BUEHLER

STRUCTURAL CALCULATIONS

FOR

DMV DELANO

DELANO, CA

4/3/2020

REGISTERED PROFESSIONAL ENGINEER
LAWRENCE E. JONES
No. 3138
STRUCTURAL
STATE OF CALIFORNIA

Client: Nacht and Lewis
Buehler Principal in Charge: Larry Jones ,SE
Buehler Project Engineer: Gaurav Bali, PE, Ian Geocaris
Buehler Job No.: 2018-0187

Project Description and Design Criteria *T1*
Roof Framing Design *R1*
Column and Wall (Vert + Out of Plane) Design *C1*
Lateral Design and Force Distribution *L1*
Diaphragm Design *D1*
Chord, Collector, and Drag Design *CD1*
Foundation Design *FD1*
Miscellaneous *M1*
Covered Walkways (Detached) *CW1*
Equipment Anchorage Design *E1*
Appendix/Supplemental Information *A1*

OFFICE OF THE STATE FIRE MARSHAL
APPROVED FIRE AND PANIC ONLY

CALIFORNIA DEPARTMENT OF FORESTRY & FIRE PROTECTION
CAL FIRE
SINCE 1885

01/07/21

20-1059

Approval of this plan does not authorize or approve any omission or deviation from applicable regulations. Final approval is subject to field inspection. One set of approved plans shall be available on the project site at all times.

STATE FIRE MARSHAL APPROVAL

Project Description:

This project consists of a one story public service and office building in Delano, CA. Main building structure is concrete masonry walls; steel frame roof with open-web joists and metal deck. Lateral system is metal deck diaphragm spanning to perimeter concrete masonry walls. Floor is slab on grade. Foundations are conventional grade beams and pad footings at columns. Building is to attain Net Zero sustainability goals.

Project Location: Dover Parkway in Delano, CA

Design Criteria:

Building Type: Risk Category: II

Design Codes: 2019 California Building Code (2018 IBC)

ASCE 7-16:	Minimum Design Loads for Buildings and Other Structures
AISC 360-16:	Specification for Structural Steel Buildings
AISC 341-16:	Seismic Provisions for Structural Steel Buildings
AISI S100-16:	North American Specification for the Design of Cold-Formed Steel Structural Members
AISI S240-15:	North American Standard for Cold-Formed Steel Structural Framing
AISI S400-15:	North American Standard for Seismic Design of Cold-Formed Steel Structural Systems
ACI 318-14:	Building Code Requirements for Structural Concrete
TMS 402/602-16:	Building Code Requirements and Specifications for Masonry Structures
ANSI/AWC NDS-18:	National Design Specification (NDS) for Wood Construction with 2018 Supplement.
AWC SDPWS-15:	Special Design Provisions for Wind and Seismic

Wind Criteria: 95 mph, Exposure C $GC_{pi} = \pm 0.18$ (Enclosed Structure)

Seismic Criteria: Site Class: D $S_S = 0.723$ $S_1 =$ 0.N/A $S_{DS} = 0.589$

Seismic Design Category: D $I = 1.00$ $I_p = 1.00$

Lateral Force-Resisting System: Bearing Wall: Special Reinforced Masonry Shear Wall
$R = 5.0$ $\Omega_o = 2.0$ $C_d = 3.5$

Foundation Criteria:

Soils Report #11743.01P by Wallace Kuhl and Assoc., dated January 10, 2018

Allowable Bearing Pressures:

D + L	3000 psf
D + L + 0.7E	4000 psf

Coefficient of Friction	0.30
Passive Pressure	300 pcf

Reduce friction 50% when used in combination with passive pressure

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Material Specifications:

Concrete: (Cement ASTM C-150)

		W/C Ratio
Footings:	f'_c = 3000 psi	0.58
Pier / pile caps:	f'_c = 4000 psi	0.50
Slab-on-grade:	f'_c = 3500 psi	0.45
Elevated Slabs & Bms:	f'_c = 4000 psi	0.50
Columns:	f'_c = 4000 psi	0.50
Shear Walls:	f'_c = 4000 psi	0.50

Masonry: (Follow up with Dan to confirm values)

Concrete Masonry:	f'_m = 2000 psi
Clay Masonry:	f'_m = 2500 psi
Grout:	Strength to equal f'_m value noted above
Mortar:	Type S

Structural Steel:

WF shapes:	ASTM A992 (50 ksi)
HSS (rectangular):	ASTM A500-C (50 ksi)
HSS (round):	ASTM A500-C (46 ksi)
Pipe:	ASTM A53 (35 ksi)
Channel & Angle:	ASTM A36 (36 ksi)
Plate:	ASTM A36 (36 ksi)
Anchor Rods (non-frame):	ASTM F1554 Gr 36

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A: Roof Loads – Steel Non-Composite

	deck	beams	girders	columns	seismic
single ply w/o rigid insulation	0.6	0.6	0.6	0.6	0.6
1/2" DensDeck	2.0	2.0	2.0	2.0	2.0
6" max rigid insulation	1.2	1.2	1.2	1.2	1.2
1 1/2"x18ga galv 'B' deck	2.9	2.9	2.9	2.9	2.9
LH steel joists @ 6'-9"cc		3.5	3.5	3.5	3.5
HSS columns				0.8	0.8
sprinklers	1.5	1.5	1.5	1.5	1.5
MEP	3.0	3.0	3.0	3.0	3.0
susp gyp ceilings	3.8	3.8	3.8	3.8	3.8
PV panels		5.0	5.0	5.0	5.0
misc	1.0	1.0	1.0	0.7	0.7
DPR - non-composite DL	16.0 psf	24.5 psf	24.5 psf	25.0 psf	25.0 psf
LLR - reducible roof LL	20.0 psf	20.0 psf	20.0 psf	20.0 psf	)sf
TL	36.0 psf	44.5 psf	44.5 psf	45.0 psf	45.0 psf

B: Wall Loads

Wall Finish:	8" cmu w/ insul conc panel	8" cmu	6" Stud
8" cmu	80	80	
4" conc panel w/ veneer	5		
6" metal stud furring			1.5
5/8" gyp board			2.8
thin brick veneer	11.0		
1/2" shtg			1.7
insulation	2.0	2.0	2.0
MEP	0.5	0.5	0.5
misc	1.5	2.5	1.5
	100psf	85psf	10psf

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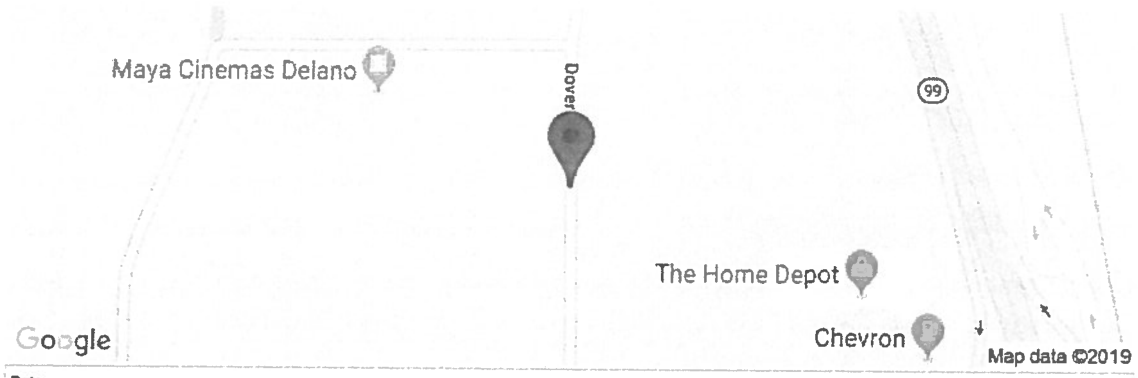
OSHPD

DMV Delano

Dover Pkwy, Delano, CA 93215, USA

Latitude, Longitude: 35.749084, -119.24962900000003

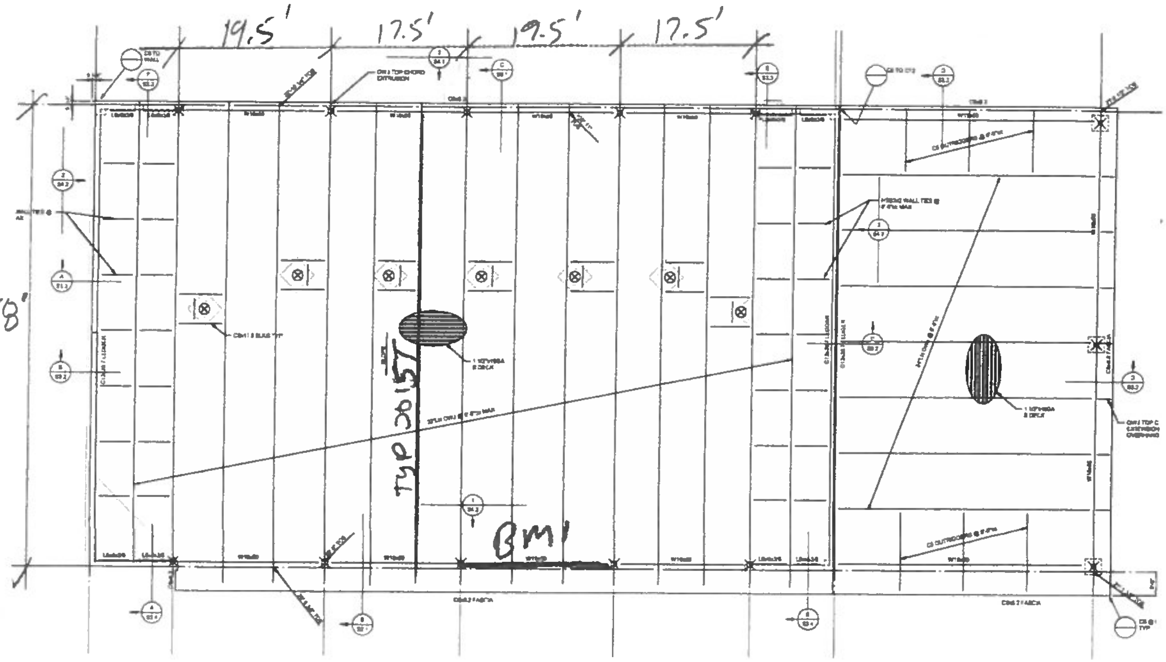


Date	11/7/2019, 10:51:01 AM
Design Code Reference Document	ASCE7-16
Risk Category	II
Site Class	D - Stiff Soil

Type	Value	Description
S_S	0.723	MCE_R ground motion. (for 0.2 second period)
S_1	0.272	MCE_R ground motion. (for 1.0s period)
S_{MS}	0.883	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8	Site-modified spectral acceleration value
S_{DS}	0.589	Numeric seismic design value at 0.2 second SA
S_{D1}	null -See Section 11.4.8	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	null -See Section 11.4.8	Seismic design category
F_a	1.221	Site amplification factor at 0.2 second
F_v	null -See Section 11.4.8	Site amplification factor at 1.0 second
PGA	0.316	MCE_G peak ground acceleration
F_{PGA}	1.284	Site amplification factor at PGA
PGA_M	0.406	Site modified peak ground acceleration
T_L	12	Long-period transition period in seconds
SsRT	0.723	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	0.784	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	1.5	Factored deterministic acceleration value. (0.2 second)
S1RT	0.272	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.295	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.6	Factored deterministic acceleration value. (1.0 second)
PGAd	0.5	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.923	Mapped value of the risk coefficient at short periods
C_{R1}	0.923	Mapped value of the risk coefficient at a period of 1 s

Framing Design



TYPICAL JOIST: DL – 24.5 psf Lr = 20 psf

$w = 1.2(24.5) + 1.6(20\,psf) = 61.5\,psf(6.7') = 412\,plf$

$w_{ASD} = 298\,plf$

USE 32LH O.W.J

Reactions: $DL = 24.5(6.7')(58')/2 = 4.8^k$

$Lr = 20(6.7')(58')/2 = 3.9^k$

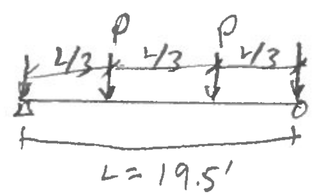
BEAM 1:

$P_{ASD} = 8.7^k$

$P_{LRFD} = 1.2(4.8^k) + 1.6(3.9^k)$

$= 12^k$

NO LL reduction.



$M_U = 12^k(19.5'/3) = 78\,k\text{-}ft$

$I_{req} = \dfrac{8.7^k(19.5' \times 12'')^3}{28(29000\,ksi)\left(\dfrac{19.5' \times 12''}{240}\right)} = 141\,in^4$

USE W18X35 min

$I = 510\,in^4$

$\phi M_n = 216\,k\text{-}ft$

seismic will govern size

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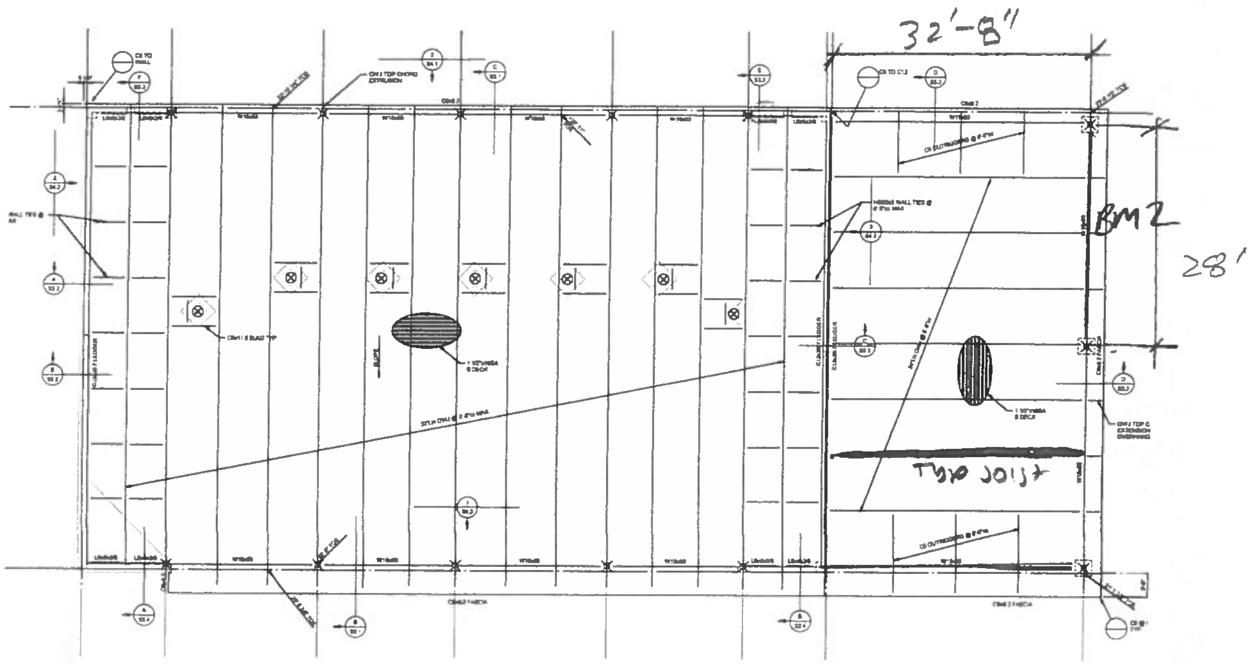
Framing Design



TYPICAL OWJ @ CARPORT: $W = (1.2(24.5psf) + 1.6(20psf))(6.7')$

$= 408$ plf (LRFD)

