,350	2,530	*	
2x10	LUS28	18	1 9/16	6⅝	1¾	—	(6) 10d	(4) 10d	1,165	1,100	1,255	1,360	Lowest	I7, I27, FL, L5, L17
	LU28	20	1 9/16	6⅜	1½	—	(8) 16d	(6) 10d x 1½"	850	1,110	1,270	1,335	+13%	
	LUS210	18	1 9/16	7 13/16	1¾	—	(8) 10d	(4) 10d	1,165	1,340	1,525	1,650	+15%	
	LU210	20	1 9/16	7 13/16	1½	—	(10) 16d	(6) 10d x 1½"	850	1,390	1,585	1,715	+28%	I7, FL, L17
	U210	16	1 9/16	7 13/16	2	—	(10) 16d	(6) 10d x 1½"	1,110	1,440	1,635	1,685	+76%	
	LUC210Z	18	1 9/16	7¾	1¾	—	(10) 16d	(6) 10d x 1½"	1,100	1,410	1,605	1,735	+180%	
	HU210	14	1 9/16	7⅛	2¼	—	(8) 16d	(4) 10d x 1½"	610	1,190	1,345	1,445	+225%	
	HUS210	16	1⅝	9	3	—	(30) 16d	(10) 16d	3,000	4,255	4,445	4,575	+450%	
DBL 2X10	LUS28-2	18	3⅛	7	2	—	(6) 16d	(4) 16d	1,165	1,315	1,500	1,625	Lowest	I7, I27, FL, L5, L17
	LUS210-2	18	3⅛	9	2	—	(8) 16d	(6) 16d	1,745	1,830	2,090	2,265	+34%	
	U210-2	16	3⅛	8½	2	—	(14) 16d	(6) 10d	1,110	2,015	2,285	2,465	+88%	I7, FL, L17
	HUS210-2	14	3⅛	9 3/16	2	—	(8) 16d	(8) 16d	3,295	2,125	2,420	2,615	+217%	I7, FL, L17
	HU210-2 / HUC210-2	14	3⅛	8 13/16	2½	Min.	(14) 16d	(6) 10d	1,135	2,085	2,350	2,530	+441%	I7, FL, L17
		14	3⅛	8 13/16	2½	Max.	(18) 16d	(10) 10d	1,895	2,680	3,020	3,250	+467%	
	HUCQ210-2-SDS	14	3¼	9	3	—	(12) ¼"x2½" SDS	(6) ¼"x2½" SDS	2,510	4,680	4,955	4,955	*	FL
	HHUS210-2	14	3 5/16	9 5/32	3	—	(30) 16d	(10) 16d	3,735	5,640	6,385	6,890	*	I7, FL, L17
TPL 2X10	LUS28-3	18	4⅝	6¼	2	—	(6) 16d	(4) 16d	1,165	1,315	1,500	1,625	*	I7, FL
	LUS210-3	18	4⅝	8 3/16	2	—	(8) 16d	(6) 16d	1,745	1,830	2,090	2,265	*	
	U210-3	16	4⅝	7¾	2	—	(14) 16d	(6) 10d	1,110	2,015	2,285	2,465	*	
	HU210-3 / HUC210-3	14	4 11/16	8 1/16	2½	Min.	(14) 16d	(6) 10d	1,135	2,085	2,350	2,530	*	I7, FL, L17
		14	4 11/16	8 1/16	2½	Max.	(18) 16d	(10) 10d	1,895	2,680	3,020	3,250	*	
	HHUS210-3	14	4 11/16	8⅞	3	—	(30) 16d	(10) 16d	3,735	5,640	6,385	6,890	*	FL
	HGUS210-3	12	4 15/16	9⅛	4	—	(46) 16d	(16) 16d	4,095	9,100	9,100	9,100	*	I7, FL
	HUCQ210-3-SDS	14	4⅝	9	3	—	(12) ¼"x2½" SDS	(6) ¼"x2½" SDS	2,510	4,680	4,955	4,955	*	FL
QUAD 2x10	HU210-4 / HUC210-4	14	6⅛	8⅜	2½	Min.	(14) 16d	(6) 16d	1,345	2,085	2,350	2,530	*	I7, FL
		14	6⅛	8⅜	2½	Max.	(18) 16d	(8) 16d	1,795	2,680	3,020	3,250	*	
	HHUS210-4	14	6⅛	8⅞	3	—	(30) 16d	(10) 16d	3,735	5,635	6,380	6,880	*	FL
	HGUS210-4	12	6 9/16	9⅛	4	—	(46) 16d	(16) 16d	4,095	9,100	9,100	9,100	*	I7, FL
2x12	LUS210	18	1 9/16	7 13/16	1¾	—	(8) 10d	(4) 10d	1,165	1,340	1,525	1,650	Lowest	I7, I27, FL, L5, L17
	LU210	20	1 9/16	7 13/16	1½	—	(10) 16d	(6) 10d x 1½"	850	1,390	1,585	1,715	+11%	
	U210	16	1 9/16	7 13/16	2	—	(10) 16d	(6) 10d x 1½"	1,110	1,440	1,635	1,685	+53%	I7, FL, L17
	LUC210Z	18	1 9/16	7¾	1¾	—	(10) 16d	(6) 10d x 1½"	1,100	1,410	1,605	1,735	+180%	I7, I27, FL, L5, L17
	HU212	14	1 9/16	9	2¼	—	(10) 16d	(6) 10d x 1½"	1,135	1,490	1,680	1,805	+347%	I7, FL, L17
	HUS210	16	1⅝	9	3	—	(30) 16d	(10) 16d	3,000	4,255	4,445	4,575	+378%	
DBL 2x12	LUS210-2	18	3⅛	9	2	—	(8) 16d	(6) 16d	1,745	1,830	2,090	2,265	Lowest	I7, I27, FL, L5, L17
	U210-2	16	3⅛	8½	2	—	(14) 16d	(6) 10d	1,110	2,015	2,285	2,465	+40%	I7, FL, L17
	LUS214-2	18	3⅛	10 15/16	2	—	(10) 16d	(6) 16d	1,745	2,110	2,410	2,610	+56%	
	HUS210-2	14	3⅛	9 3/16	2	—	(8) 16d	(8) 16d	3,295	2,125	2,420	2,615	*	
	HUS212-2	14	3⅛	10¾	2	—	(10) 16d	(10) 16d	3,635	2,660	3,025	3,265	*	
	HU212-2 / HUC212-2	14	3⅛	10 9/16	2½	Min.	(16) 16d	(6) 10d	1,135	2,380	2,685	2,890	*	
		14	3⅛	10 9/16	2½	Max.	(22) 16d	(10) 10d	1,895	3,275	3,695	3,970	+411%	
	HUCQ210-2-SDS	14	3¼	9	3	—	(12) ¼"x2½" SDS	(6) ¼"x2½" SDS	2,510	4,680	4,955	4,955	*	FL

See footnotes on p. 130.

— FLOOR

— ROOF

Codes: See p. 14 for Code Reference Key Chart

PEC
PROFESSIONAL ENGINEERING CONSULTANTS PA
303 SOUTH TOPEKA | WICHITA, KANSAS 67202
316-262-2691 | FAX 316-262-2014 | www.pec1.com

Project 180248-001 : Dole VA   Date ________

Item Calc capacity of 10 x 22 @ Dole VA   By ________

Load from Attic : Loads : 20 psf Dead, 40 psf Live load × 10.625 trib

= 213.6 PlF (DL) + 425 PLF (LL)

Load from 2nd Floor   Loads : 20 psf Dead Load → 214 PLF
50 psf live Load → 530 PLF

Load from Main Floor   Loads : 20 psf DL → 214 PLF
100 psf LL → 1062 PLF

Total Loads : DL : 214 + 214 = 428 plf
LL : 530 + 1062 = 1592 plf
+ Pt Load of attic

Loads inputted in Enercalc : Ext. Floor Load : .428 K/ft (DL)
1.6 K/ft (LL)

PSL Load : 2.14^k DL (.213 x 10')
4.25^k LL (.425 x 10')

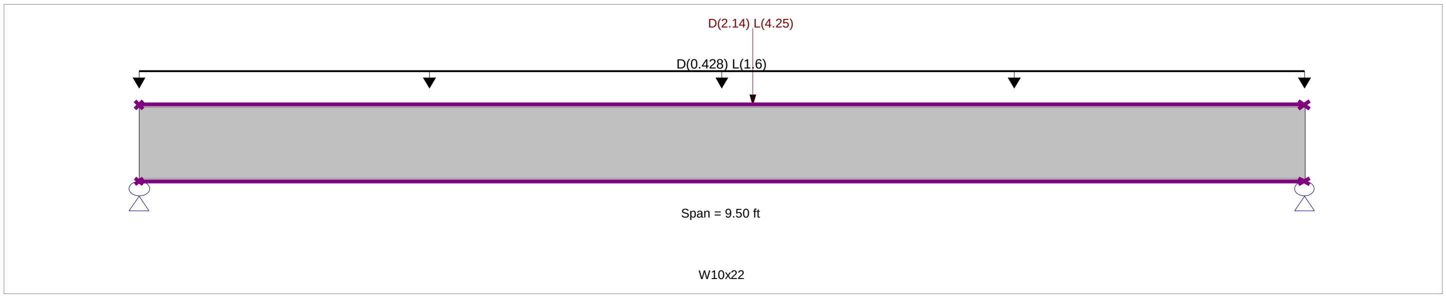
Description : EXISTING STEEL BEAM. Single Span 10x22

CODE REFERENCES

Calculations per AISC 360-10, IBC 2015, ASCE 7-10
Load Combination Set : IBC 2015

Material Properties

Analysis Method :	Allowable Strength Design	Fy : Steel Yield :	50.0 ksi
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling	E: Modulus :	29,000.0 ksi
Bending Axis :	Major Axis Bending		



Applied Loads

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

Beam self weight NOT internally calculated and added
Load(s) for Span Number 1
Point Load : D = 2.140, L = 4.250 k @ 5.0 ft, (New Load from PSL Column)
Uniform Load : D = 0.4280, L = 1.60 k/ft, Tributary Width = 1.0 ft, (Existing Floor Loads on from 2nd)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	**0.585** : 1	Maximum Shear Stress Ratio =	**0.265** : 1
Section used for this span	**W10x22**	Section used for this span	**W10x22**
Ma : Applied	37.935 k-ft	Va : Applied	12.996 k
Mn / Omega : Allowable	64.870 k-ft	Vn/Omega : Allowable	48.960 k
Load Combination	+D+L+H	Load Combination	+D+L+H
Location of maximum on span	4.994 ft	Location of maximum on span	9.500 ft
Span # where maximum occurs	Span # 1	Span # where maximum occurs	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.124 in	Ratio =	916 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.167 in	Ratio =	684 >=180
Max Upward Total Deflection	0.000 in	Ratio =	0 <180

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
+D+H													
Dsgn. L = 9.50 ft	1	0.152	0.065	9.88		9.88	108.33	64.87	1.00	1.00	3.16	73.44	48.96
+D+L+H													
Dsgn. L = 9.50 ft	1	0.585	0.265	37.93		37.93	108.33	64.87	1.00	1.00	13.00	73.44	48.96
+D+Lr+H													
Dsgn. L = 9.50 ft	1	0.152	0.065	9.88		9.88	108.33	64.87	1.00	1.00	3.16	73.44	48.96
+D+S+H													
Dsgn. L = 9.50 ft	1	0.152	0.065	9.88		9.88	108.33	64.87	1.00	1.00	3.16	73.44	48.96
+D+0.750Lr+0.750L+H													
Dsgn. L = 9.50 ft	1	0.477	0.215	30.92		30.92	108.33	64.87	1.00	1.00	10.54	73.44	48.96
+D+0.750L+0.750S+H													
Dsgn. L = 9.50 ft	1	0.477	0.215	30.92		30.92	108.33	64.87	1.00	1.00	10.54	73.44	48.96
+D+0.60W+H													
Dsgn. L = 9.50 ft	1	0.152	0.065	9.88		9.88	108.33	64.87	1.00	1.00	3.16	73.44	48.96
+D+0.70E+H													
Dsgn. L = 9.50 ft	1	0.152	0.065	9.88		9.88	108.33	64.87	1.00	1.00	3.16	73.44	48.96
+D+0.750Lr+0.750L+0.450W+H													
Dsgn. L = 9.50 ft	1	0.477	0.215	30.92		30.92	108.33	64.87	1.00	1.00	10.54	73.44	48.96
+D+0.750L+0.750S+0.450W+H													
Dsgn. L = 9.50 ft	1	0.477	0.215	30.92		30.92	108.33	64.87	1.00	1.00	10.54	73.44	48.96
+D+0.750L+0.750S+0.5250E+H													
Dsgn. L = 9.50 ft	1	0.477	0.215	30.92		30.92	108.33	64.87	1.00	1.00	10.54	73.44	48.96
+0.60D+0.60W+0.60H													

Description : EXISTING STEEL BEAM. Single Span 10x22

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
Dsgn. L = 9.50 ft	1	0.091	0.039	5.93		5.93	108.33	64.87	1.00	1.00	1.90	73.44	48.96
+0.60D+0.70E+0.60H													
Dsgn. L = 9.50 ft	1	0.091	0.039	5.93		5.93	108.33	64.87	1.00	1.00	1.90	73.44	48.96

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+L+H	1	0.1668	4.804		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	12.660	12.996
Overall MINimum	1.828	1.896
+D+H	3.047	3.159
+D+L+H	12.660	12.996
+D+Lr+H	3.047	3.159
+D+S+H	3.047	3.159
+D+0.750Lr+0.750L+H	10.257	10.537
+D+0.750L+0.750S+H	10.257	10.537
+D+0.60W+H	3.047	3.159
+D+0.70E+H	3.047	3.159
+D+0.750Lr+0.750L+0.450W+H	10.257	10.537
+D+0.750L+0.750S+0.450W+H	10.257	10.537
+D+0.750L+0.750S+0.5250E+H	10.257	10.537
+0.60D+0.60W+0.60H	1.828	1.896
+0.60D+0.70E+0.60H	1.828	1.896
D Only	3.047	3.159
Lr Only		
L Only	9.613	9.837
S Only		
W Only		
E Only		
H Only		

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Project 186248-001 : Dole VA   Date ______

Item Canopy Analysis   By ______

Loading : Dead Load = 3/4 Plywood ⇒ 3 psf /in thickness

↳ 2 layers = $(3/4)(3\text{ psf})(2) = 4.5\ \#/SF$

Rigid Insul ⇒ 1/2" = .75 #/SF

light gauge ⟹ 4 psf

Misc ⟹ 1.75 psf

Total DL = 11 PSF

Wind Load on Fascia of Canopy

(WW) (LW)
36.6 + 24.4 PSF = 61 #/SF

Allowable Deflection

canopy → $\frac{2\ell}{360} = \frac{2(11')(12)}{360} = \frac{264''}{360} = .73''$

From Enercalc → Max Δ = .68"