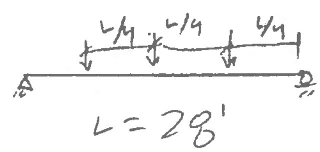
$w_{ASD} = 298$ plf

USE 24LH03

BEAM 2:

$P_{ASD} = 44.5psf(6.7')(33'/2) = 5^k$

$P_{LRFD} = 408psf(33')/2 = 6.7^k$



$M_U = 0.5(6.7^k)(28') = 93.8$ k-ft

$$I_{req} = \frac{.050(5^k)(28'\times 12")^3}{(29000)(28'\times 12"/240)} = 234 \text{ in}^4$$

USE W18X35 MIN

$I = 510 \text{ in}^4$

$\phi M_n = 216$ k-ft

R3

LRFD

STANDARD LOAD TABLE FOR LONGSPAN STEEL JOISTS, LH-SERIES
Based on a 50 ksi Maximum Yield Strength - Loads Shown In Pounds Per Linear Foot (plf)

Joist Designation	Approx. Wt in Lbs. Per Linear Ft. (Joists only)	Depth in inches	Max Load (plf)	SAFELOAD* in Lbs. Between		SPAN IN FEET														
			< 29	29-33		34	35	36	37	38	39	40	41	42	43	44	45	46	47	48
24LH03	11	24	601	17430		513	508	504	484	460	439	418	400	382	366	351	336	322	310	298
						235	226	218	204	188	175	162	152	141	132	124	116	109	102	96
24LH04	12	24	737	21360		628	597	568	540	514	490	468	447	427	409	393	376	361	346	333
						288	265	246	227	210	195	182	169	158	148	138	130	122	114	107
24LH05	13	24	789	22890		673	669	660	628	598	570	544	520	496	475	456	436	420	403	387
						308	297	285	264	244	226	210	196	182	171	160	150	141	132	124
24LH06	16	24	1061	30780		906	868	832	795	756	720	685	655	625	598	571	546	522	501	480
						411	382	356	331	306	284	263	245	228	211	197	184	172	161	152
24LH07	17	24	1166	33810		997	957	919	882	847	811	774	736	702	669	639	610	583	559	535
						452	421	393	367	343	320	297	276	257	239	223	208	195	182	171
24LH08	18	24	1243	36060		1060	1015	973	933	895	858	817	780	745	712	682	652	625	600	576
						480	447	416	388	362	338	314	292	272	254	238	222	208	196	184
24LH09	21	24	1464	42450		1248	1212	1177	1146	1096	1044	994	948	903	861	822	786	751	720	690
						562	530	501	460	424	393	363	337	313	292	272	254	238	223	209
24LH10	23	24	1547	44850		1323	1284	1248	1213	1182	1152	1105	1053	1002	955	912	873	834	799	766
						596	559	528	500	474	439	406	378	351	326	304	285	266	249	234
24LH11	25	24	1630	47280		1390	1350	1312	1276	1243	1210	1180	1152	1101	1051	1006	963	924	885	850
						624	588	555	525	498	472	449	418	388	361	337	315	294	276	259
			< 34	34-41		42	43	44	45	46	47	48	49	50	51	52	53	54	55	56
28LH05	13	28	623	21180		505	484	465	445	429	412	397	382	367	356	342	330	319	309	298
						219	205	192	180	169	159	150	142	133	126	119	113	107	102	97
28LH06	16	28	828	28140		672	643	618	592	568	546	525	505	486	469	451	436	421	406	393
						289	270	253	238	223	209	197	186	175	166	156	148	140	133	126
28LH07	17	28	934	31770		757	726	696	667	640	615	591	568	547	528	508	490	474	457	442
						326	305	285	267	251	236	222	209	197	186	176	166	158	150	142
28LH08	18	28	1001	34020		810	775	744	712	684	657	630	604	580	556	535	516	496	478	462
						348	325	305	285	268	252	236	222	209	196	185	175	165	156	148
28LH09	21	28	1232	41880		1000	958	918	879	844	810	778	748	721	694	669	645	622	601	580
						428	400	375	351	329	309	291	274	258	243	228	216	204	193	183
28LH10	23	28	1347	45810		1093	1056	1018	976	937	900	864	831	799	769	742	715	690	666	643
						466	439	414	388	364	342	322	303	285	269	255	241	228	215	204
28LH11	25	28	1445	49140		1170	1143	1104	1066	1023	982	943	907	873	841	810	781	753	727	702
						498	475	448	423	397	373	351	331	312	294	278	263	249	236	223
28LH12	27	28	1587	53970		1285	1255	1227	1200	1173	1149	1105	1063	1023	984	948	913	880	849	819
						545	520	496	476	454	435	408	383	361	340	321	303	285	270	256
28LH13	30	28	1654	56250		1342	1311	1281	1252	1224	1198	1173	1149	1126	1083	1041	1002	964	930	897
						569	543	518	495	472	452	433	415	396	373	352	332	314	297	281
			< 39	39-46	47-49	50	51	52	53	54	55	56	57	58	59	60	61	62	63	64
32LH06	14	32	647	25230	25230	507	489	472	456	441	426	412	399	385	373	363	351	340	330	321
						211	199	189	179	169	161	153	145	138	131	125	119	114	108	104
32LH07	16	32	728	28380	28380	568	549	529	511	493	477	462	447	432	418	406	393	381	370	360
						235	223	211	200	189	179	170	162	154	146	140	133	127	121	116
32LH08	17	32	790	30810	30810	616	595	574	553	535	517	499	483	468	453	439	426	412	400	388
						255	242	229	216	205	194	184	175	167	159	151	144	137	131	125
32LH09	21	32	992	38670	38670	774	747	720	694	670	648	627	606	586	568	550	534	517	502	487
						319	302	285	270	256	243	230	219	208	198	189	180	172	164	157
32LH10	21	32	1096	42750	42750	856	825	796	768	742	717	693	667	646	624	603	583	564	546	529
						352	332	315	297	282	267	254	240	228	217	206	196	186	178	169
32LH11	24	32	1201	46830	46830	937	903	870	840	811	783	757	732	709	687	664	643	624	604	585
						385	363	343	325	308	292	277	263	251	239	227	216	206	196	187
32LH12	27	32	1409	54960	54960	1101	1068	1032	996	961	928	897	867	838	811	786	762	738	715	694
						450	428	406	384	364	345	327	311	295	281	267	255	243	232	221
32LH13	30	32	1572	61320	61320	1225	1201	1177	1156	1113	1072	1035	999	964	931	900	871	843	816	790
						500	480	461	444	420	397	376	354	336	319	304	288	275	262	249
32LH14	33	32	1618	63120	63120	1264	1239	1215	1192	1170	1149	1107	1069	1032	997	964	933	903	874	848
						515	495	476	458	440	417	395	374	355	337	321	304	290	276	264
32LH15	35	32	1673	65250	65250	1305	1279	1255	1231	1207	1186	1164	1144	1125	1087	1051	1017	984	952	924
						532	511	492	473	454	438	422	407	393	374	355	338	322	306	292
			< 43	43-46	47-56 57	58	59	60	61	62	63	64	65	66	67	68	69	70	71	72
36LH07	16	36	590	25350	25350	438	424	411	399	387	376	366	355	345	336	327	318	310	301	294
						177	168	160	153	146	140	134	128	122	117	112	107	103	99	95
36LH08	18	36	649	27900	27900	481	466	453	439	426	414	402	390	379	369	358	349	340	331	322
						194	185	176	168	160	153	146	140	134	128	123	118	113	109	104
36LH09	21	36	832	35760	35760	616	597	579	561	544	528	513	499	484	471	459	445	433	423	412
						247	235	224	214	204	195	186	179	171	163	157	150	144	138	133
36LH10	21	36	916	39390	39390	681	660	639	619	601	583	567	550	535	520	507	492	480	466	454
						273	260	248	236	225	215	206	197	188	180	173	165	159	152	146
36LH11	23	36	1000	42990	42990	742	720	697	676	657	637	618	601	583	567	552	537	522	508	495
						297	283	269	257	246	234	224	214	205	196	188	180	173	166	159
36LH12	25	36	1197	51450	51450	889	862	835	810	784	762	739	717	696	675	655	636	618	600	583
						354	338	322	307	292	279	267	255	243	232	222	213	204	195	187
36LH13	30	36	1407	60510	60510	1045	1012	981	951	922	894	868	843	819	796	774	753	732	712	694
						415	395	376	359	342	327	312	298	285	273	262	251	240	231	222
36LH14	36	36	1551	66690	66690	1152	1132	1093	1059	1024	991	961	931	903	876	850	826	802	780	757
						456	434	412	392	373	356	339	323	309	295	283	270	259	247	237
36LH15	36	36	1635	70320	70320	1213	1192	1171	1153	1116	1081	1047	1015	984	955	927	900	874	850	826
						480	464	448	434	413	394	375	358	342	327	312	299	286	274	263

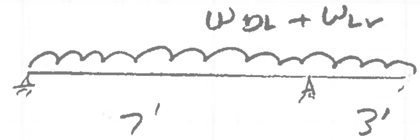
Strength
L/360

For L/240: 208 plf (360/240) = 312 plf > 298 plf
586 plf − 21 plf = 565 plf > 412 plf
→ OK.
32LH09

ROOF FRAMING CONT

C5 OUTRIGGERS

Trib. width = 7'

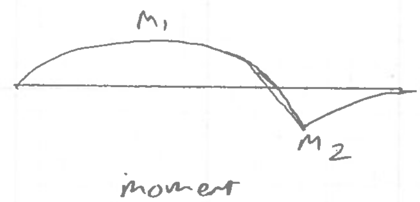


$w_{DL} = 24.5\,psf(7') = 168\,plf$

$w_{Lr} = 20\,psf(7') = 140\,plf$

$$M_1 = \frac{(1.2(168)+1.6(140))}{8(7')^2}(7+3)^2(7-3)^2$$

$= 1737$ #-ft



$$M_2 = \frac{(1.2(168)+1.6(140))(3')^2}{2}$$

$= 1915$ #-ft

$V_2 = 1277$# (FACTORED) $\quad V_2 = 924$ (ASD)

$$\Delta_{overhang} = \frac{(168+140)(3')}{24(E)(I)}\left(4(3')^2 7' - 7^3 + 6(3')^2(3') - 4(3')(3')^2 + 3'^3\right)$$

$= -0''$ ~ flat O.K.

$I = 7.48\,in^4$

Moment capacity of channel:

$L_p = 2' \quad L_r = 10.4' \quad L_b = 7'$

$$M_n = M_p - (M_p - 0.7(F_y S_x))\left(\frac{L_b - L_p}{L_r - L_p}\right)$$

$$= 9.59 - (9.59_{k\text{-}ft} - 0.7(36\,ksi)(2.99))\left(\frac{7-2}{10.4-2'}\right)$$

$= 91.5$ k-in $= 7.6$ k-ft

$M_p = 9.59$ k-ft

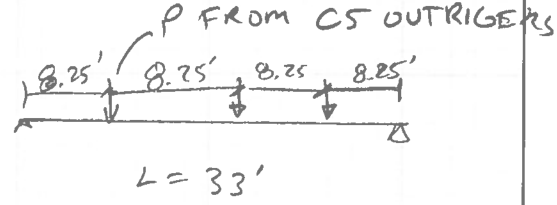
$S_x = 2.99\,in^3$

DCM = 2kft / 7.6 k-ft

= 0.26 O.K

ROOF FRAMING CONT.

- W12×26 @ ⊥ TO CARPORT



$w_{eq} = fP = 4P$

$P = 1277^{\#}$ $\qquad P_{ASD} = 924\ \#$

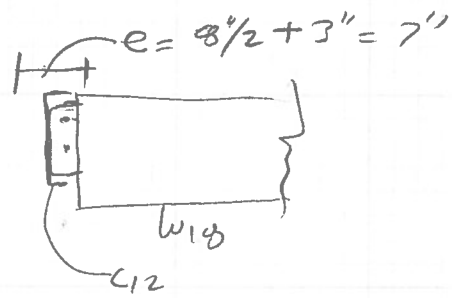
$W_{LRFD} = 4(1277^{\#})/33' = 155\ plf$ $\qquad w_{ASD} = 112\ plf$

$M = .155\,klf(33')^2/8 = 21\ k\text{-}ft < \phi M_n \approx 125\ k\text{-}ft$ w/ $L_b = 7'$

$$\Delta = \frac{5(.112)(33')^4(12^3)}{384(29000)(204\,in^4)} = 0.5'' < L/240 = 1.65''\quad O.K.$$

Reaction: $155\,plf(33')/2 = 2.6^k$ (LRFD)

M @ embed $= 2600^{\#}(7'') = 18.2\,k\text{-}in$



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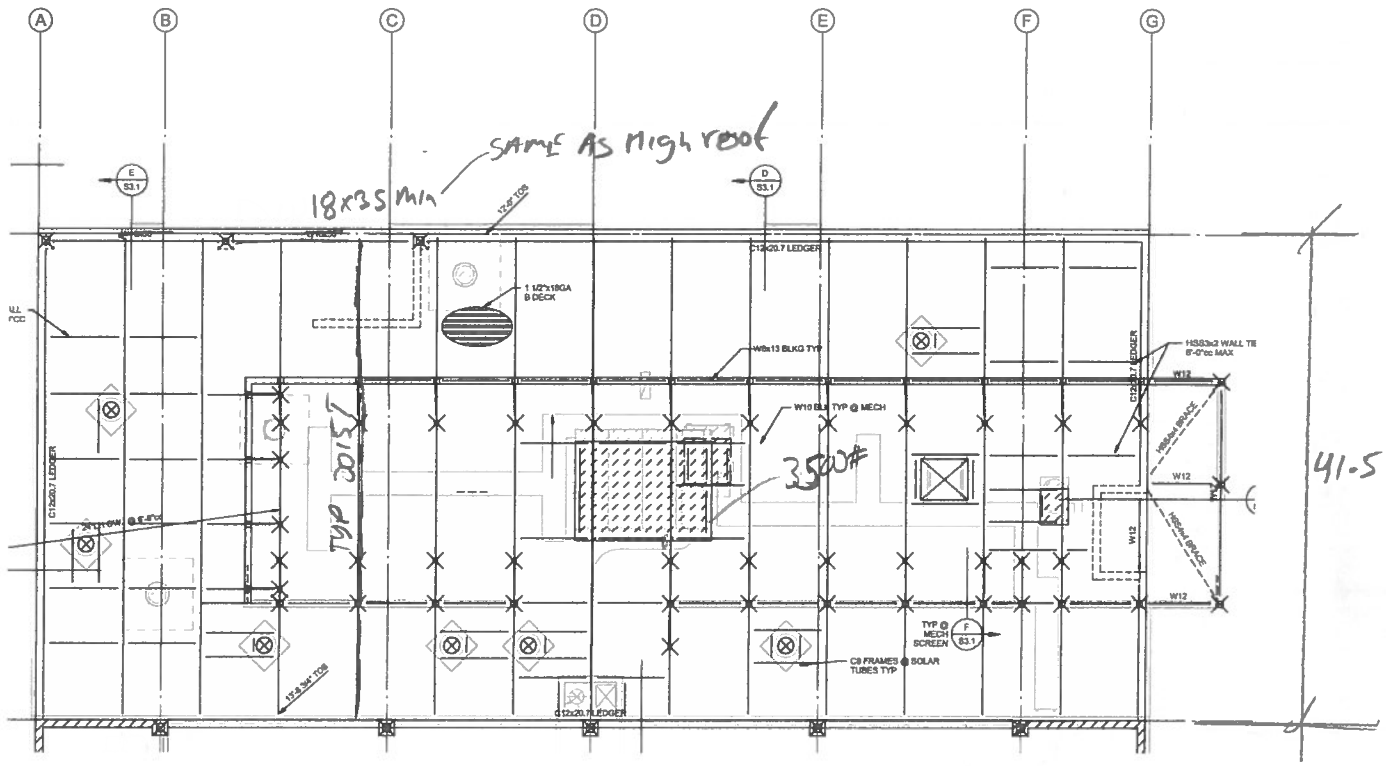
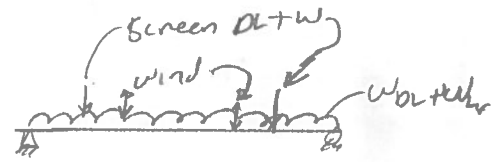
Framing Design



TYPICAL LOW ROOF JOIST :



$w_{ASD} = 298$ plf

$w_{LRFD} = 408$ plf

Add overall load : Screen DL = 560#

Wind T/C = 3k

USE 24LH06

Total load = 41.5'(408) + 3.5k(1.2)

+ 3500#/3 joists

= 22.3k < 30780# ... ok

R7

Low Roof

LRFD

STANDARD LOAD TABLE FOR LONGSPAN STEEL JOISTS, LH-SERIES
Based on a 50 ksi Maximum Yield Strength - Loads Shown In Pounds Per Linear Foot (plf)

Joist Designation	Approx. Wt in Lbs. Per Linear Ft. (Joists only)	Depth in inches	Max Load (plf)	SAFELOAD* in Lbs. Between		SPAN IN FEET														
			< 29	29-33		34	35	36	37	38	39	40	41	42	43	44	45	46	47	48
24LH03	11	24	601	17430		513 235	508 226	504 218	484 204	460 188	439 175	418 162	400 152	382 141	366 132	351 124	336 116	322 109	310 102	298 96
24LH04	12	24	737	21360		628 288	597 265	568 246	540 227	514 210	490 195	468 182	447 169	427 158	409 148	393 138	376 130	361 122	346 114	333 107
24LH05	13	24	789	22890		673 308	669 297	660 285	628 264	598 244	570 226	544 210	520 198	496 182	475 171	456 160	436 150	420 141	403 132	387 124
24LH06	16	24	1061	30780		906 411	868 382	832 356	795 331	756 306	720 284	685 263	655 245	625 228	598 211	571 197	546 184	522 172	501 161	480 152
24LH07	17	24	1166	33810		997 452	957 421	919 393	882 367	847 343	811 320	774 297	736 276	702 257	669 239	639 223	610 208	583 195	559 182	535 171
24LH08	18	24	1243	36060		1060 480	1015 447	973 416	933 388	895 362	858 338	817 314	780 292	745 272	712 254	682 238	652 222	625 208	600 196	576 184
24LH09	21	24	1464	42450		1248 562	1212 530	1177 501	1146 460	1096 424	1044 393	994 363	948 337	903 313	861 292	822 272	786 254	751 238	720 223	690 209
24LH10	23	24	1547	44850		1323 596	1284 559	1248 528	1213 500	1182 474	1152 439	1105 406	1053 378	1002 351	955 326	912 304	873 285	834 266	799 249	766 234
24LH11	25	24	1630	47280		1390 624	1350 588	1312 555	1276 525	1243 498	1210 472	1180 449	1152 418	1101 388	1051 361	1006 337	963 315	924 294	885 276	850 259
			< 34	34-41		42	43	44	45	46	47	48	49	50	51	52	53	54	55	56
28LH05	13	28	623	21180		505 219	484 205	465 192	445 180	429 169	412 159	397 150	382 142	367 133	355 126	342 119	330 113	319 107	309 102	298 97
28LH06	16	28	828	28140		672 289	643 270	618 253	592 238	568 223	546 209	525 197	505 186	486 175	469 166	451 158	436 148	421 140	406 133	393 126
28LH07	17	28	934	31770		757 326	726 305	696 285	667 267	640 251	615 236	591 222	568 209	547 197	528 186	508 176	490 166	474 158	457 150	442 142
28LH08	18	28	1001	34020		810 348	775 325	744 305	712 285	684 268	657 252	630 236	604 222	580 209	556 196	535 185	516 175	496 165	478 156	462 148
28LH09	21	28	1232	41880		1000 428	958 400	918 375	879 351	844 329	810 309	778 291	748 274	721 258	694 243	669 228	645 216	622 204	601 193	580 183
28LH10	23	28	1347	45810		1093 466	1056 439	1018 414	976 388	937 364	900 342	864 322	831 303	799 285	769 269	742 255	715 241	690 228	666 215	643 204
28LH11	25	28	1445	49140		1170 498	1143 475	1104 448	1066 423	1023 397	982 373	943 351	907 331	873 312	841 294	810 278	781 263	753 249	727 236	702 223
28LH12	27	28	1587	53970		1285 545	1255 520	1227 496	1200 476	1173 454	1149 435	1105 408	1063 383	1023 361	984 340	948 321	913 303	880 285	849 270	819 256
28LH13	30	28	1654	56250		1342 569	1311 543	1281 518	1252 495	1224 472	1198 452	1173 433	1149 415	1126 396	1083 373	1041 352	1002 332	964 314	930 297	897 281
			< 39	39-46	47-49	50	51	52	53	54	55	56	57	58	59	60	61	62	63	64
32LH06	14	32	647	25230	25230	507 211	489 199	472 189	456 179	441 169	426 161	412 153	399 145	385 138	373 131	363 125	351 119	340 114	330 108	321 104
32LH07	16	32	728	28380	28380	568 235	549 223	529 211	511 200	493 189	477 179	462 170	447 162	432 154	418 146	406 140	393 133	381 127	370 121	360 116
32LH08	17	32	790	30810	30810	616 255	595 242	574 229	553 216	535 205	517 194	499 184	483 175	468 167	453 159	439 151	426 144	412 137	400 131	388 125
32LH09	21	32	992	38670	38670	774 319	747 302	720 285	694 270	670 256	648 243	627 230	606 219	586 208	568 198	550 189	534 180	517 172	502 164	487 157
32LH10	21	32	1096	42750	42750	856 352	825 332	796 315	768 297	742 282	717 267	693 254	667 240	645 228	624 217	603 206	583 196	564 186	546 178	529 169
32LH11	24	32	1201	46830	46830	937 385	903 363	870 343	840 325	811 308	783 292	757 277	732 263	709 251	687 239	664 227	643 216	624 206	604 196	585 187
32LH12	27	32	1409	54960	54960	1101 450	1068 428	1032 406	996 384	961 364	928 345	897 327	867 311	838 295	811 281	786 267	762 255	738 243	715 232	694 221
32LH13	30	32	1572	61320	61320	1225 500	1201 480	1177 461	1156 444	1113 420	1072 397	1035 376	999 354	964 336	931 319	900 304	871 288	843 275	816 262	790 249
32LH14	33	32	1618	63120	63120	1264 515	1239 495	1215 476	1192 458	1170 440	1149 417	1107 395	1069 374	1032 355	997 337	964 321	933 304	903 290	874 276	846 264
32LH15	35	32	1673	65250	65250	1305 532	1279 511	1255 492	1231 473	1207 454	1186 438	1164 422	1144 407	1125 393	1087 374	1051 355	1017 338	984 322	952 308	924 292
			< 43	43-46	47-56 | 57	58	59	60	61	62	63	64	65	66	67	68	69	70	71	72
36LH07	16	36	590	25350	25350	438 177	424 168	411 160	399 153	387 146	376 140	366 134	355 128	345 122	336 117	327 112	318 107	310 103	301 99	294 95
36LH08	18	36	649	27900	27900	481 194	466 185	453 176	439 168	426 160	414 153	402 146	390 140	379 134	369 128	358 123	349 118	340 113	331 109	322 104
36LH09	21	36	832	35760	35760	616 247	597 235	579 224	561 214	544 204	528 195	513 186	499 179	484 171	471 163	459 157	445 150	433 144	423 138	412 133
36LH10	21	36	916	39390	39390	681 273	660 260	639 248	619 236	601 225	583 215	567 206	550 197	535 188	520 180	507 173	492 165	480 159	466 152	454 146
36LH11	23	36	1000	42990	42990	742 297	720 283	697 269	676 257	657 246	637 234	618 224	601 214	583 205	567 196	552 188	537 180	522 173	508 166	495 159
36LH12	25	36	1197	51450	51450	889 354	862 338	835 322	810 307	784 292	762 279	739 267	717 255	696 243	675 232	655 222	636 213	618 204	600 195	583 187
36LH13	30	36	1407	60510	60510	1045 415	1012 395	981 376	951 359	922 342	894 327	868 312	843 298	819 285	796 273	774 262	753 251	732 240	712 231	694 222
36LH14	36	36	1551	66690	66690	1152 456	1132 434	1093 412	1059 392	1024 373	991 356	961 339	931 323	903 309	876 295	850 283	826 270	802 259	780 247	757 237
36LH15	36	36	1635	70320	70320	1213 480	1192 464	1171 448	1153 434	1116 413	1081 394	1047 375	1015 358	984 342	955 327	927 312	900 299	874 286	850 274	826 263

228(360/240) = 342 plf > 298 .. ok.

625 > 408 plf .. ok.

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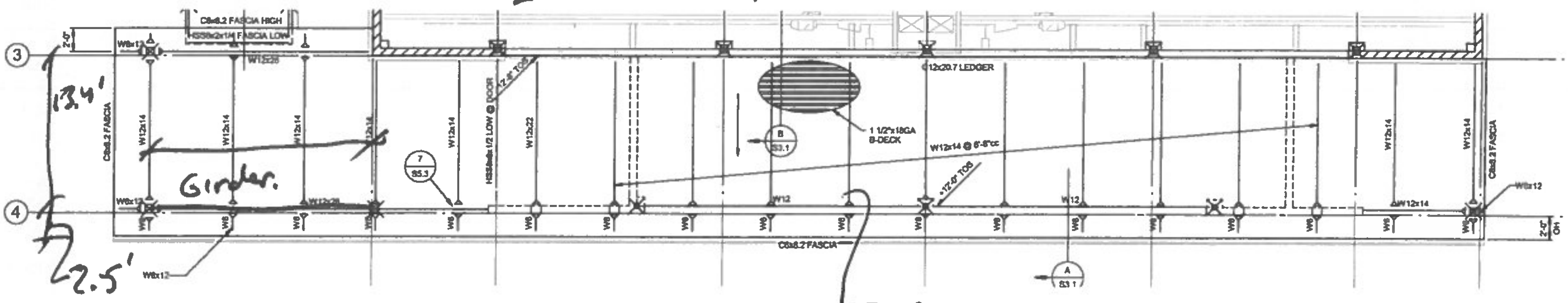
Framing Design

LOW ROOF FRAMING PLAN



Typical Beam

$$M_u = \left(\left(1.2(25\,psf) + 1.6(20\,psf)\right)6.8'\right)14'^2/8 = 10.4\,k\text{-}ft$$

$$I_{req} = \frac{5(.045\,ksf)(6.8')(14')^4(12^3)}{384(29000)(14'\times 12"/240)} = 13\,in^4$$

USE W12x14

$\phi M_n = 65\,k\text{-}f$ $\quad L_b = 1'$ (deck)

$I = 88\,in^4$

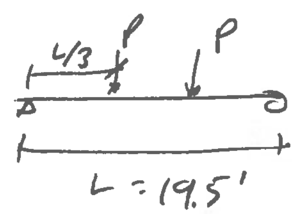
Girder

$$P_{ASD} = 45\,psf(6.8')(13.4'/2) = 2.1^k$$

$$P_{LRFD} = \left(1.2(25) + 1.6(20)\right)6.8'(13.4'/2)$$
$$= 2.9^k$$

$$M_u = 2.9^k(6.8') = 19.7\,k\text{-}ft$$

$$I_{req'd} = \frac{2.1^k(19.5'\times 12")^3}{28(29000)(19.5\times 12/240)} = 34\,in^4$$



USE W12x14 min

$\phi M_n = 60\,k\text{-}ft$ $\quad L_b = 8'$

$I = 88.6\,in^4$

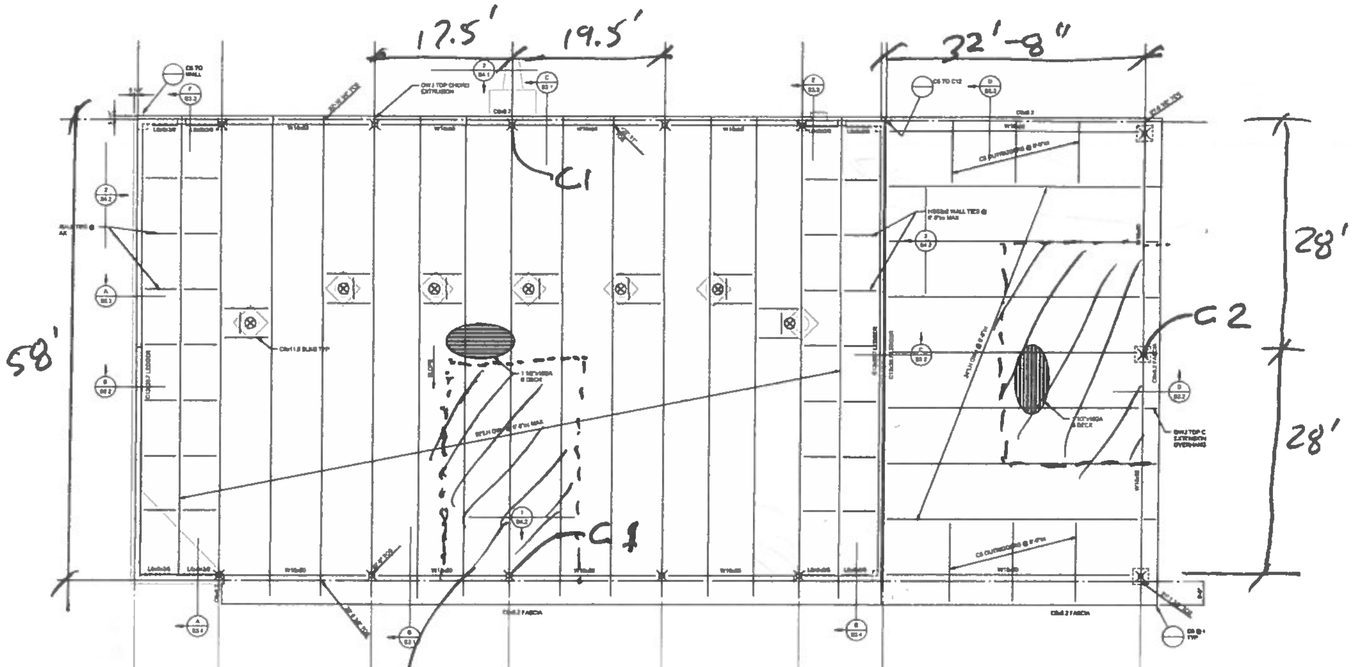
BUEHLER

PROJECT DMV Delano

PROJECT NO. 2018-0187 DATE 11/11/2019

CLIENT NLA BY GB PAGE NO. C1

COLUMN DESIGN



Column 1

L_r = 20 psf

$L_{r\,reduced} = 20(1.2 - 0.001(565))$
$= 12.7$ psf use 13

$A_T = \frac{58'}{2}(19.5') = 565.5$ sf

$P_{DL} = 25\text{ psf}(565.5) = 14.2^k$

$P_{Lr} = 13\text{ psf}(565.5) = 7.4^k$

$P_U = 1.2(14.2^k) + 1.6(7.4^k) = 29^k$

COLUMN 2

$A_T = 33/2\,(28') = 462$ sf

$L_{r\,reduced} = 20(1.2 - 0.001(462)) = 15$ psf

$P_{DL} = 25\text{ psf}(462\text{ sf}) = 11.6^k$

$P_{Lr} = 15(462) = 7^k$

$P_U = 25^k$

HSS 6x6x1/4 MIN

$\phi P_n = 85^k$ w/ L = 23'