.68 < .73" ✓ OK

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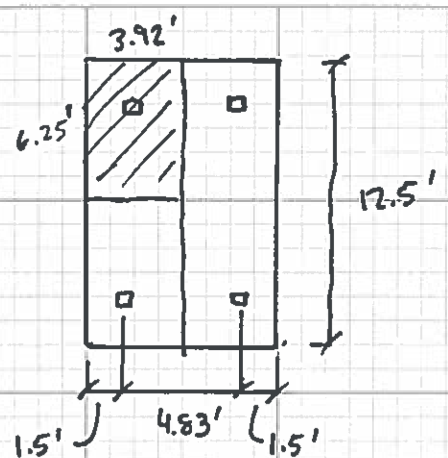
Project 180248-001 : Dole VA Date

Item Canopy Analysis By



$4.83/2 = 2.42 + 1.5 = 3.92$

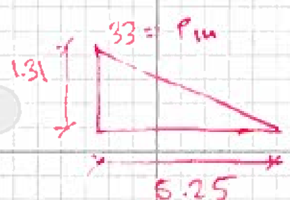
Area $= 3.92 \times 6.25 = \boxed{24.5 \text{ SF}}$

DL : 11 PSF × 24.5 = 270 #

RL : 20 PSF × 24.5 = 490 #

SL : 33 PSF × 24.5 = 808.5 #

Drift



WL (vertical) : (24.9 PSF)(24.5) = 610 #

Tube 6×4×½ → 28.43 #/ft

Support reaction :

	#1	#2
DL –	~~.27~~ .45 k	~~.27~~ .45 k
Lr –	.49	.49
S –	.4	.31
W –	.61	.61

Lateral

Area $= \left(\frac{16}{12}\right)(12.5)$

$= 16.67$ SF

Wind Load = 61 PSF

$16.67 \times 61 = 1017/2 = 508$ #

Base shear = .618 k

Moment = 3.56 k-ft

STRUCTURAL CALCULATIONS

FOR

Dole VA - Building 6 Canopy

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Code Search

Code: International Building Code 2012

Occupancy:

Occupancy Group = B Business

Risk Category & Importance Factors:

Risk Category =	II
Wind factor =	1.00
Snow factor =	1.00
Seismic factor =	1.00

Type of Construction:

Fire Rating:

Roof =	0.0 hr
Floor =	0.0 hr

Building Geometry:

Roof angle (θ)	0.00 / 12	0.0 deg
Building length (L)	13.0 ft	
Least width (B)	5.0 ft	
Mean Roof Ht (h)	12.0 ft	
Parapet ht above grd	0.0 ft	
Minimum parapet ht	0.0 ft	

Live Loads:

<u>**Roof**</u>

0 to 200 sf:	20 psf
200 to 600 sf:	24 - 0.02Area, but not less than 12 psf
over 600 sf:	12 psf

<u>**Floor:**</u>

Typical Floor	40 psf
Partitions	15 psf
Lobbies & first floor corridors	100 psf
Corridors above first floor	80 psf
Balconies (exterior) - same as occupa	40 psf

Seismic Loads: IBC 2012

Strength Level Forces

Risk Category : II
Importance Factor (I) : 1.00
Site Class : D

Ss (0.2 sec) = 10.90 %g
S1 (1.0 sec) = 5.40 %g

Fa =	1.600	Sms =	0.174	S_{DS} =	0.116	Design Category =	A
Fv =	2.400	Sm1 =	0.130	S_{D1} =	0.086	Design Category =	B

Seismic Design Category = B
Redundancy Coefficient ρ = 1.00
Number of Stories: 2
Structure Type: All other building systems
Horizontal Struct Irregularities: No plan Irregularity
Vertical Structural Irregularities: No vertical Irregularity
Flexible Diaphragms: Yes
Building System: **Structural steel systems not specifically detailed for seismic resistance**
Seismic resisting system: **Structural steel systems not specifically detailed for seismic resistance**
System Structural Height Limit: **Height not limited**
Actual Structural Height (hn) = 12.0 ft

DESIGN COEFFICIENTS AND FACTORS

Response Modification Coefficient (R) = 3
Over-Strength Factor (Ωo) = 2.5
Deflection Amplification Factor (Cd) = 3
S_{DS} = 0.116
S_{D1} = 0.086

Seismic Load Effect (E) = Eh +/-Ev = ρQ_E +/- $0.2S_{DS}D$ = Qe +/- 0.000D Q_E = horizontal seismic force
Special Seismic Load Effect (Em) = Emh +/- Ev = $\Omega o\, Q_E$ +/- $0.2S_{DS}D$ = 2.5Qe +/- 0.023D D = dead load

PERMITTED ANALYTICAL PROCEDURES

Simplified Analysis - Use Equivalent Lateral Force Analysis

Equivalent Lateral-Force Analysis - Permitted
Building period coef. (C_T) = 0.020 Cu = 1.70
Approx fundamental period (Ta) = $C_T h_n^{\wedge}$ = 0.129 sec x= 0.75 Tmax = CuTa = 0.219
User calculated fundamental period (T) = 0 sec Use T = 0.129
Long Period Transition Period (TL) = ASCE7 map = 12
Seismic response coef. (Cs) = $S_{DS}I/R$ = 0.039
need not exceed Cs = Sd1 I /RT = 0.223
but not less than Cs = 0.010
USE Cs = 0.039
Design Base Shear V = 0.039W

Model & Seismic Response Analysis - Permitted (see code for procedure)

ALLOWABLE STORY DRIFT

Structure Type: All other structures
Allowable story drift Δa = 0.020hsx where hsx is the story height below level x

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JOB TITLE Dole VA - Building 6 Canopy

JOB NO. 180248 SHEET NO.
CALCULATED BY YFC DATE
CHECKED BY DATE

Wind Loads - Open Buildings: 0.25 ≤ h/L ≤ 1.0

Ultimate Wind Pressures

Type of roof = Monoslope Free Roofs
Wind Flow = Clear

G = 0.85
Roof Angle = 0.0 deg

NOTE: The code requires the MWFRS be designed for a minimum pressure of 16 psf.

Main Wind Force Resisting System

Kz = Kh (case 2) = 0.85

Base pressure (qh) = **24.4 psf**

Roof pressures - Wind Normal to Ridge

Procedure not allowed h/L is greater than 1.0

Wind Flow	Load Case		Wind Direction γ = 0 & 180 deg	
			Cnw	Cnl
Clear Wind Flow	A	Cn =	1.20	0.30
		p =	24.9 psf	6.2 psf
	B	Cn =	-1.10	-0.10
		p =	-22.8 psf	-2.1 psf

NOTE: 1). Cnw and Cnl denote combined pressures from top and bottom roof surfaces.
2). Cnw is pressure on windward half of roof. Cnl is pressure on leeward half of roof.
3). Positive pressures act toward the roof. Negative pressures act away from the roof.