PROJECT DMV Delano

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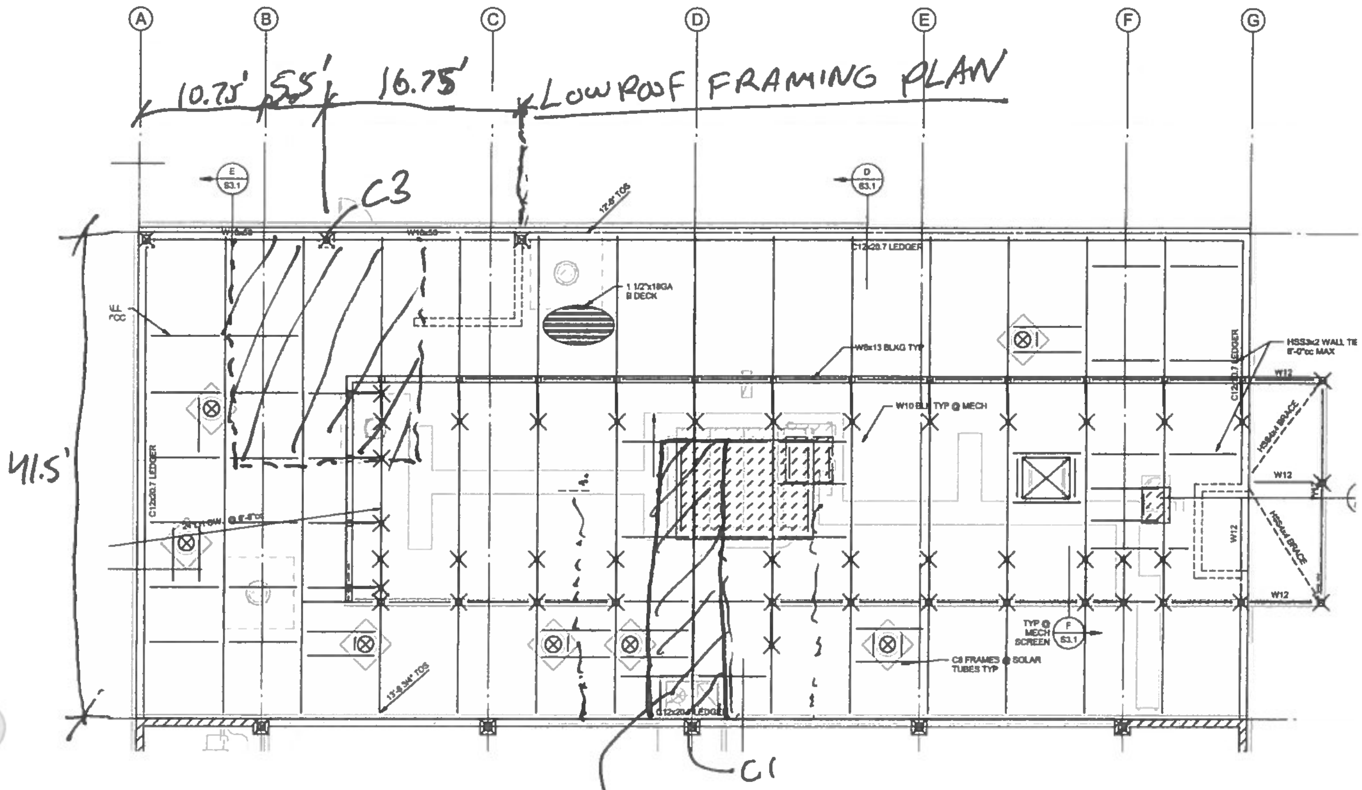
COLUMN DESIGN



Assume joist load goes to col

Column C1 cont.

$A_T = 7'(41.5'/2) = 50\text{ sf}$

$P_{DL} = 50(25) = 1.3^k \quad P_{Lr} = 20(25) = 0.5^k$

$P_U = 2.4^k$

$P_{U\,total} = 2.4^k + 29^k = 31.5^k$

$\phi P_n = 85^k$

HSS6x6x1/4 min
w/ $L_{unbraced} = 23\text{ft}$

Column C3

$A_T = 41.5/2\,(16.5') = 343^{sf}$

$P_U = 1.2(25\text{psf}(343)) + 1.6(20\text{psf}(1.2 - 0.001(343)))(343)$
$= 20k$

HSS6x6x1/4 min
w/ $L_{unbraced} = 23'$
$\phi P_n = 85^k$

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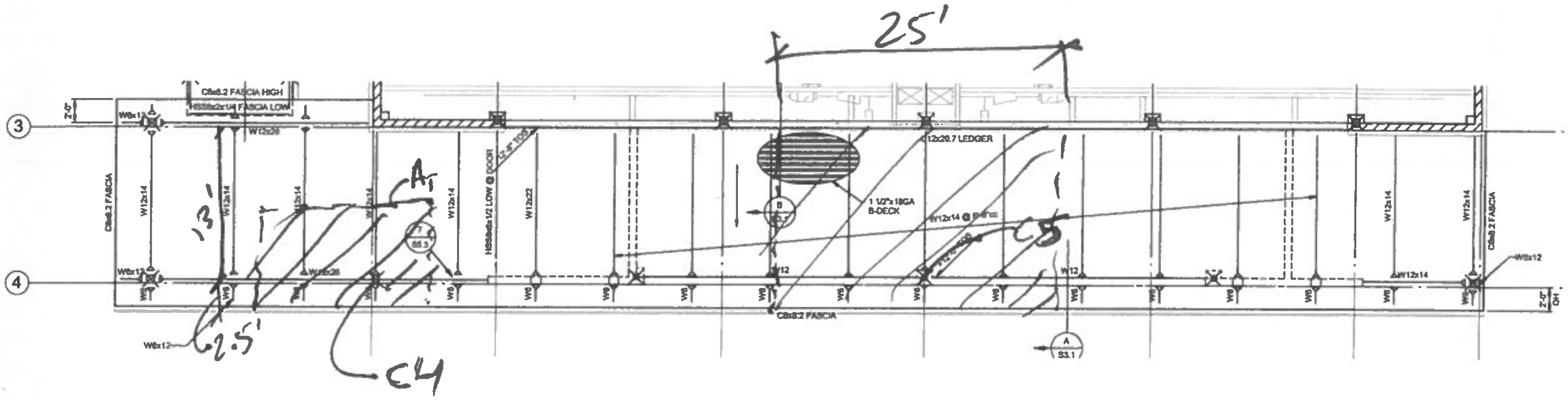
COLUMN DESIGN



COLUMN C4 :

$A_T = (19/2 + 10/2)(13/2 + 2.5') = 131 sf$

$P_u = (1.2(25 psf) + 1.6(20 psf))131 sf$

$= 8122^{\#}$

$\phi P_n = 107^k$

w/ KL = 20'

HSS6x6x1/4, MIN

COLUMN C5 :

$A_T = 25'(13/2 + 2.5') = 225 sf$

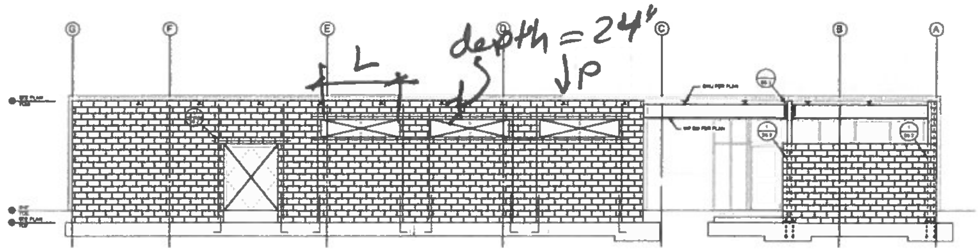
$P_{DL} = 25psf(225) = 5.7^k$

$P_{Lr} = 20(225) = 4.5^k$

$P_u = 1.2(5.7) + 1.6(4.5) = 14^k$ (LRFD)

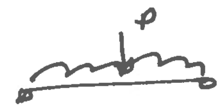
$P = 10.2^k$ (ASD)

Lintel Design: Lintel 1



NORTH WALL ELEV.; GRID 1

$L_{max} = 8'-8"$



$w_{s.w.}$

$P_{DL} = 25psf(41½)(6.7')$
$= 3434^{\#}$
$1750^{\#}$ (mech unit)
$100psf(2') = 200plf$

$P_{DL\ total} = 5180^{\#}$

$P_{Lr} = 20\left(\frac{41.5'}{2}\right)(6.7') = 2.8^{k}$

$M_{DL} = 5.2^{k}(8.67')/4 + 200(8.67)^2/8 = 30\ k\text{-}ft$

$M_{Lr} = 2.8(8.67)/4 = 6\ k\text{-}ft$

$V_{DL} = 3.5^{k}$ $V_{Lr} = 1.4^{k}$

Seismic - out-of-plane

$F_p = 0.4(0.6)(k=2)(I=1.0)(100psf) = 48\ psf$

$w = 48psf(2') = 96\ plf$

$M_u = 96(8.75')^2/8 = 1\ k\text{-}ft$

$V_u = 96(8.75)/2 = 420^{\#}$

$M/S = \frac{12k\text{-}in}{24"(7.625)^2/6} = 206psi$
(1.3)
$= 268 \text{ vs } 267\ psi$

Does not crack
Lintel ok for O-O-P

(2) #5 w/ 24" deep
Lintel OK.

See following pg.

T:
F: Lintel 1

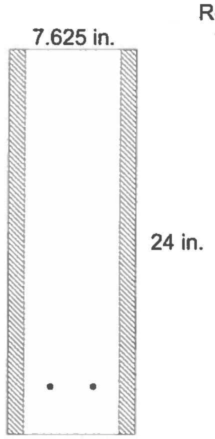
Project:
Number: C5
Date: December 02, 2019
By:
Sheet of

CMD15.00.00

Filename: lintel 1.dat

Reinforced Concrete Masonry Beam per 2016 CBC Section 2108 (2013 MSJC Section 9.3.4.2)
- Ultimate Strength Design -

Input:



Reinforcing dimensioned from bottom of beam:

2 # 5 @ 3 inches

Shear Steel:
0 # 3 @ 8 in. o.c.
Beam Section

Beam Geometry:
9.00' Long, 24.00" deep CMU beam
8" nominal thickness

Beam Material Properties:

f'_m = 2,000 psi
f_y = 60,000 psi
E_m = 1,800 ksi
E_s = 29,000 ksi
n = 16.11

Applied Loads:	P kips	V kips	M kip-ft
Dead Load	0.0	3.50	30.00
Floor Live Load	0.0	0.00	0.00
Roof Live Load	0.0	1.40	6.00
Snow Load	0.0	0.00	0.00
Horizontal Seismic Load	13.9	0.00	0.00
Vertical Seismic Load	0.0	0.00	0.00
Wind Load	0.0	0.00	0.00

collector load, see lateral design

Output:

P vs M Diagram Points:

		Positive Moments P_n, M_n	ϕP_n, ϕM_n	ϕ	Negative Moments P_n, M_n	ϕP_n, ϕM_n	ϕ
Maximum Axial Load (kips)	P_{max} =	329	237	0.90	329	237	0.90
Moment at P_{max} (kip-ft)	$M(P_{max})$ =	0	28	0.90	0	-28	0.90
Balanced Axial Load (kips)	P_b =	74	67	0.90	-21	-19	0.90
Balanced Moment (kip-ft)	M_b =	97	87	0.90	13	11	0.90
Moment at P=0 (kip-ft)	M_o =	60	54	0.90	-4	-3	0.90

General Calculations:

Total A_s = 0.62 in², ρ = 0.0039 ≤ 0.0088 = ρ_{max} - OK
Cracking Moment: M_{cr} = 16.3 kip-ft = $S_g * f_r$ = (732 in³)(0.267 ksi)/12
Minimum Nominal Flexural Strength of the Beam = 42.3 kip-ft ≥ 1.3*M_{cr} OK

Strength Checks - Moments:

	P_n kips	M_n kip-ft	ϕ	ϕP_n kips	ϕM_n kip-ft	P_u kips	M_u kip-ft	Stress Ratio
1.4D	0.0	60	0.90	0.0	54	0.0	42	0.77
1.2D+1.6Lr+0.5L	0.0	60	0.90	0.0	54	0.0	46	0.84
1.2D+1.6Lr+0.5W	0.0	60	0.90	0.0	54	0.0	46	0.84
1.2D-1.0E+0.5L+0.7S	-17.7	46	0.90	-15.9	41	-13.9	36	0.87
1.2D-1.0E	-17.7	46	0.90	-15.9	41	-13.9	36	0.87

Strength Checks - Shears:

	V_n kips	ϕV_n kips	ϕ	V_u kips	Stress Ratio
1.4D	28.3	22.7	0.80	4.9	0.22
1.2D+1.6L+0.5Lr	28.3	22.7	0.80	4.9	0.22
1.2D+1.6Lr+0.5L	28.3	22.7	0.80	6.4	0.28
1.2D+1.6Lr+0.5W	28.3	22.7	0.80	6.4	0.28
1.2D+1.0W+0.5L+0.5Lr	28.3	22.7	0.80	4.9	0.22

BUEHLER

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Lintel Design : Lintel 2

$L = 18.5'$

$M_{DL} = 25psf(7')(18.5)^2/8$

$= 7.5$ k-ft

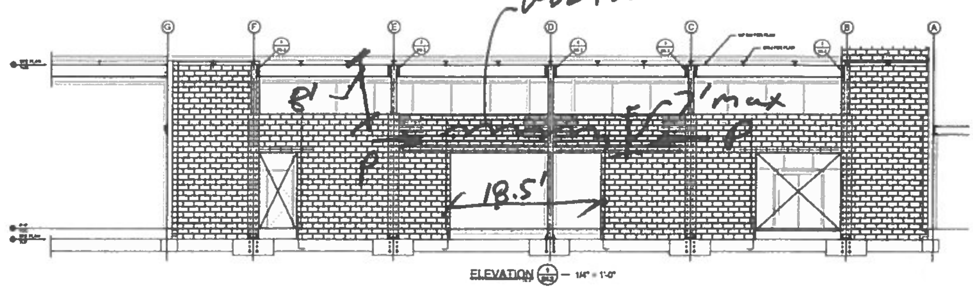
$+ 15psf(8')(18.5)^2/8$

$+ 100psf(7')(18.5)^2/8$

$= 8.2$ k-ft

15.7 k-ft

$V = 9^k$



Trib width from Low roof $= 14/2 = 7'$
No trib from High roof. Supports window

$M_{rr} = 6$k-ft
$V_{rr} = 1.3$k-ft

$P_{seismic} = 6.3^k$
collector load.

$M_{seismic} = 48psf(7')(18.5)^2/8 = 14.4$ k-ft.