Roof pressures - Wind Parallel to Ridge, γ = 90 deg

Wind Flow	Load Case		Horizontal Distance from Windward Edge		
			≤ h	>h ≤ 2h	> 2h
Clear Wind Flow	A	Cn =	-0.80	-0.60	-0.30
		p =	-16.6 psf	-12.5 psf	-6.2 psf
	B	Cn =	0.80	0.50	0.30
		p =	16.6 psf	10.4 psf	6.2 psf

h = 12.0 ft
2h = 24.0 ft

Fascia Panels -Horizontal pressures

qp = 24.4 psf

Windward fascia: 36.6 psf (GCpn = +1.5)
Leeward fascia: -24.4 psf (GCpn = -1.0)

Components & Cladding - roof pressures

Kz = Kh (case 1) = 0.85
Base pressure (qh) = **24.4 psf**
G = 0.85

a = 3.0 ft

a^2 = 9.0 sf
$4a^2$ = 36.0 sf

	Effective Wind Area	Clear Wind Flow					
		zone 3		zone 2		zone 1	
		positive	negative	positive	negative	positive	negative
C_N	≤ 9 sf	2.40	-3.30	1.80	-1.70	1.20	-1.10
	>9, ≤ 36 sf	1.80	-1.70	1.80	-1.70	1.20	-1.10
	> 36 sf	1.20	-1.10	1.20	-1.10	1.20	-1.10
Wind pressure	≤ 9 sf	49.8 psf	-68.5 psf	37.4 psf	-35.3 psf	24.9 psf	-22.8 psf
	>9, ≤ 36 sf	37.4 psf	-35.3 psf	37.4 psf	-35.3 psf	24.9 psf	-22.8 psf
	> 36 sf	24.9 psf	-22.8 psf	24.9 psf	-22.8 psf	24.9 psf	-22.8 psf

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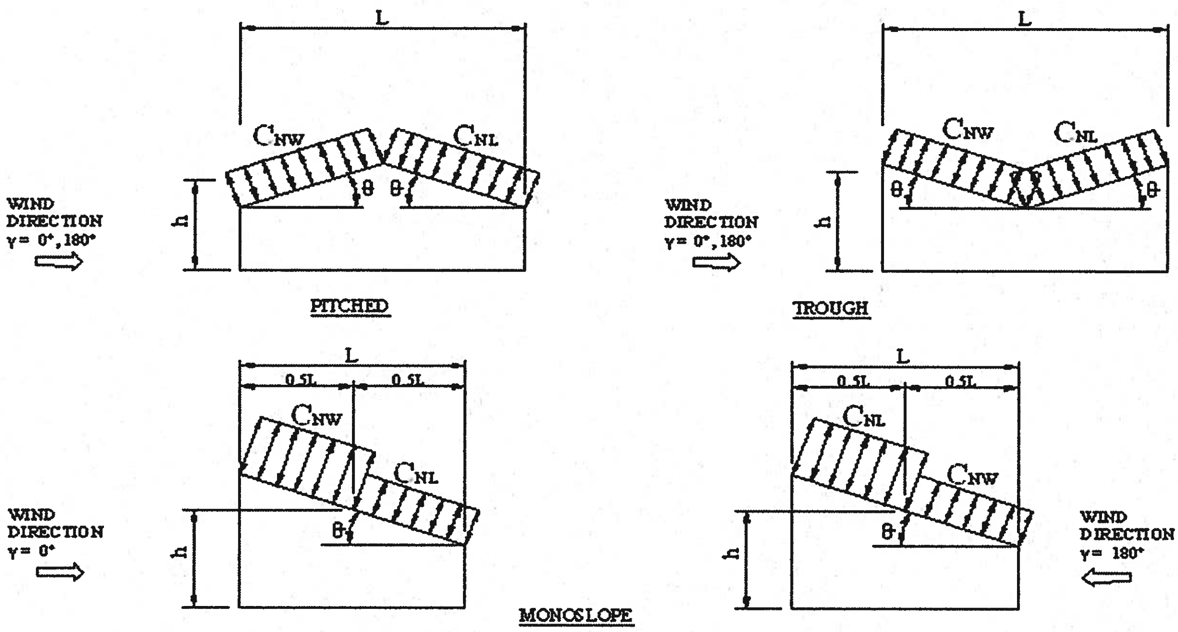
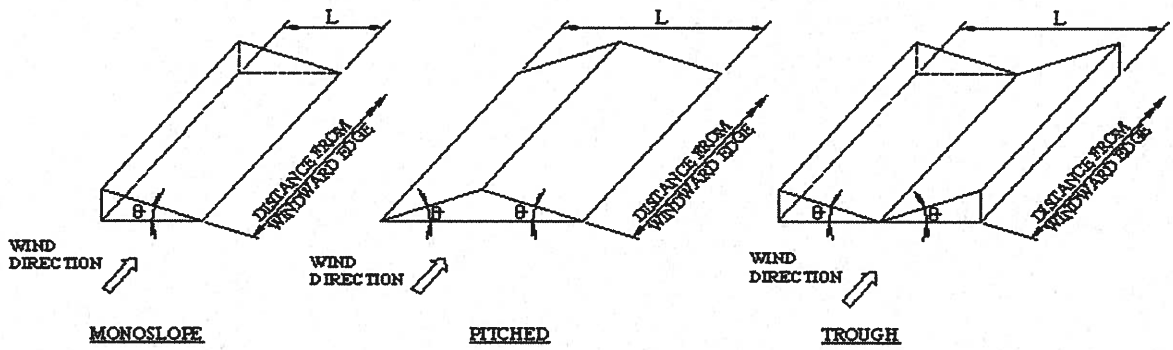
JOB TITLE	Dole VA - Building 6 Canopy		
JOB NO.	180248	SHEET NO.	
CALCULATED BY	YFC	DATE	
CHECKED BY		DATE	

Location of Wind Pressure Zones

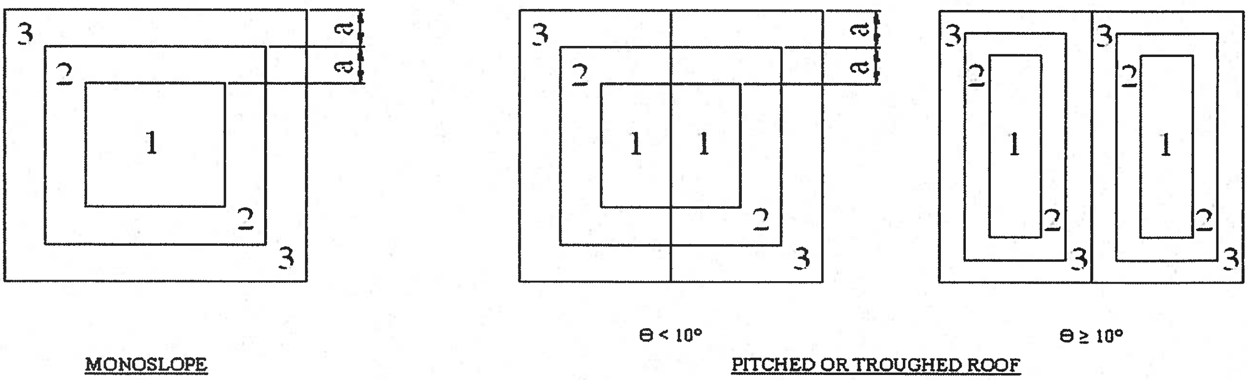


WIND DIRECTION γ= 0°, 180°



WIND DIRECTION γ= 90°

MAIN WIND FORCE RESISTING SYSTEM



COMPONENTS AND CLADDING

Steel Column

Description : Canopy Column (next to existing wall)

Code References

Calculations per AISC 360-10, IBC 2015, CBC 2016, ASCE 7-10
Load Combinations Used : IBC 2015

General Information

Steel Section Name : **HSS4x4x1/2**
Analysis Method : Load Resistance Factor
Steel Stress Grade
Fy : Steel Yield 36.0 ksi
E : Elastic Bending Modulus 29,000.0 ksi

Overall Column Height 11.0 ft
Top & Bottom Fixity Top Free, Bottom Fixed
Brace condition for deflection (buckling) along columns :
X-X (width) axis :
Unbraced Length for X-X Axis buckling = 10 ft, K = 2.1
Y-Y (depth) axis :
Unbraced Length for Y-Y Axis buckling = 10 ft, K = 2.1

Applied Loads

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

Column self weight included : 237.930 lbs * Dead Load Factor
AXIAL LOADS . . .
Axial Load at 11.0 ft, D = 0.450, LR = 0.490, S = 0.40, W = 0.610 k
BENDING LOADS . . .
Lat. Point Load at 10.50 ft creating My-y, W = 0.5080 k
wind load on face of col: Lat. Uniform Load creating My-y, W = 0.010 k/ft

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio = **0.3054** : 1
Load Combination +1.20D+0.50Lr+0.50L+W+1.60H
Location of max.above base 0.0 ft
At maximum location values are . . .
Pu 1.681 k
0.9 * Pn 42.577 k
Mu-x 0.0 k-ft
0.9 * Mn-x : 20.790 k-ft
Mu-y -5.939 k-ft
0.9 * Mn-y : 20.790 k-ft

PASS Maximum Shear Stress Ratio = **0.01575** : 1
Load Combination +1.20D+0.50Lr+0.50L+W+1.60H
Location of max.above base 0.0 ft
At maximum location values are . . .
Vu : Applied 0.6180 k
Vn * Phi : Allowable 39.247 k

Maximum SERVICE Load Reactions . .
Top along X-X 0.0 k
Bottom along X-X 0.6180 k
Top along Y-Y 0.0 k
Bottom along Y-Y 0.0 k

Maximum SERVICE Load Deflections . . .
Along Y-Y 0.0 in at 0.0 ft above base
for load combination :
Along X-X 0.6826 in at 11.0 ft above base
for load combination : +D+0.60W+H

Load Combination Results

Load Combination	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Maximum Shear Ratios: Stress Ratio	Status	Location
+1.40D+1.60H	0.023	PASS	0.00 ft	0.000	PASS	0.00 ft
+1.20D+0.50Lr+1.60L+1.60H	0.025	PASS	0.00 ft	0.000	PASS	0.00 ft
+1.20D+1.60L+0.50S+1.60H	0.024	PASS	0.00 ft	0.000	PASS	0.00 ft
+1.20D+1.60Lr+0.50L+1.60H	0.038	PASS	0.00 ft	0.000	PASS	0.00 ft
+1.20D+1.60Lr+0.50W+1.60H	0.165	PASS	0.00 ft	0.008	PASS	0.00 ft
+1.20D+0.50L+1.60S+1.60H	0.034	PASS	0.00 ft	0.000	PASS	0.00 ft
+1.20D+1.60S+0.50W+1.60H	0.164	PASS	0.00 ft	0.008	PASS	0.00 ft
+1.20D+0.50Lr+0.50L+W+1.60H	0.305	PASS	0.00 ft	0.016	PASS	0.00 ft
+1.20D+0.50L+0.50S+W+1.60H	0.305	PASS	0.00 ft	0.016	PASS	0.00 ft
+1.20D+0.50L+0.70S+E+1.60H	0.026	PASS	0.00 ft	0.000	PASS	0.00 ft
+0.90D+W+0.90H	0.300	PASS	0.00 ft	0.016	PASS	0.00 ft
+0.90D+E+0.90H	0.015	PASS	0.00 ft	0.000	PASS	0.00 ft

Description : Canopy Column (next to existing wall)

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	Axial Reaction @ Base	X-X Axis Reaction k @ Base	X-X Axis Reaction k @ Top	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Mx - End Moments k-ft @ Base	Mx - End Moments k-ft @ Top	My - End Moments @ Base	My - End Moments @ Top
+D+H	0.688								
+D+L+H	0.688								
+D+Lr+H	1.178								
+D+S+H	1.088								
+D+0.750Lr+0.750L+H	1.055								
+D+0.750L+0.750S+H	0.988								
+D+0.60W+H	1.054	-0.371						-3.563	
+D+0.70E+H	0.688								
+D+0.750Lr+0.750L+0.450W+H	1.330	-0.278						-2.673	
+D+0.750L+0.750S+0.450W+H	1.262	-0.278						-2.673	
+D+0.750L+0.750S+0.5250E+H	0.988								
+0.60D+0.60W+0.60H	0.779	-0.371						-3.563	
+0.60D+0.70E+0.60H	0.413								
D Only	0.688								
Lr Only	0.490								
L Only									
S Only	0.400								
W Only	0.610	-0.618						-5.939	
E Only									
H Only									

Extreme Reactions

Item	Extreme Value	Axial Reaction @ Base	X-X Axis Reaction k @ Base	X-X Axis Reaction k @ Top	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Mx - End Moments k-ft @ Base	Mx - End Moments k-ft @ Top	My - End Moments @ Base	My - End Moments @ Top
Axial @ Base	Maximum	1.330	-0.278						-2.673	
"	Minimum									
Reaction, X-X Axis	Maximum	0.688								
"	Minimum	0.610	-0.618						-5.939	
Reaction, Y-Y Axis	Maximum	0.688								
"	Minimum	0.688								
Reaction, X-X Axis	Maximum	0.688								
"	Minimum	0.688								
Reaction, Y-Y Axis	Maximum	0.688								
"	Minimum	0.610	-0.618						-5.939	
Moment, X-X Axis Ba	Maximum	0.688								
"	Minimum	0.688								
Moment, Y-Y Axis Ba	Maximum	0.688								
"	Minimum	0.610	-0.618						-5.939	
Moment, X-X Axis To	Maximum	0.688								
"	Minimum	0.688								
Moment, Y-Y Axis To	Maximum	0.688								
"	Minimum	0.688								

Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection		Distance		Max. Y-Y Deflection		Distance	
+D+H	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+L+H	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+Lr+H	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+S+H	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+0.750Lr+0.750L+H	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+0.750L+0.750S+H	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+0.60W+H	0.6826	in	11.000	ft	0.000	in	0.000	ft
+D+0.70E+H	0.0000	in	0.000	ft	0.000	in	0.000	ft
+D+0.750Lr+0.750L+0.450W+H	0.5120	in	11.000	ft	0.000	in	0.000	ft
+D+0.750L+0.750S+0.450W+H	0.5120	in	11.000	ft	0.000	in	0.000	ft
+D+0.750L+0.750S+0.5250E+H	0.0000	in	0.000	ft	0.000	in	0.000	ft
+0.60D+0.60W+0.60H	0.6826	in	11.000	ft	0.000	in	0.000	ft
+0.60D+0.70E+0.60H	0.0000	in	0.000	ft	0.000	in	0.000	ft
D Only	0.0000	in	0.000	ft	0.000	in	0.000	ft

Description : Canopy Column (next to existing wall)

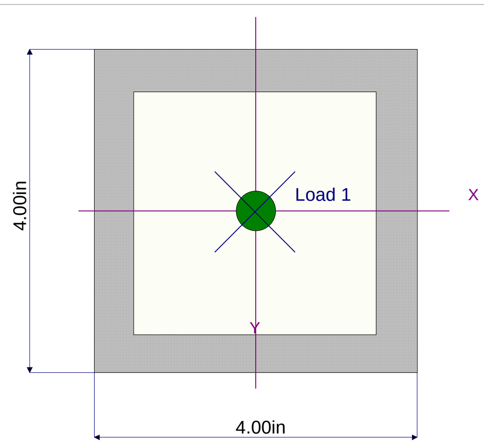
Maximum Deflections for Load Combinations

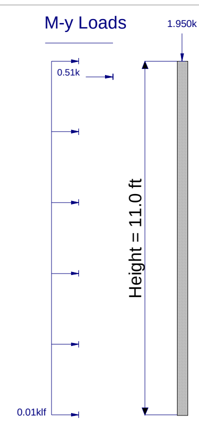
Load Combination	Max. X-X Deflection		Distance		Max. Y-Y Deflection		Distance	
Lr Only	0.0000	in	0.000	ft	0.000	in	0.000	ft
L Only	0.0000	in	0.000	ft	0.000	in	0.000	ft
S Only	0.0000	in	0.000	ft	0.000	in	0.000	ft
E Only	0.0000	in	0.000	ft	0.000	in	0.000	ft
H Only	0.0000	in	0.000	ft	0.000	in	0.000	ft