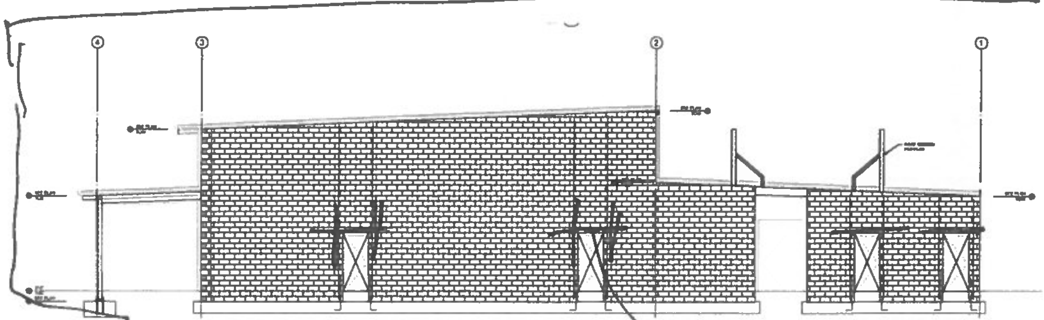
$V_{seismic} = 3.1^k$

$M/S = \dfrac{172.8\text{k-in}}{7' \times 12" \times 7.625^2/6} =$

$= 212psi(1.3)$

$= 275$ ~~287~~ psi

(lintel ok for o-o-p higher w/ reinf.

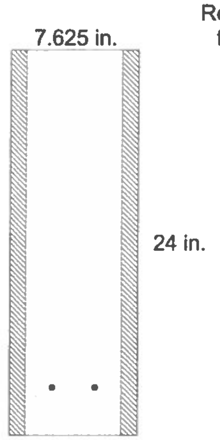


	Project:	C7
	Number:	
	Date:	December 02, 2019
T: Lintel 2	By:	
F:	Sheet	of

CMD15.00.00 Filename: lintel 2.dat

Reinforced Concrete Masonry Beam per 2016 CBC Section 2108 (2013 MSJC Section 9.3.4.2)
- Ultimate Strength Design -

Input:



Reinforcing dimensioned from bottom of beam:

2 # 5 @ 3 inches

Shear Steel:
0 # 3 @ 8 in. o.c.

Beam Section

Beam Geometry:
19.00' Long, 24.00" deep CMU beam
8" nominal thickness

Beam Material Properties:

f'_m = 2,000 psi

f_y = 60,000 psi

E_m = 1,800 ksi

E_s = 29,000 ksi

n = 16.11

Applied Loads:	P kips	V kips	M kip-ft
Dead Load	0.0	9.00	15.70
Floor Live Load	0.0	0.00	0.00
Roof Live Load	0.0	1.30	6.00
Snow Load	0.0	0.00	0.00
Horizontal Seismic Load	6.3	0.00	0.00
Vertical Seismic Load	0.0	0.00	0.00
Wind Load	0.0	0.00	0.00

Collector load, see lateral design

Output:

P vs M Diagram Points:

		Positive Moments			Negative Moments		
		P_n, M_n	ϕP_n, ϕM_n	ϕ	P_n, M_n	ϕP_n, ϕM_n	ϕ
Maximum Axial Load (kips)	P_{max} =	329	237	0.90	329	237	0.90
Moment at P_{max} (kip-ft)	$M(P_{max})$ =	0	26	0.90	0	-26	0.90
Balanced Axial Load (kips)	P_b =	71	64	0.90	-20	-18	0.90
Balanced Moment (kip-ft)	M_b =	97	87	0.90	13	11	0.90
Moment at P=0 (kip-ft)	M_o =	60	54	0.90	-4	-3	0.90

General Calculations:

Total A_s = 0.62 in², ρ = 0.0039 $\leq$ 0.0092 = ρ_{max} - OK
Cracking Moment: M_{cr} = 16.3 kip-ft = $S_g * f_r$ = (732 in³)(0.267 ksi)/12
Minimum Nominal Flexural Strength of the Beam = 43.6 kip-ft $\geq$ 1.3*M_{cr} OK

Strength Checks - Moments:

	P_n kips	M_n kip-ft	ϕ	ϕP_n kips	ϕM_n kip-ft	P_u kips	M_u kip-ft	Stress Ratio
1.4D	0.0	60	0.90	0.0	54	0.0	22	0.40
1.2D+1.6Lr+0.5L	0.0	60	0.90	0.0	54	0.0	28	0.52
1.2D+1.6Lr+0.5W	0.0	60	0.90	0.0	54	0.0	28	0.52
1.2D-1.0E+0.5L+0.7S	-15.7	47	0.90	-14.1	42	-6.3	19	0.45
1.2D-1.0E	-15.7	47	0.90	-14.1	42	-6.3	19	0.45

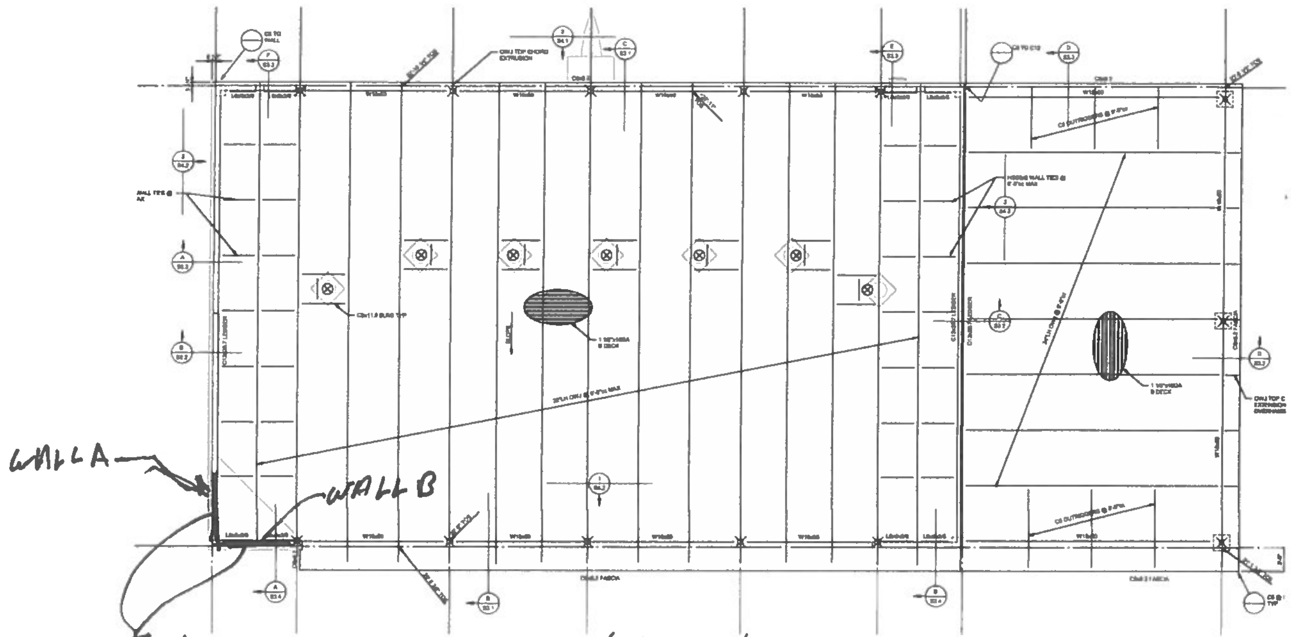
Strength Checks - Shears:

	V_n kips	ϕV_n kips	ϕ	V_u kips	Stress Ratio
1.4D	30.1	24.1	0.80	12.6	0.52
1.2D+1.6L+0.5Lr	29.0	23.2	0.80	11.5	0.49
1.2D+1.6Lr+0.5L	28.3	22.7	0.80	12.9	0.57
1.2D+1.6Lr+0.5W	28.3	22.7	0.80	12.9	0.57
1.2D+1.0W+0.5L+0.5Lr	29.0	23.2	0.80	11.5	0.49

BUEHLER

PROJECT	DMV Delano		
PROJECT NO.	2018-0187	DATE	11/11/2019
CLIENT	NLA	BY GB	PAGE NO. C8

WALL OUT OF PLANE DESIGN



THESE TWO WALLS GOVERN DESIGN

- WALL A : TALLEST & least amount of Axial load
- WALL B : TALLEST w/ most Axial load
  : Intermediate roof that braces wall
  envelope design by checking Unbraced & braced.

8" THICKNESS TYP, USE same HORIZ & vert reinf. everywhere

LOADS: WALL A – Roof DL : $25psf(7/2) = 88plf$

Roo Lr $= 20(7/2) = 70plf$

$L = \pm 21'$ 6" eccentricity max

WALL B – $25psf(7')(58'/2) = 5^K$: DL

$20psf(7')(58'/2) = 4.1^K$: Lr

$F_p = 0.4(0.6)w_p = 24psf$

w_p = 100psf

C9

BUEHLER

PROJECT	DMV				
PROJECT NO.	2018-0187			DATE	12/2/2019
CLIENT	NLA	BY	GB	SHEET NO.	

WALL B - lower brace height

Masonry Slender Wall Design- Typical Wall Spanning 8ft Horiztonally — TMS 402-16 Section 9.3.5

Note: Wall analysis is based on an effective vertical strip of the user's choice. Unfactored design vertical and lateral loads are calculated by hand and input by the user.

Wall Properties

Nominal Wall Thickness:		8.000 in
Actual Wall Thickness, t:		7.625 in
Wall Segment:		Wall Pier
Vertical Bar Layout:		Centered
Horizontal Bar (or tie) Size:		#4
Vert reinf. (wall pier):	6 -	#5
~~Vert reinf. (1' strip):~~		#5 ~~@ 16 in cc~~
Effective $A_{s,vert}$:		1.705 sq in
Distance, d:		3.81 in
Unbraced Wall Height, h:		13.00 ft
h / t Ratio:		20
Vert Load Ecc. (from face of wall), w:		3.000 in
Eccentricity $e = t_s/2 + w$ =		6.81 in
Effective leg width, b_{eff}:	11ft	132 in
$S = b_{eff} \times t^2 / 6$ =		1279 in^3
$I_g = b_{eff} \times t^3 / 12$ =		4877 in^4
A_g:		1007 sq in

Material Properties

Masonry Strength, f'_m =	2000 psi
Masonry Modulus, E_m =	1800 ksi
Modulus of Rupture, f_r =	163 psi
(f_r per Table 9.1.9.2)	
Steel Strength, f_y =	60000 psi
Steel Modulus, E_s =	29000 ksi
Modulus Ratio $n = E_s/E_m$ =	16.1

Misc. Coefficients & Properties

S_{ds} =	0.6
ϕ =	0.90
β_1=	0.80
$M_{cr} = S \times f_r$ =	208 in-kips
$\Delta_{cr} = 5 \times M_{cr} \times h^2 / (48 \times E_m \times I_g)$ =	0.060 in

Note: Engineer must establish loads based on effective leg width.

Input loads are unfactored

Design Vert. Loads (at mid-height of panel leg)

Roof DL	5.00 kips
Roof LL_r	4.10 kips
Floor DL	0.00 kips
Floor LL	0.00 kips
Wall DL	25.30 kips

Design Uniform Lateral Loads

Seismic Uniform Load w_{seis}	264plf	1.0E
Wind Uniform Load w_{wind}	330plf	1.0W

Check TMS 402-16 Code Requirements

Axial Stress (9.3.5.4.2)

$P_{u\,max} = P_{uw} + P_{uf}$ = 42.92 kips — Compression limit for h / t ≤ 30 = 0.20 x f'_m = 0.400 ksi — Eq (9-22) — **Compression OK 11%**

$P_{u\,max} / A_g$ = 0.043 ksi — Compression limit for h / t > 30 = 0.05 x f'_m = 0.100 ksi

Maximum Reinforcement (9.3.3.2)

ρ = 0.0034 — ρ_{max} = 0.0085 — **Max Reinf. OK 40%**

Load Combinations (ASCE 7-16)

			U_{dead}	U_{live}	$U_{RF\,live}$	$U_{lateral}$
Seismic 1	$(1.2+0.2S_{DS})D+f_1L+1.0E$	2.3.6 (6)	1.32	0.50	0.00	1.00
Seismic 2	$(0.9-0.2S_{DS})D+1.0E$	2.3.6 (7)	0.78	0.00	0.00	1.00
Wind 1	$1.2D+1.6L_r+0.5W$	2.3.1 (3)	1.20	0.00	1.60	0.50
Wind 2	$1.2D+1.0W+f_1L+0.5L_r$	2.3.1 (4)	1.20	0.50	0.50	1.00
Wind 3	$0.9D+1.0W$	2.3.1 (5)	0.90	0.00	0.00	1.00

Use $U_{RF\,live}$ =0.2 for Snow

U_{live} reduced per exception ASCE 2.3.1

Use U_{live} = 1.0 at garages, public assembly or if LL > 100 psf

ULTIMATE STRENGTH CHECK

	ULTIMATE LOAD CASES					
	Seismic 1	**Seismic 2**	**Wind 1**	**Wind 2**	**Wind 3**	
Lateral Loads- out of plane						
$w_{u\,seis} = U_{lateral} \times w_{seis}$	264 plf	264 plf	-	-	-	
$w_{u\,wind} = U_{lateral} \times w_{wind}$	-	-	165 plf	330 plf	330 plf	
Vertical Roof & Floor Loads						
$P_f = (U_{dl} \times \Sigma DL + U_{ll} \times \Sigma LL)$	6.60 kips	3.90 kips	12.56 kips	6.00 kips	4.50 kips	
Panel Weight						
$P_w = U_{dl} \times$ Panel DL	33.40 kips	19.73 kips	30.36 kips	30.36 kips	22.77 kips	
$P_u = P_w + P_f$ =	40.00 kips	23.63 kips	42.92 kips	36.36 kips	27.27 kips	Eq (9-24)
Nominal Moment Capacity (TMS 9.3.5.2)						
$A_{se} = (A_s f_y + P_u / \phi) / f_y$ =	2.45	2.14	2.50	2.38	2.21	
$a = (P_u / \phi + A_s f_y) / (0.8 f'_m b_{eff})$ =	0.69 in	0.61 in	0.71 in	0.68 in	0.63 in	Section 9.3.5.2
$c = a / \beta_1$	0.87 in	0.76 in	0.89 in	0.84 in	0.78 in	
$I_{cr} = n (A_s + (P_u{*}t_{sp})/(f_y{*}2d)){*}(d-c)^2 + b \times c^3/3$ =	360.0 in^4	334.3 in^4	364.4 in^4	354.5 in^4	340.2 in^4	Eq (9-30)
$M_{cr} = S \times (f_r + P_u/A_g)$=	259 in-kips	239 in-kips	263 in-kips	255 in-kips	243 in-kips	Section 9.3.5.2
$\phi M_n = \phi(P_u / \phi + A_s f_y)(d - a/2)$ =	**508 in-kips**	**451 in-kips**	**519 in-kips**	**496 in-kips**	**464 in-kips**	
$M_n > 1.3\, M_{cr}$ Check	**OK**	**OK**	**OK**	**OK**	**OK**	**Mn>1.3*Mcr - OK**
Deflection from Factored Loads						
If $M_u \geq M_{cr}$ $\Delta_u = \Delta_{cr} + (5x(M_u - M_{cr})xh^2)/(48xE_m xI_{cr})$ =	-	-	-	-	-	Eq (9-26)
If $M_u < M_{cr}$ $\Delta_u = 5 * M_u * h^2 / (48 * E_m * I_g)$ =	0.03 in	0.02 in	0.02 in	0.03 in	0.03 in	Eq (9-25)
Moment at Mid-height of Wall (TMS 9.3.5.4.2)						
$M_u = q_u \times h^2/8 + P_{uf} \times e/2 + P_u \times \Delta_u$ =	**90 in-kips**	**81 in-kips**	**86 in-kips**	**105 in-kips**	**100 in-kips**	Eq (9-23)
Demand / Capacity Ratio =	**18%**	**18%**	**17%**	**21%**	**22%**	**Strength OK 22%**

SERVICE LOAD DEFLECTION CHECK

TMS 402-16 9.3.5.5

		SERVICE LOAD CASES		
		Seismic	**Wind**	
Allowable $\Delta_s = 0.007 \times h$ = 1.09 in	Eq (9-32)			
$P_{s1} = \Sigma DL$ @ $(1.0+0.14{*}Sds) + \Sigma LL$ =		5.42 kips	5.00 kips	$P_{s1} = \Sigma DL + \Sigma LL$ =
P_{s2} = Panel DL @ $(1.0+0.14{*}S_{ds})$ =		27.43 kips	25.30 kips	P_{s2} = Panel DL =
$P_s = P_{s1} + P_{s2}$ =		32.85 kips	30.30 kips	$P_s = P_{s1} + P_{s2}$ =
$w_{ser} = 0.7{*}w_{seis}$ (or 0.6*w for wind)		185 plf	198 plf	
$I_{cr} = n (A_{se} + (P_u{*}t_{sp})/(f_y{*}2d)){*}(d-c)^2 + b \times c^3/3$ =	Eq (9-30)	334 in^4	340 in^4	
$M_{ser} = (w_{ser} \times h^2 \times 12)/8 + P_{s1} \times e/2 + (P_s) \times \Delta s$ =	Mser <= Mcr	66 in-kips	68 in-kips	Mser <= Mcr
If $M_{ser} < M_{cr}$ $\Delta_s = 5 * M_{ser} * L_u^2 / (48 * E_c * I_g)$ =	Eq (9-25)	0.02 in	0.02 in	
If $M_{ser} \geq M_{cr}$ $\Delta_s = \Delta_{cr} + (5x(M_{ser} - M_{cr})xh^2)/(48xE_m xI_{cr})$ =	Eq (9-26)	-	-	
Deflection/Allowable Ratio		**2%**	**2%**	**Deflection OK 2%**

BUEHLER

PROJECT	DMV				
PROJECT NO	2018-0187			DATE	12/2/2019
CLIENT	NLA	BY	GB	SHEET NO	C10

WALL B

Masonry Slender Wall Design- Typical Wall Spanning 8ft Horiztonally TMS 402-16 Section 9.3.5

Note: Wall analysis is based on an effective vertical strip of the user's choice. Unfactored design vertical and lateral loads are calculated by hand and input by the user.

Wall Properties

Nominal Wall Thickness:		8.000 in
Actual Wall Thickness, t:		7.625 in
Wall Segment:		Wall Pier
Vertical Bar Layout:		Centered
Horizontal Bar (or tie) Size:		#4
Vert reinf. (wall pier):	6 -	#5
~~Vert reinf. (1' strip):~~		#5 ~~@ 16 in cc~~
Effective $A_{s,\,vert}$:		1.705 sq in
Distance, d:		3.81 in
Unbraced Wall Height, h:		22.00 ft
h / t Ratio:		35
Vert Load Ecc. (from face of wall), w:		3.000 in
Eccentricity $e = t_a/2+w$ =		6.81 in
Effective leg width, b_{eff}:	11ft	132 in
$S = b_{eff} \times t^2 / 6$ =		1279 in^3
$I_g = b_{eff} \times t^3 / 12$ =		4877 in^4
A_g:		1007 sq in

Material Properties

Masonry Strength, f'_m =	2000 psi
Masonry Modulus, E_m =	1800 ksi
Modulus of Rupture, f_r =	163 psi
(f_r per Table 9.1.9.2)	
Steel Strength, f_y =	60000 psi
Steel Modulus, E_s =	29000 ksi
Modulus Ratio $n = E_s/E_m$ =	16.1

Misc. Coefficients & Properties

S_{ds} =	0.6
ϕ =	0.90
β_1=	0.80
$M_{cr} = S \times f_r$ =	208 in-kips
$\Delta_{cr} = 5 \times M_{cr} \times h^2 / (48 \times E_m \times I_g)$ =	0.172 in

Note: Engineer must establish loads based on effective leg width.

Input loads are unfactored

Design Vert. Loads (at mid-height of panel leg)

Roof DL	5.00 kips
Roof LL_r	4.10 kips
Floor DL	0.00 kips
Floor LL	0.00 kips
Wall DL	25.30 kips

Design Uniform Lateral Loads

Seismic Uniform Load W_{seis}	264plf	1.0E
Wind Uniform Load W_{wind}	330plf	1.0W

Check TMS 402-16 Code Requirements

Axial Stress (9.3.5.4.2)

$P_{u\,max} = P_{uw} + P_{uf}$ = 42.92 kips

$P_{u\,max} / A_g$ = 0.043 ksi

Compression limit for $h / t \le 30 = 0.20 \times f'_m$ = 0.400 ksi Eq (9-22)

Compression limit for $h / t > 30 = 0.05 \times f'_m$ = 0.100 ksi

Compression OK	43%

Maximum Reinforcement (9.3.3.2)

ρ = 0.0034 ρ_{max} = 0.0085

Max Reinf. OK	40%

Load Combinations (ASCE 7-16)

			U_{dead}	U_{live}	$U_{RF\,live}$	$U_{lateral}$
Seismic 1	$(1.2+0.2S_{DS})D+f_1L+1.0E$	2.3.6 (6)	1.32	0.50	0.00	1.00
Seismic 2	$(0.9-0.2S_{DS})D+1.0E$	2.3.6 (7)	0.78	0.00	0.00	1.00
Wind 1	$1.2D+1.6L_r+0.5W$	2.3.1 (3)	1.20	0.00	1.60	0.50
Wind 2	$1.2D+1.0W+f_1L+0.5L_r$	2.3.1 (4)	1.20	0.50	0.50	1.00
Wind 3	$0.9D+1.0W$	2.3.1 (5)	0.90	0.00	0.00	1.00

Use $U_{RF\,live}$ =0.2 for Snow

U_{live} reduced per exception ASCE 2.3.1

Use U_{live} = 1.0 at garages, public assembly or if LL > 100 psf

ULTIMATE STRENGTH CHECK

	ULTIMATE LOAD CASES					
	Seismic 1	**Seismic 2**	**Wind 1**	**Wind 2**	**Wind 3**	
Lateral Loads- out of plane						
$w_{u\,seis} = U_{lateral} \times w_{seis}$	264 plf	264 plf	-	-	-	
$w_{u\,wind} = U_{lateral} \times w_{wind}$	-	-	165 plf	330 plf	330 plf	
Vertical Roof & Floor Loads						
$P_f = (U_d \times \Sigma DL + U_l \times \Sigma LL)$	6.60 kips	3.90 kips	12.56 kips	6.00 kips	4.50 kips	
Panel Weight						
$P_w = U_d \times$ Panel DL	33.40 kips	19.73 kips	30.36 kips	30.36 kips	22.77 kips	
$P_u = P_w + P_f$ =	40.00 kips	23.63 kips	42.92 kips	36.36 kips	27.27 kips	Eq (9-24)
Nominal Moment Capacity (TMS 9.3.5.2)						
$A_{se} = (A_s f_y + P_u / \phi) / f_y$ =	2.45	2.14	2.50	2.38	2.21	
$a = (P_u / \phi + A_s f_y) / (0.8 f'_m b_{eff})$ =	0.69 in	0.61 in	0.71 in	0.68 in	0.63 in	Section 9.3.5.2
$c = a / \beta_1$	0.87 in	0.76 in	0.89 in	0.84 in	0.78 in	
$I_{cr} = n (A_s + (P_u{*}t_{sp})/(f_y{*}2d)){*}(d-c)^2 + b \times c^3/3$ =	360.0 in^4	334.3 in^4	364.4 in^4	354.5 in^4	340.2 in^4	Eq (9-30)
$M_{cr} = S \times (f_r + P_u/A_g)$=	259 in-kips	239 in-kips	263 in-kips	255 in-kips	243 in-kips	Section 9.3.5.2
$\phi M_n = \phi(P_u / \phi + A_s f_y)(d - a/2)$ =	**508 in-kips**	**451 in-kips**	**519 in-kips**	**496 in-kips**	**464 in-kips**	
$M_n > 1.3\, M_{cr}$ Check	**OK**	**OK**	**OK**	**OK**	**OK**	**Mn>1.3*Mcr - OK**
Deflection from Factored Loads						
If $M_u >= M_{cr}$ $\Delta_u = \Delta_{cr} + (5x(M_u-M_{cr})xh^2)/(48xE_m x I_{cr})$ =	0.18 in	0.17 in	-	0.46 in	0.50 in	Eq (9-26)
If $M_u < M_{cr}$ $\Delta_u = 5 * M_u * h^2 / (48 * E_m * I_g)$ =	-	-	0.14 in	-	-	Eq (9-25)
Moment at Mid-height of Wall (TMS 9.3.5.4.2)						
$M_u = q_u \times h^2/8 + P_{uf} \times e/2 + P_u \times \Delta_u$ =	**221 in-kips**	**209 in-kips**	**169 in-kips**	**277 in-kips**	**269 in-kips**	Eq (9-23)
Demand / Capacity Ratio =	**44%**	**46%**	**33%**	**56%**	**58%**	**Strength OK 58%**

SERVICE LOAD DEFLECTION CHECK

TMS 402-16 9.3.5.5

		SERVICE LOAD CASES		
		Seismic	**Wind**	
Allowable $\Delta_s = 0.007 \times h$ = 1.85 in	Eq (9-32)			
$P_{s1} = \Sigma DL$ @ $(1.0+0.14{*}Sds) + \Sigma LL$ =		5.42 kips	5.00 kips	$P_{s1} = \Sigma DL + \Sigma LL$ =
P_{s2} = Panel DL @ $(1.0+0.14{*}S_{ds})$ =		27.43 kips	25.30 kips	P_{s2} = Panel DL =
$P_s = P_{s1} + P_{s2}$ =		32.85 kips	30.30 kips	$P_s = P_{s1} + P_{s2}$ =
$w_{ser} = 0.7{*}w_{seis}$ (or 0.6*w for wind)		185 plf	198 plf	
$I_{cr} = n (A_{se} + (P_u{*}t_{sp})/(f_y{*}2d)){*}(d-c)^2 + b \times c^3/3$ =	Eq (9-30)	334 in^4	340 in^4	
$M_{ser} = (w_{ser} \times h^2 \times 12)/8 + P_{s1} \times e/2 + (P_s) \times \Delta s$ =	Mser <= Mcr	157 in-kips	165 in-kips	Mser <= Mcr
If $M_{ser} < M_{cr}$ $\Delta_s = 5 * M_{ser} * L_u^2 / (48 * E_c * I_g)$ =	Eq (9-25)	0.13 in	0.14 in	
If $M_{ser} >= M_{cr}$ $\Delta_s = \Delta_{cr} + (5x(M_{ser}-M_{cr})xh^2)/(48xE_m x I_{cr})$ =	Eq (9-26)	-	-	
Deflection/Allowable Ratio		**7%**	**7%**	**Deflection OK 7%**

BUEHLER

PROJECT	DMV		
PROJECT NO.	2018-0187	DATE	12/2/2019
CLIENT	NLA	BY GB	SHEET NO. C11

WALL A

Masonry Slender Wall Design- Typical Wall Spanning 8ft Horiztonally — TMS 402-16 Section 9.3.5

Note: Wall analysis is based on an effective vertical strip of the user's choice. Unfactored design vertical and lateral loads are calculated by hand and input by the user.

Wall Properties

Nominal Wall Thickness:	8.000 in
Actual Wall Thickness, t:	7.625 in
Wall Segment:	Wall Pier
Vertical Bar Layout:	Centered
Horizontal Bar (or tie) Size:	#4
Vert reinf. (wall pier):	4 - #5
~~Vert reinf. (1' strip):~~	#5 ~~@ 16 in cc~~
Effective $A_{s,vert}$:	1.240 sq in
Distance, d:	3.81 in
Unbraced Wall Height, h:	21.00 ft
h / t Ratio:	33
Vert Load Ecc. (from face of wall), w:	3.000 in
Eccentricity $e = t_s/2 + w$ =	6.81 in
Effective leg width, b_{eff}:	96 in
$S = b_{eff} \times t^2 / 6$ =	930 in^3
$I_g = b_{eff} \times t^3 / 12$ =	3547 in^4
A_g:	732 sq in

Material Properties

Masonry Strength, f'_m =	2000 psi
Masonry Modulus, E_m =	1800 ksi
Modulus of Rupture, f_r =	163 psi
(f_r per Table 9.1.9.2)	
Steel Strength, f_y =	60000 psi
Steel Modulus, E_s =	29000 ksi
Modulus Ratio $n = E_s/E_m$ =	16.1

Misc. Coefficients & Properties

S_{ds} =	0.6
ϕ =	0.90
β_1=	0.80
$M_{cr} = S \times f_r$ =	152 in-kips
$\Delta_{cr} = 5 \times M_{cr} \times h^2 / (48 \times E_m \times I_g)$ =	0.157 in

Note: Engineer must establish loads based on effective leg width.

Input loads are unfactored

Design Vert. Loads (at mid-height of panel leg)

Roof DL	0.64 kips
Roof LL_r	0.56 kips
Floor DL	0.00 kips
Floor LL	0.00 kips
Wall DL	18.40 kips

Design Uniform Lateral Loads

Seismic Uniform Load w_{seis}	192plf	1.0E
Wind Uniform Load w_{wind}	240plf	1.0W

Check TMS 402-16 Code Requirements

Axial Stress (9.3.5.4.2)

$P_{u\,max} = P_{uw} + P_{uf}$ = 25.13 kips — Compression limit for h / t ≤ 30 = 0.20 x f'_m = 0.400 ksi — Eq (9-22) — **Compression OK 34%**

$P_{u\,max} / A_g$ = 0.034 ksi — Compression limit for h / t > 30 = 0.05 x f'_m = 0.100 ksi

Maximum Reinforcement (9.3.3.2)

ρ = 0.0034 — ρ_{max} = 0.0087 — **Max Reinf. OK 39%**

Load Combinations (ASCE 7-16)

			U_{dead}	U_{live}	$U_{RF\,live}$	$U_{lateral}$	
Seismic 1	$(1.2+0.2S_{DS})D + f_1L + 1.0E$	2.3.6 (6)	1.32	0.50	0.00	1.00	Use $U_{RF\,live}$ =0.2 for Snow
Seismic 2	$(0.9-0.2S_{DS})D + 1.0E$	2.3.6 (7)	0.78	0.00	0.00	1.00	
Wind 1	$1.2D+1.6L_r+0.5W$	2.3.1 (3)	1.20	0.00	1.60	0.50	U_{live} reduced per exception ASCE 2.3.1
Wind 2	$1.2D+1.0W+f_1L+0.5L_r$	2.3.1 (4)	1.20	0.50	0.50	1.00	Use U_{live} = 1.0 at garages, public
Wind 3	$0.9D+1.0W$	2.3.1 (5)	0.90	0.00	0.00	1.00	assembly or if LL > 100 psf