Steel Section Properties : HSS4x4x1/2

Depth	=	4.000	in	I xx	=	11.90	in^4	J	=	21.000	in^4
Design Thick	=	0.465	in	S xx	=	5.97	in^3				
Width	=	4.000	in	R xx	=	1.410	in				
Wall Thick	=	0.500	in	Zx	=	7.700	in^3				
Area	=	6.020	in^2	I yy	=	11.900	in^4	C	=	11.200	in^3
Weight	=	21.630	plf	S yy	=	5.970	in^3				
				R yy	=	1.410	in				

Ycg = 0.000 in





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

Steel Beam

File = u:\Wichita-Facil\2018\180248\001\Struct\Calcs\180248 dole va building 6.ec6
ENERCALC, INC. 1983-2017, Build:6.17.3.29, Ver:6.17.3.29

Lic. # : KW-06005112 **Licensee : PEC**

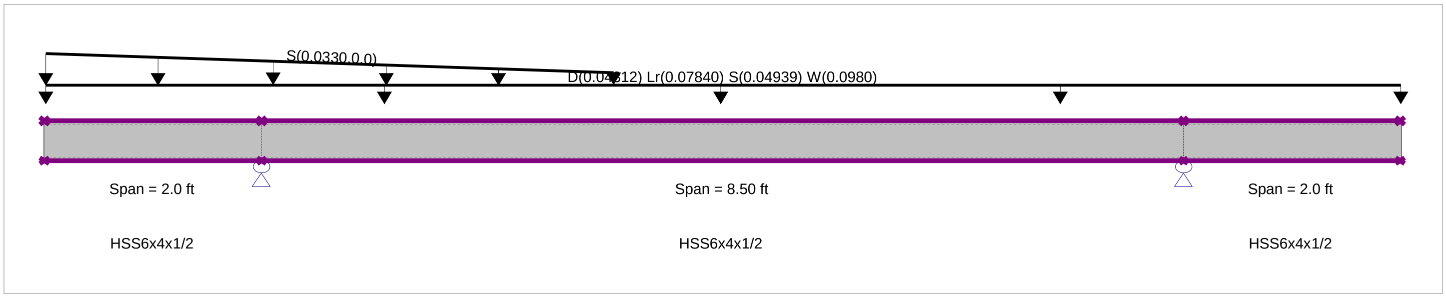
Description : Tube Supporting Deck

CODE REFERENCES

Calculations per AISC 360-10, IBC 2015, ASCE 7-10
Load Combination Set : IBC 2015

Material Properties

Analysis Method :	Allowable Strength Design	Fy : Steel Yield :	50.0 ksi
Beam Bracing :	Beam is Fully Braced against lateral-torsional buckling	E: Modulus :	29,000.0 ksi
Bending Axis :	Major Axis Bending		



Applied Loads

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

Beam self weight calculated and added to loading
Loads on all spans...
Uniform Load on ALL spans : D = 0.0110, Lr = 0.020, S = 0.01260, W = 0.0250 ksf, Tributary Width = 3.920 ft
Varying Uniform Load : S(S,E) = 0.0330->0.0 k/ft, Extent = 0.0 -->> 5.250 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =	0.034 : 1	Maximum Shear Stress Ratio =	0.010 : 1
Section used for this span	HSS6x4x1/2	Section used for this span	HSS6x4x1/2
Ma : Applied	1.227 k-ft	Va : Applied	0.7414 k
Mn / Omega : Allowable	36.427 k-ft	Vn/Omega : Allowable	76.934 k
Load Combination	+D+0.750Lr+0.750L+0.450W+H	Load Combination	+D+0.750Lr+0.750L+0.450W+H
Location of maximum on span	4.250 ft	Location of maximum on span	8.500 ft
Span # where maximum occurs	Span # 2	Span # where maximum occurs	Span # 2

Maximum Deflection

Max Downward Transient Deflection	0.009 in	Ratio =	11,785 >=360
Max Upward Transient Deflection	-0.006 in	Ratio =	8,683 >=360
Max Downward Total Deflection	0.015 in	Ratio =	6621 >=180
Max Upward Total Deflection	-0.010 in	Ratio =	4878 >=180

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
+D+H													
Dsgn. L = 2.00 ft	1	0.004	0.004		-0.14	0.14	60.83	36.43	1.00	1.00	0.30	128.48	76.93
Dsgn. L = 8.50 ft	2	0.014	0.004	0.50	-0.14	0.50	60.83	36.43	1.00	1.00	0.30	128.48	76.93
Dsgn. L = 2.00 ft	3	0.004	0.002		-0.14	0.14	60.83	36.43	1.00	1.00	0.14	128.48	76.93
+D+L+H													
Dsgn. L = 2.00 ft	1	0.004	0.004		-0.14	0.14	60.83	36.43	1.00	1.00	0.30	128.48	76.93
Dsgn. L = 8.50 ft	2	0.014	0.004	0.50	-0.14	0.50	60.83	36.43	1.00	1.00	0.30	128.48	76.93
Dsgn. L = 2.00 ft	3	0.004	0.002		-0.14	0.14	60.83	36.43	1.00	1.00	0.14	128.48	76.93
+D+Lr+H													
Dsgn. L = 2.00 ft	1	0.008	0.008		-0.30	0.30	60.83	36.43	1.00	1.00	0.64	128.48	76.93
Dsgn. L = 8.50 ft	2	0.029	0.008	1.05	-0.30	1.05	60.83	36.43	1.00	1.00	0.64	128.48	76.93
Dsgn. L = 2.00 ft	3	0.008	0.004		-0.30	0.30	60.83	36.43	1.00	1.00	0.30	128.48	76.93
+D+S+H													
Dsgn. L = 2.00 ft	1	0.008	0.007		-0.30	0.30	60.83	36.43	1.00	1.00	0.55	128.48	76.93
Dsgn. L = 8.50 ft	2	0.023	0.007	0.84	-0.30	0.84	60.83	36.43	1.00	1.00	0.55	128.48	76.93
Dsgn. L = 2.00 ft	3	0.007	0.003		-0.24	0.24	60.83	36.43	1.00	1.00	0.24	128.48	76.93
+D+0.750Lr+0.750L+H													
Dsgn. L = 2.00 ft	1	0.007	0.007		-0.26	0.26	60.83	36.43	1.00	1.00	0.55	128.48	76.93
Dsgn. L = 8.50 ft	2	0.025	0.007	0.92	-0.26	0.92	60.83	36.43	1.00	1.00	0.55	128.48	76.93
Dsgn. L = 2.00 ft	3	0.007	0.003		-0.26	0.26	60.83	36.43	1.00	1.00	0.26	128.48	76.93
+D+0.750L+0.750S+H													
Dsgn. L = 2.00 ft	1	0.007	0.006		-0.26	0.26	60.83	36.43	1.00	1.00	0.49	128.48	76.93
Dsgn. L = 8.50 ft	2	0.021	0.006	0.76	-0.26	0.76	60.83	36.43	1.00	1.00	0.49	128.48	76.93