ULTIMATE STRENGTH CHECK

ULTIMATE LOAD CASES	Seismic 1	Seismic 2	Wind 1	Wind 2	Wind 3	
Lateral Loads- out of plane						
$w_{u\,seis} = U_{lateral} \times w_{seis}$	192 plf	192 plf	-	-	-	
$w_{u\,wind} = U_{lateral} \times w_{wind}$	-	-	120 plf	240 plf	240 plf	
Vertical Roof & Floor Loads						
$P_f = (U_{dl} \times \Sigma DL + U_{ll} \times \Sigma LL)$	0.84 kips	0.50 kips	1.66 kips	0.77 kips	0.58 kips	
Panel Weight						
$P_w = U_{dl} \times$ Panel DL	24.29 kips	14.35 kips	22.08 kips	22.08 kips	16.56 kips	
$P_u = P_w + P_f$ =	25.13 kips	14.85 kips	23.74 kips	22.85 kips	17.14 kips	Eq (9-24)
Nominal Moment Capacity (TMS 9.3.5.2)						
$A_{se} = (A_s f_y + P_u / \phi) / f_y$ =	1.71	1.52	1.68	1.66	1.56	
$a = (P_u / \phi + A_s f_y) / (0.8 f'_m b_{eff})$ =	0.67 in	0.59 in	0.66 in	0.65 in	0.61 in	Section 9.3.5.2
$c = a / \beta_1$	0.83 in	0.74 in	0.82 in	0.81 in	0.76 in	
$I_{cr} = n (A_s + (P_u{*}t_{sp})/(f_y{*}2d)){*}(d-c)^2 + b \times c^3/3$ =	255.8 in^4	239.2 in^4	253.6 in^4	252.2 in^4	243.0 in^4	Eq (9-30)
$M_{cr} = S \times (f_r + P_u/A_g)$=	184 in-kips	171 in-kips	182 in-kips	181 in-kips	173 in-kips	Section 9.3.5.2
$\phi M_n = \phi(P_u / \phi + A_s f_y)(d - a/2)$ =	**356 in-kips**	**320 in-kips**	**351 in-kips**	**348 in-kips**	**328 in-kips**	
$M_n > 1.3\,M_{cr}$ Check	**OK**	**OK**	**OK**	**OK**	**OK**	**Mn>1.3*Mcr - OK**
Deflection from Factored Loads						
If $M_u >= M_{cr}$ $\Delta_u = \Delta_{cr} + (5x(M_u - M_{cr})xh^2)/(48xE_m xI_{cr})$ =	-	-	-	0.17 in	0.17 in	Eq (9-26)
If $M_u < M_{cr}$ $\Delta_u = 5 * M_u * h^2 / (48 * E_m * I_g)$ =	0.14 in	0.14 in	0.09 in	-	-	Eq (9-25)
Moment at Mid-height of Wall (TMS 9.3.5.4.2)						
$M_u = q_u \times h^2/8 + P_{uf} \times e/2 + P_u \times \Delta_u$ =	**133 in-kips**	**131 in-kips**	**87 in-kips**	**165 in-kips**	**164 in-kips**	Eq (9-23)
Demand / Capacity Ratio =	**37%**	**41%**	**25%**	**47%**	**50%**	**Strength OK 50%**

SERVICE LOAD DEFLECTION CHECK

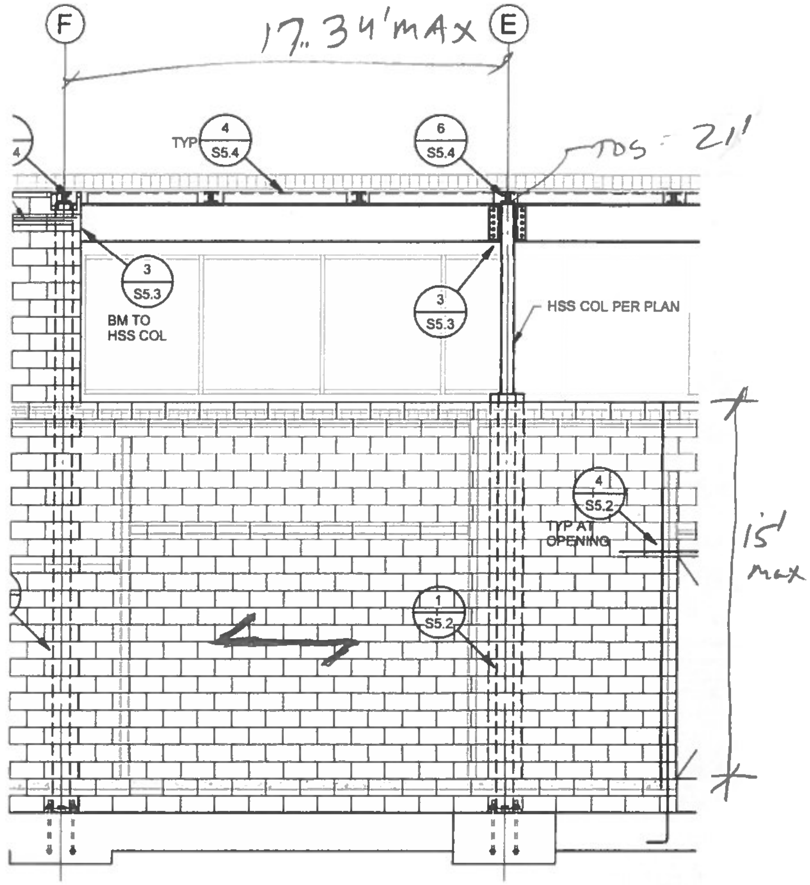
TMS 402-16 9.3.5.5

SERVICE LOAD CASES		Seismic	Wind	
Allowable $\Delta_s = 0.007 \times h$ = 1.76 in	Eq (9-32)			
$P_{s1} = \Sigma DL$ @ $(1.0+0.14{*}Sds) + \Sigma LL$ =		0.69 kips	0.64 kips	$P_{s1} = \Sigma DL + \Sigma LL$ =
P_{s2} = Panel DL @ $(1.0+0.14{*}S_{ds})$ =		19.95 kips	18.40 kips	P_{s2} = Panel DL =
$P_s = P_{s1} + P_{s2}$ =		20.64 kips	19.04 kips	$P_s = P_{s1} + P_{s2}$ =
$w_{ser} = 0.7{*}w_{seis}$ (or 0.6*w for wind)		134 plf	144 plf	
$I_{cr} = n (A_{se} + (P_u{*}t_{sp})/(f_y{*}2d)){*}(d-c)^2 + b \times c^3/3$ =	Eq (9-30)	239 in^4	243 in^4	
$M_{ser} = (w_{ser} \times h^2 \times 12)/8 + P_{s1} \times e/2 + (P_s) \times \Delta s$ =	Mser <= Mcr	93 in-kips	99 in-kips	Mser <= Mcr
If $M_{ser} < M_{cr}$ $\Delta_s = 5 * M_{ser} * L_u^2 / (48 * E_c * I_g)$ =	Eq (9-25)	0.10 in	0.10 in	
If $M_{ser} >= M_{cr}$ $\Delta_s = \Delta_{cr} + (5x(M_{ser} - M_{cr})xh^2)/(48xE_m xI_{cr})$ =	Eq (9-26)	-	-	
Deflection/Allowable Ratio		**5%**	**6%**	**Deflection OK 6%**

CMU WALL O-O-P SUPPORT @ CLEARSTORY WINDOWS

$F_p = .4(0.6)(k_a = 2)(I = 1.0)(W_p = 80psf)$
$= 38.4 psf$

- Conservatively assume wall spans horizontally, see spreadsheet for wall design



ELEVATION 2/S4.1

$P_{DL} = 14^k$ (unfactored)

$\frac{17.34'}{2}(38.4)$
$= 333 plf$

HSS COL

Add wt. of pilaster: $38.4psf(2' \times 2') = 154 plf$ (full ht.)

$$M = 154plf(21')^2/8 + \left(\frac{333plf(15')}{2(21')}(2(21') - 15')\right)^2 / 2(333plf)$$

$= 25.2$ k-ft.

HSS 6x6x1/2 $\quad \phi M_n = 38.7$ k-ft.

$\phi P_n = 100$ kips

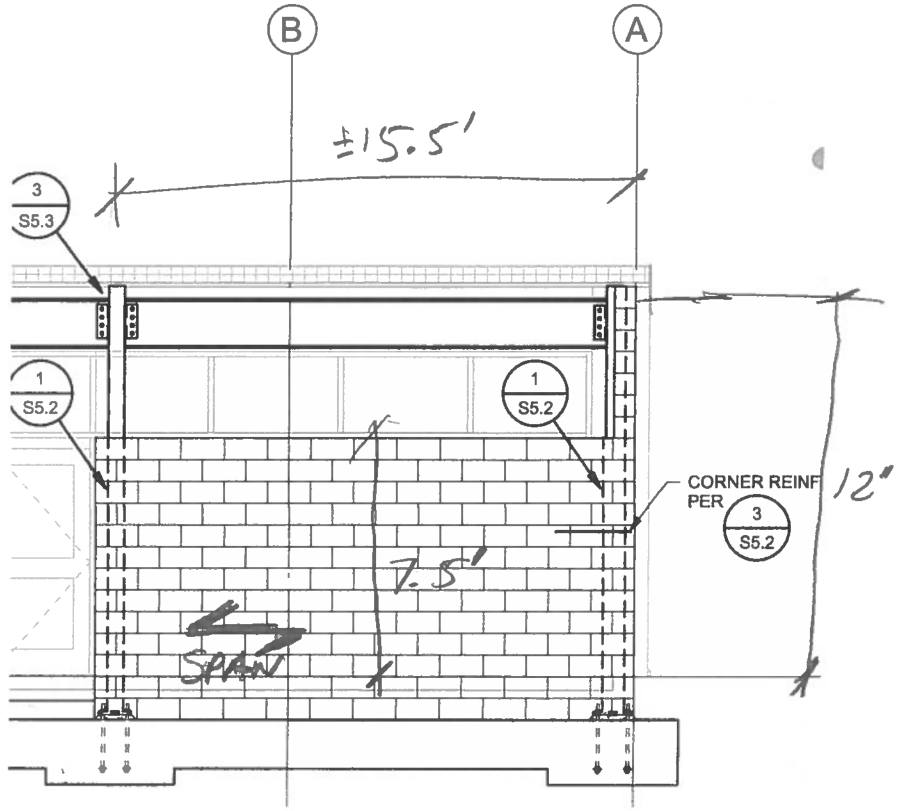
$P_u = 1.2(14^k) = 16.8^k$

$P_r/P_c < 0.2$

$$\frac{16.8^k}{2(100^k)} + \frac{25.2}{38.7} = 0.74 < 1.0$$

HSS 6x6x1/4" Cols OK

CMU WALL O-O-P Support @ Clear Story Windows



$F_p = 38.4\text{ psf}$

$P_w = 20^k$ (conservative)

$w = 38.4\text{ psf}(7.5'/2)$

$= 144\text{ plf}$

$$M = \frac{\left(\frac{144(12')}{2(12')}\left(2(12')-7.5\right)\right)^2}{2(144\text{ plf})}$$

$\approx 5\text{ k-ft}$

HSS 6x6x1/4 Col $\phi M_n = 25.2\text{ k-ft}$

Col ok by inspection

PROJECT	DMV				
PROJECT NO.	2018-0187			DATE	12/2/2019
CLIENT	NLA	BY	GB	SHEET NO.	C14

BUEHLER

Masonry Slender Wall Design- Wall at Clearstory Spanning 8ft Horiztonally — TMS 402-16 Section 9.3.5

Note: Wall analysis is based on an effective vertical strip of the user's choice. Unfactored design vertical and lateral loads are calculated by hand and input by the user.

Wall Properties

Nominal Wall Thickness:	8.000 in
Actual Wall Thickness, t:	7.625 in
Wall Segment:	1' Strip
Vertical Bar Layout:	Centered
Horizontal Bar (or tie) Size:	#4
~~Vert reinf. (wall pier):~~	~~1~~ #5
Vert reinf. (1' strip):	#4 @ 24 in cc
Effective $A_{s,vert}$:	0.100 sq in
Distance, d:	3.81 in
Unbraced Wall Height, h:	17.34 ft
h / t Ratio:	27
Vert Load Ecc. (from face of wall), w:	3.000 in
Eccentricity $e = t_s/2 + w$ =	6.81 in
Effective leg width, b_{eff}:	12 in
$S = b_{eff} \times t^2 / 6$ =	116 in^3
$I_g = b_{eff} \times t^3 / 12$ =	443 in^4
A_g:	92 sq in

Material Properties

Masonry Strength, f'_m =	2000 psi
Masonry Modulus, E_m =	1800 ksi
Modulus of Rupture, f_r =	163 psi
(f_r per Table 9.1.9.2)	
Steel Strength, f_y =	60000 psi
Steel Modulus, E_s =	29000 ksi
Modulus Ratio $n = E_s/E_m$ =	16.1

Misc. Coefficients & Properties

S_{ds} =	0.6
ϕ =	0.90
β_1 =	0.80
$M_{cr} = S \times f_r$ =	19 in-kips
$\Delta_{cr} = 5 \times M_{cr} \times h^2 / (48 \times E_m \times I_g)$ =	0.107 in

Note: Engineer must establish loads based on effective leg width.

Input loads are unfactored

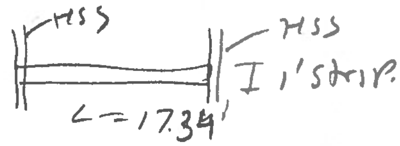
Design Vert. Loads (at mid-height of panel leg)

Roof DL	0.00 kips
Roof LL_r	0.00 kips
Floor DL	0.00 kips
Floor LL	0.00 kips
Wall DL	1.40 kips

Just Self wt.

Design Uniform Lateral Loads

Seismic Uniform Load w_{seis}	0.038plf	1.0E
Wind Uniform Load w_{wind}	0plf	1.0W



Check TMS 402-16 Code Requirements

Axial Stress (9.3.5.4.2)

$P_{u\,max} = P_{uw} + P_{uf}$ = 1.85 kips

$P_{u\,max} / A_g$ = 0.020 ksi

Compression limit for h / t ≤ 30 = 0.20 x f'_m = 0.400 ksi — Eq (9-22)

Compression limit for h / t > 30 = 0.05 x f'_m = 0.100 ksi

Compression OK 5%

Maximum Reinforcement (9.3.3.2)

ρ = 0.0022 ρ_{max} = 0.0090

Max Reinf. OK 24%

Load Combinations (ASCE 7-16)

			U_{dead}	U_{live}	$U_{RF\,live}$	$U_{lateral}$
Seismic 1	$(1.2+0.2S_{DS})D+f_1L+1.0E$	2.3.6 (6)	1.32	0.50	0.00	1.00
Seismic 2	$(0.9-0.2S_{DS})D+1.0E$	2.3.6 (7)	0.78	0.00	0.00	1.00
Wind 1	$1.2D+1.6L_r+0.5W$	2.3.1 (3)	1.20	0.00	1.60	0.50
Wind 2	$1.2D+1.0W+f_1L+0.5L_r$	2.3.1 (4)	1.20	0.50	0.50	1.00
Wind 3	$0.9D+1.0W$	2.3.1 (5)	0.90	0.00	0.00	1.00