Description : Tube Supporting Deck

Load Combination		Max Stress Ratios		Summary of Moment Values							Summary of Shear Values		
Segment Length	Span #	M	V	Mmax +	Mmax -	Ma Max	Mnx	Mnx/Omega	Cb	Rm	Va Max	Vnx	Vnx/Omega
Dsgn. L = 2.00 ft	3	0.006	0.003		-0.22	0.22	60.83	36.43	1.00	1.00	0.22	128.48	76.93
+D+0.60W+H													
Dsgn. L = 2.00 ft	1	0.007	0.007		-0.26	0.26	60.83	36.43	1.00	1.00	0.55	128.48	76.93
Dsgn. L = 8.50 ft	2	0.025	0.007	0.92	-0.26	0.92	60.83	36.43	1.00	1.00	0.55	128.48	76.93
Dsgn. L = 2.00 ft	3	0.007	0.003		-0.26	0.26	60.83	36.43	1.00	1.00	0.26	128.48	76.93
+D+0.70E+H													
Dsgn. L = 2.00 ft	1	0.004	0.004		-0.14	0.14	60.83	36.43	1.00	1.00	0.30	128.48	76.93
Dsgn. L = 8.50 ft	2	0.014	0.004	0.50	-0.14	0.50	60.83	36.43	1.00	1.00	0.30	128.48	76.93
Dsgn. L = 2.00 ft	3	0.004	0.002		-0.14	0.14	60.83	36.43	1.00	1.00	0.14	128.48	76.93
+D+0.750Lr+0.750L+0.450W+H													
Dsgn. L = 2.00 ft	1	0.010	0.010		-0.35	0.35	60.83	36.43	1.00	1.00	0.74	128.48	76.93
Dsgn. L = 8.50 ft	2	0.034	0.010	1.23	-0.35	1.23	60.83	36.43	1.00	1.00	0.74	128.48	76.93
Dsgn. L = 2.00 ft	3	0.010	0.005		-0.35	0.35	60.83	36.43	1.00	1.00	0.35	128.48	76.93
+D+0.750L+0.750S+0.450W+H													
Dsgn. L = 2.00 ft	1	0.010	0.009		-0.35	0.35	60.83	36.43	1.00	1.00	0.68	128.48	76.93
Dsgn. L = 8.50 ft	2	0.029	0.009	1.07	-0.35	1.07	60.83	36.43	1.00	1.00	0.68	128.48	76.93
Dsgn. L = 2.00 ft	3	0.008	0.004		-0.31	0.31	60.83	36.43	1.00	1.00	0.31	128.48	76.93
+D+0.750L+0.750S+0.5250E+H													
Dsgn. L = 2.00 ft	1	0.007	0.006		-0.26	0.26	60.83	36.43	1.00	1.00	0.49	128.48	76.93
Dsgn. L = 8.50 ft	2	0.021	0.006	0.76	-0.26	0.76	60.83	36.43	1.00	1.00	0.49	128.48	76.93
Dsgn. L = 2.00 ft	3	0.006	0.003		-0.22	0.22	60.83	36.43	1.00	1.00	0.22	128.48	76.93
+0.60D+0.60W+0.60H													
Dsgn. L = 2.00 ft	1	0.006	0.006		-0.20	0.20	60.83	36.43	1.00	1.00	0.43	128.48	76.93
Dsgn. L = 8.50 ft	2	0.020	0.006	0.72	-0.20	0.72	60.83	36.43	1.00	1.00	0.43	128.48	76.93
Dsgn. L = 2.00 ft	3	0.006	0.003		-0.20	0.20	60.83	36.43	1.00	1.00	0.20	128.48	76.93
+0.60D+0.70E+0.60H													
Dsgn. L = 2.00 ft	1	0.002	0.002		-0.09	0.09	60.83	36.43	1.00	1.00	0.18	128.48	76.93
Dsgn. L = 8.50 ft	2	0.008	0.002	0.30	-0.09	0.30	60.83	36.43	1.00	1.00	0.18	128.48	76.93
Dsgn. L = 2.00 ft	3	0.002	0.001		-0.09	0.09	60.83	36.43	1.00	1.00	0.09	128.48	76.93

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
	1	0.0000	0.000	+D+0.750Lr+0.750L+0.450W+H	-0.0098	0.000
+D+0.750Lr+0.750L+0.450W+H	2	0.0154	4.307		0.0000	0.000
	3	0.0000	4.307	+D+0.750Lr+0.750L+0.450W+H	-0.0098	2.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

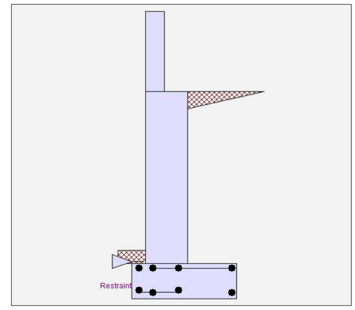
Load Combination	Support 1	Support 2	Support 3	Support 4
Overall MAXimum		1.090	1.090	
Overall MINimum		0.268	0.268	
+D+H		0.447	0.447	
+D+L+H		0.447	0.447	
+D+Lr+H		0.937	0.937	
+D+S+H		0.845	0.753	
+D+0.750Lr+0.750L+H		0.815	0.815	
+D+0.750L+0.750S+H		0.746	0.677	
+D+0.60W+H		0.815	0.815	
+D+0.70E+H		0.447	0.447	
+D+0.750Lr+0.750L+0.450W+H		1.090	1.090	
+D+0.750L+0.750S+0.450W+H		1.021	0.952	
+D+0.750L+0.750S+0.5250E+H		0.746	0.677	
+0.60D+0.60W+0.60H		0.636	0.636	
+0.60D+0.70E+0.60H		0.268	0.268	
D Only		0.447	0.447	
Lr Only		0.490	0.490	
L Only				
S Only		0.398	0.306	
W Only		0.613	0.613	
E Only				
H Only				

Cantilevered Retaining Wall

Code: IBC 2012,ACI 318-11,ACI 530-11

Criteria

Retained Height	=	6.50 ft
Wall height above soil	=	3.00 ft
Slope Behind Wall	=	0.00
Height of Soil over Toe	=	6.00 in
Water height over heel	=	0.0 ft



Load Factors

Building Code	IBC 2012,ACI
Dead Load	1.200
Live Load	1.600
Earth, H	1.600
Wind, W	1.000
Seismic, E	1.000

Soil Data and Lateral Earth Pressure

Allow Soil Bearing	=	2,700.0 psf	Soil Density, Heel	=	115.00 pcf
Equivalent Fluid Pressure Method			Soil Density, Toe	=	0.00 pcf
At-Rest Heel Pressure	=	35.0 psf/ft	Footing||Soil Friction	=	0.400
	=		Soil height to ignore		
Passive Pressure	=	250.0 psf/ft	for passive pressure	=	12.00 in

Surcharge Loads

Surcharge Over Heel	=	0.0 psf	Surcharge Over Toe	=	0.0 psf
Used To Resist Sliding & Overturning			Used for Sliding & Overturning		

Axial Load Applied to Stem

Axial Dead Load	=	250.0 lbs	Axial Load Eccentricity	=	0.0 in
Axial Live Load	=	0.0 lbs			

Lateral Load Applied to Stem

Lateral Load	=	0.0 #/ft
...Height to Top	=	0.00 ft
...Height to Bottom	=	0.00 ft
Load Type	=	Wind (W) (Service Level)

Wind on Exposed Stem

Wind on Exposed Stem (Service Level)	=	20.0 psf

Adjacent Footing Load

Adjacent Footing Load	=	0.0 lbs	Footing Type		Line Load
Footing Width	=	0.00 ft	Base Above/Below Soil		
Eccentricity	=	0.00 in	at Back of Wall	=	0.0 ft
Wall to Ftg CL Dist	=	0.00 ft	Poisson's Ratio	=	0.300

Wall Design Summary

Stability Ratios

Overturning	=	2.25 OK
Slab Resists All Sliding !		

Soil Bearing

Total Bearing Load	=	4,071 lbs
...resultant ecc.	=	10.14 in
Soil Pressure @ Toe	=	2,634 psf OK
Soil Pressure @ Heel	=	0 psf OK
Allowable	=	2,700 psf
Soil Pressure Less Than Allowable		
ACI Factored @ Toe	=	3,688 psf
ACI Factored @ Heel	=	0 psf
Footing Shear @ Toe	=	1.2 psi OK
Footing Shear @ Heel	=	7.9 psi OK
Allowable	=	75.0 psi

Sliding

Resisting Forces

Vertical Forces	Force	
Soil Over Heel	1,308.1	lbs
Sloped Soil Over Heel	0.0	
Surcharge Over Heel	0.0	
Adjacent Footing Load	0.0	
Axial Dead Load on Stem	250.0	
Axial Live Load on Stem *	Omit	
Soil Over Toe	0.0	
Surcharge Over Toe	0.0	
Stem Weight(s)	1,762.5	
Earth @ Stem Transitions	0.0	
Footing Weight	750.0	
Key Weight	0.0	
Vert. Component **	0.0	
Total Vertical Loads	4,070.6	lbs

* Axial live load NOT included in total displayed , or used for overturning or sliding resistance, but is included for soil pressure calculations.

Sliding Forces

Lateral Forces	Force	
* Heel Active Pressure	1,073.8	lbs
Surcharge over Heel	0.0	
Adjacent Footing	0.0	
Surcharge Over Toe	0.0	
Load @ Stem Above Soil	60.0	
Added Lateral Load	0.0	
Seismic Load	0.0	
Seismic-Self-weight	0.0	
Lateral on Key	0.0	
Totals =	1,133.8	lbs

*Includes water table effect

Sliding Calcs

Lateral Sliding Force	=	1,133.8 lbs

Vertical component of active lateral soil pressure IS NOT considered in the calculation of soil bearing pressures.

Overturning

Resisting Moments

Resisting Moments	Force		Distance		Moment	
Soil Over Heel	1,308.1	lbs	2.88	ft	3,760.9	ft-#
Sloped Soil Over Heel	0.0					
Surcharge Over Heel	0.0					
Adjacent Footing Load	0.0					
Axial Dead Load on Stem	250.0		1.25		312.5	
Axial Live Load on Stem *	0.0					
Soil Over Toe	0.0		0.25			
Surcharge Over Toe	0.0					
Stem Weight(s)	1,762.5		1.18		2,078.1	
Earth @ Stem Transitions	0.0					
Footing Weight	750.0		1.88		1,406.3	
Key Weight	0.0		2.50			
Vert. Component	0.0					
Total Vertical Loads	4,070.6	lbs				
Resisting Moment					**7,557.7**	ft-#
Eccentricity					**10.1**	in

* Axial live load NOT included in total displayed, or used for overturning or sliding resistance, but is included for soil pressure calculations.

Overturning

Overturning Moments

Overturning Moments	Force		Distance		Moment	
Heel Active Pressure	1,073.8	lbs	2.61	ft	2,803.9	ft-#
Surcharge over Heel	0.0					
Adjacent Footing	0.0					
Surcharge Over Toe	0.0					
Load @ Stem Above Soil	60.0		9.33		560.0	
Added Lateral Load	0.0					
Seismic Load	0.0					
Seismic-Self-weight	0.0					
Totals =	1,133.8	lbs				
Overturning Moment					3,363.9	ft-#

Stem Design Summary

			2nd	Bottom
			Stem OK	As < Min %
Design Height Above Ftg	ft	=	6.50	0.00
Wall Material Above "Ht"		=	Concrete	Concrete
Design Method		=	LRFD	LRFD
Thickness		=	8.00	18.00
Rebar Size		=	# 5	# 5
Rebar Spacing		=	12.00	12.00
Rebar Placed at		=	Center	14 in
Design Data				
fb/FB + fa/Fa		=	**0.018**	**No Good**
Total Force @ Section				
Service Level	lbs	=		
Strength Level	lbs	=	60.0	1,243.0
Moment....Actual				
Service Level	ft-#	=		
Strength Leve	ft-#	=	90.0	3,043.2
Moment.....Allowable	ft-#	=	5,069.7	19,019.7
Shear.....Actual				
Service Level	psi	=		
Strength Leve	psi	=	1.3	7.4
Shear.....Allowable	psi	=	75.0	75.0
Anet	in2	=		
Rebar Depth 'd'	in	=	4.00	14.00
Masonry Data				
f'm	psi	=		
Fs	psi	=		
Solid Grouting		=		
Modular Ratio 'n'		=		
Wall Weight	psf	=	100.0	225.0
Short Term Factor		=		
Equiv. Solid Thick.		=		
Masonry Block Type		=	Medium Weight	
Masonry Design Method		=	ASD	
Concrete Data				
f'c	psi	=	2,500.0	2,500.0
Fy	psi	=	60,000.0	60,000.0

Concrete Stem Rebar Area Details

2nd Stem	Vertical Reinforcing	Horizontal Reinforcing
As (based on applied moment) :	0.0054 in2/ft	
(4/3) * As :	0.0072 in2/ft	Min Stem T&S Reinf Area 0.576 in2
200bd/fy : 200(12)(4)/60000 :	0.16 in2/ft	Min Stem T&S Reinf Area per ft of stem Height : 0.192 in2/ft
0.0018bh : 0.0018(12)(8) :	0.1728 in2/ft	Horizontal Reinforcing Options :
	============	One layer of : Two layers of :
Required Area :	0.1728 in2/ft	#4@ 12.50 in #4@ 25.00 in
Provided Area :	0.31 in2/ft	#5@ 19.38 in #5@ 38.75 in
Maximum Area :	0.5419 in2/ft	#6@ 27.50 in #6@ 55.00 in

Bottom Stem	Vertical Reinforcing	Horizontal Reinforcing
As (based on applied moment) :	0.0494 in2/ft	
(4/3) * As :	0.0659 in2/ft	Min Stem T&S Reinf Area 2.808 in2
200bd/fy : 200(12)(14)/60000 :	0.56 in2/ft	Min Stem T&S Reinf Area per ft of stem Height : 0.432 in2/ft
0.0018bh : 0.0018(12)(18) :	0.3888 in2/ft	Horizontal Reinforcing Options :
	============	One layer of : Two layers of :
Required Area :	0.3888 in2/ft	#4@ 5.56 in #4@ 11.11 in
Provided Area :	0.31 in2/ft	#5@ 8.61 in #5@ 17.22 in
Maximum Area :	1.8966 in2/ft	#6@ 12.22 in #6@ 24.44 in

Footing Design Results

		Toe	Heel	
Factored Pressure	=	3,688	0	psf
Mu' : Upward	=	436	258	ft-#
Mu' : Downward	=	39	1,741	ft-#
Mu: Design	=	397	1,483	ft-#
Actual 1-Way Shear	=	1.20	7.90	psi
Allow 1-Way Shear	=	75.00	75.00	psi

Toe Reinforcing = # 5 @ 16.00 in
Heel Reinforcing = # 5 @ 18.00 in
Key Reinforcing = None Spec'd

Other Acceptable Sizes & Spacings
Toe: Not req'd: Mu < phi*5*lambda*sqrt(f'c)*Sm
Heel: Not req'd: Mu < phi*5*lambda*sqrt(f'c)*Sm
Key: Slab Resists Sliding - No Force on Key

Min footing T&S reinf Area 1.30 in2
Min footing T&S reinf Area per fc 0.35 in2 /ft

If one layer of horizontal bars:	If two layers of horizontal bars:
#4@ 6.94 in	#4@ 13.89 in
#5@ 10.76 in	#5@ 21.53 in
#6@ 15.28 in	#6@ 30.56 in

Tilt

Horizontal Deflection at Top of Wall due to settlement of soil

(Deflection due to wall bending not considered)

Soil Spring Reaction Modulus 250.0 pci
Horizontal Defl @ Top of Wall (approximate only) 0.185 in

The above calculation is not valid if the heel soil bearing pressure exceeds that of the toe, because the wall would then tend to rotate into the retained soil.