Use $U_{RF\,live}$ =0.2 for Snow

U_{live} reduced per exception ASCE 2.3.1

Use U_{live} = 1.0 at garages, public assembly or if LL > 100 psf

ULTIMATE STRENGTH CHECK

	ULTIMATE LOAD CASES					
	Seismic 1	**Seismic 2**	**Wind 1**	**Wind 2**	**Wind 3**	
Lateral Loads- out of plane						
$w_{u\,seis} = U_{lateral} \times w_{seis}$	0 plf	0 plf	-	-	-	
$w_{u\,wind} = U_{lateral} \times w_{wind}$	-	-	0 plf	0 plf	0 plf	
Vertical Roof & Floor Loads						
$P_f = (U_{dl} \times \Sigma DL + U_{ll} \times \Sigma LL)$	0.00 kips	0.00 kips	0.00 kips	0.00 kips	0.00 kips	
Panel Weight						
$P_w = U_{dl} \times$ Panel DL	1.85 kips	1.09 kips	1.68 kips	1.68 kips	1.26 kips	
$P_u = P_w + P_f$ =	1.85 kips	1.09 kips	1.68 kips	1.68 kips	1.26 kips	Eq (9-24)
Nominal Moment Capacity (TMS 9.3.5.2)						
$A_{se} = (A_s f_y + P_u / \phi) / f_y$ =	0.13	0.12	0.13	0.13	0.12	
$a = (P_u / \phi + A_s f_y) / (0.8 f'_m b_{eff})$ =	0.42 in	0.38 in	0.41 in	0.41 in	0.39 in	Section 9.3.5.2
$c = a / \beta_1$	0.52 in	0.47 in	0.51 in	0.51 in	0.48 in	
$I_{cr} = n (A_s + (P_u{*}t_{sp})/(f_y{*}2d)){*}(d-c)^2 + b \times c^3/3$ =	23.4 in^4	21.7 in^4	23.0 in^4	23.0 in^4	22.1 in^4	Eq (9-30)
$M_{cr} = S \times (f_r + P_u/A_g)$=	21 in-kips	20 in-kips	21 in-kips	21 in-kips	21 in-kips	Section 9.3.5.2
$\phi M_n = \phi(P_u / \phi + A_s f_y)(d - a/2)$ =	**29 in-kips**	**26 in-kips**	**28 in-kips**	**28 in-kips**	**27 in-kips**	
$M_n > 1.3\,M_{cr}$ Check	**OK**	**OK**	**OK**	**OK**	**OK**	**Mn>1.3*Mcr - OK**
Deflection from Factored Loads						
If $M_u \geq M_{cr}$ $\Delta_u = \Delta_{cr} + (5x(M_u-M_{cr})xh^2)/(48xE_m xI_{cr})$ =	-	-	-	-	-	Eq (9-26)
If $M_u < M_{cr}$ $\Delta_u = 5 * M_u * h^2 / (48 * E_m * I_g)$ =	0.00 in	0.00 in	0.00 in	0.00 in	0.00 in	Eq (9-25)
Moment at Mid-height of Wall (TMS 9.3.5.4.2)						
$M_u = q_u \times h^2/8 + P_{uf} \times e/2 + P_u \times \Delta_u$ =	**0 in-kips**	**0 in-kips**	**0 in-kips**	**0 in-kips**	**0 in-kips**	Eq (9-23)
Demand / Capacity Ratio =	**0%**	**0%**	**0%**	**0%**	**0%**	**Strength OK 0%**

SERVICE LOAD DEFLECTION CHECK

TMS 402-16 9.3.5.5

Allowable $\Delta_s = 0.007 \times h$ = 1.46 in — Eq (9-32)

		SERVICE LOAD CASES		
		Seismic	**Wind**	
$P_{s1} = \Sigma DL$ @ $(1.0+0.14{*}Sds) + \Sigma LL$ =		0.00 kips	0.00 kips	$P_{s1} = \Sigma DL + \Sigma LL$ =
P_{s2} = Panel DL @ $(1.0+0.14{*}S_{ds})$ =		1.52 kips	1.40 kips	P_{s2} = Panel DL =
$P_s = P_{s1} + P_{s2}$ =		1.52 kips	1.40 kips	$P_s = P_{s1} + P_{s2}$ =
$w_{ser} = 0.7{*}w_{seis}$ (or 0.6*w for wind)		0 plf	0 plf	
$I_{cr} = n (A_{se} + (P_u{*}t_{sp})/(f_y{*}2d)){*}(d-c)^2 + b \times c^3/3$ =	Eq (9-30)	22 in^4	22 in^4	
$M_{ser} = (w_{ser} \times h^2 \times 12)/8 + P_{s1} \times e/2 + (P_s) \times \Delta s$ =	Mser <= Mcr	0 in-kips	0 in-kips	Mser <= Mcr
If $M_{ser} < M_{cr}$ $\Delta_s = 5 * M_{ser} * L_c^2 / (48 * E_c * I_g)$ =	Eq (9-25)	0.00 in	0.00 in	
If $M_{ser} \geq M_{cr}$ $\Delta_s = \Delta_{cr} + (5x(M_{ser}-M_{cr})xh^2)/(48xE_m xI_{cr})$ =	Eq (9-26)	-	-	
Deflection/Allowable Ratio		**0%**	**0%**	**Deflection OK 0%**

BUEHLER

PROJECT DMU Delano

PROJECT NO. ______ DATE ______

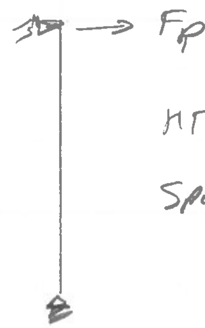
CLIENT ______ BY ______ SHEET NO. C15

OUT-OF-PLANE WALL SUPPORT

$F_p = 0.4(S_{DS} = 0.6)(K_a = 2)(I = 1.0)(W_p)$ 100 psf

$= 0.48\,W_p = 48\text{ psf}$

$F_p = 48\text{psf}(21'/2)(6.67') = 3.4^K$



HT MAX = 21'

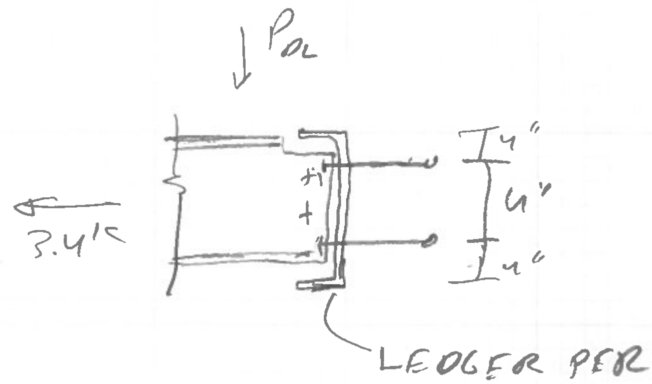
Span 6-8" MAX TYP.

CASE 1: C8 TO CONT. LEDGER

Blocking Beam

$DL = 25\text{ psf}(7')(7'\text{span}/2)$

$= 612.5\#$



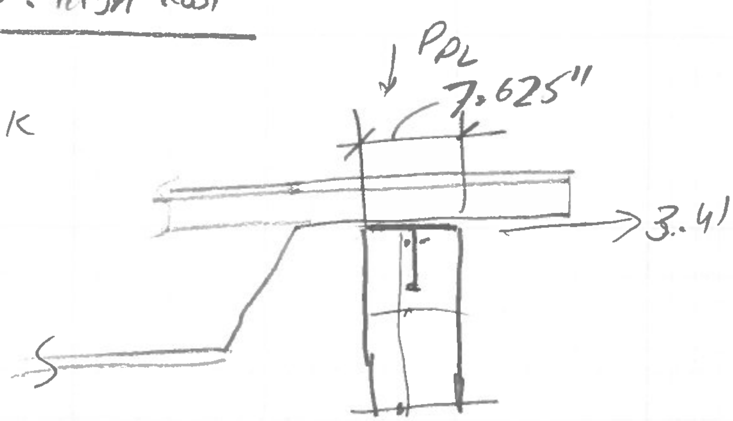
See following pages for bolt strength

(4) 3/4" ϕ Bolts ok

For vertical & O-O-P Loads.

Case 2: O.W.J. Seat on CMU: High Roof

$DL = 25\text{psf}(\frac{60}{2})(7') = 5.3^K$



C16

Project:	
Prepared by:	
Company: *Buehler & Buehler Structural Engineers, Inc.*	
Phone:	**Email:**

PROJECT INFORMATION

Anchor arrangement

Type of arrangement = Grid
Number of anchors = 4
Number of rows = 2 with spacing = 4.0 in.
Number of columns = 2 with spacing = 8.0 in.
$\ell_{be,right}$ = 12.0 in., $\ell_{be,left}$ = 12.0 in., $\ell_{be,top}$ = 4.0 in., $\ell_{be,bottom}$ = 36.0 in.
Masonry depth, t_m = 7.6 in.

Base plate properties

Length of plate in X-direction = 9.0 in.
Length of plate in Y-direction = 5.0 in.
Plates's right edge distance = 11.5 in.
Plates's left edge distance = 11.5 in.
Plates's top edge distance = 3.5 in.
Plates's bottom edge distance = 35.5 in.

Loading point eccentricities from the center of the anchors

Eccentricity in X-direction = 0.0 in.
Eccentricity in Y-direction = 0.0 in.

Masonry and Anchor properties

---

f'_m = 2000 psi
Anchor used: Headed Bar of AWS D1.1 Grade B steel
f_y = 50000 psi
Anchor diameter, d_b = 0.75 in.
Anchor effective c/s area, A_b = 0.44 in.^{2}
Anchor embedment depth, ℓ_b = 5.0 in.

Applied loads

Tension (tension is positive):
N_u = 3.4 kips
Moment about x-axis (positive moment causes compression at the top edge of the plate):
M_{ux} = 0 kips-in.

C17

Project:	
Prepared by:	
Company: *Buehler & Buehler Structural Engineers, Inc.*	
Phone:	**Email:**

Moment about y-axis (positive moment causes compression at the right edge of the plate):
$M_{uy} = 0$ kips-in.
Shear in x-direction (shear towards right edge is positive):
$V_{ux} = 0$ kips
Shear in y-directionv (shear towards top edge is positive):
$V_{uy} = 1$ kips
Moment about z-axis [pure torsion] (counterclockwise is positive):
$M_{uz} = 0$ kips-in.

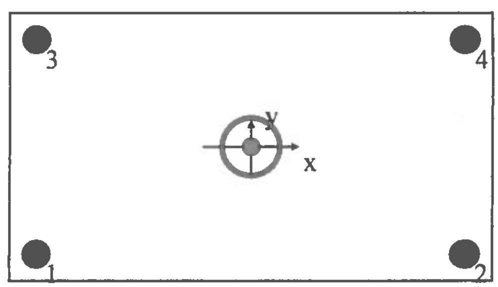
Miscellaneous information

Code used: MSJC 2013
Design type: Strength
ϕ for steel yielding = 0.9
ϕ for concrete pullout = 0.65
ϕ for concrete breakout, pryout, or crushing = 0.5

Anchor Forces (kips)

Anchor	Tension	Shear x	Shear y
1	0.85	0.00	0.25
2	0.85	0.00	0.25
3	0.85	0.00	0.25
4	0.85	0.00	0.25

Resulting tension force = 3.41 kips acting at (0.00 , 0.00) in. from the centroid of the tension anchors.



1. MASONRY BREAKOUT STRENGTH IN TENSION, ϕB_{ab}

Design strength: $\phi 4A_{pt}\sqrt{f'_m}$ [MSJC 2013 Equation 9-1]
$\phi B_{anb} = 0.5\text{x}4\ \text{x}211.82\text{x}\sqrt{2000} = 18.95$ kips

2. STEEL STRENGTH OF ANCHOR IN TENSION, ϕnB_{as}

Design strength of a single anchor: $\phi A_b f_y$ [MSJC 2013 Equation 9-2]
$\phi B_{ans} = 0.9\ \text{x}0.44\text{x}50000 = 19.80$ kips

Project:	
Prepared by:	
Company: *Buehler & Buehler Structural Engineers, Inc.*	
Phone:	Email:

Based on the most heavily loaded anchor:
Anchor No. 1 carries 0.85 kips which is equal to 25.00% of the total tension load.
Design strength of the whole group = 19.80/0.25 = 79.20 kips

3. MASONRY BREAKOUT STRENGTH IN SHEAR, ϕB_{vnb}

Design strength: $\phi 4A_{pv}\sqrt{f'_m}$ [MSJC 2013 Equation 9-6]

Shear in X-Direction

No applied shear in X-direction; masonry breakout strength calculation for shear in X-direction is skipped.

Shear in Y-Direction (positive direction)

$\phi B_{vnb} = 0.5x4\ x159.32x\sqrt{2000} = 14.25$ kips

4. MASONRY CRUSHING STRENGTH IN SHEAR, ϕnB_{vnc}

Design strength of a single anchor: $\phi 1050(f'_m\ A_b\ )^{1/4}$ [MSJC 2013 Equation 9-7]
$\phi B_{vnc} = 0.5x1050\ x(2000x0.44\ )^{1/4} = 2.86$ kips

Shear in X-Direction

No applied shear in X-direction; masonry crushing strength calculation for shear in X-direction is skipped.

Shear in Y-Direction (positive direction)

Based on the most heavily loaded anchor:
Anchor No. 4 carries 0.25 kips which is equal to 25.00% of the total shear load.
Design strength of the whole group = 2.86/0.25 = 11.44 kips

5. PRYOUT STRENGTH OF ANCHOR IN SHEAR, ϕB_{vpry}

Design strength: $\phi 2B_{anb} = \phi 8A_{pt}\sqrt{f'_m}$ [MSJC 2013 Equation 9-8]

Shear in X-Direction

No applied shear in X-direction; masonry crushing strength calculation for shear in X-direction is skipped.

C19

Project:	
Prepared by:	
Company: *Buehler & Buehler Structural Engineers, Inc.*	
Phone:	**Email:**

Shear in Y-Direction (positive direction)

$\phi B_{vpry} = 0.5 \times 8 \times 211.82 \times \sqrt{2000} = 37.89$ kips

6. STEEL STRENGTH OF ANCHOR IN SHEAR, ϕnB_{vs}

Design strength of a single anchor: $\phi 0.6 A_b f_y$ [MSJC 2013 Equation 9-9]
$\phi B_{vns} = 0.9 \times 0.6 \times 0.44 \times 50000 = 11.88$ kips

Shear in X-Direction

No applied shear in X-direction; masonry crushing strength calculation for shear in X-direction is skipped.

Shear in Y-Direction (positive direction)

Based on the most heavily loaded anchor:
Anchor No. 4 carries 0.25 kips which is equal to 25.00% of the total shear load.
Design strength of the whole group = 11.88/0.25 = 47.52 kips

SUMMARY OF DESIGN STRENGTH CALCULATIONS OF THE ANCHOR GROUP:

Tension: 18.95 kips
Shear in Y-direction: 11.44 kips

Interaction:

$b_{af} / \phi B_{an} + b_{vfy} / \phi B_{vny}$
$= (3.41/18.95) + (1.00 / 11.44)$
$= 0.27 \leq 1.0$ .... OK [MSJC 2013 Equation 9-10]

Anchor Pro v2.0.2 Simple Output **DATE: 02/13/2020**

Project:	
Prepared by:	
Company: *Buehler & Buehler Structural Engineers, Inc.*	
Phone:	**Email:**

NO APPLIED TENSION. THE FOLLOWING CALCULATIONS ARE SKIPPED:
1. MASONRY BREAKOUT STRENGTH IN TENSION; AND
2. STEEL STRENGTH OF ANCHOR IN TENSION.

3. MASONRY BREAKOUT STRENGTH IN SHEAR, ϕB_{vnb}

No applied shear in X-direction; masonry breakout strength calculation for shear in X-direction is skipped.

Design strength for shear in Y-Direction: = 4.06 kips

4. MASONRY CRUSHING STRENGTH IN SHEAR, $\phi n B_{vnc}$

No applied shear in X-direction; masonry crushing strength calculation for shear in X-direction is skipped.

Design strength for shear in Y-Direction:
Based on the shear in the most heavily loaded anchor, design strength of the whole group = 5.72 kips

5. PRYOUT STRENGTH OF ANCHOR IN SHEAR, ϕB_{vpry}

No applied shear in X-direction; masonry crushing strength calculation for shear in X-direction is skipped.

Design strength for shear in Y-Direction: 42.98 kips

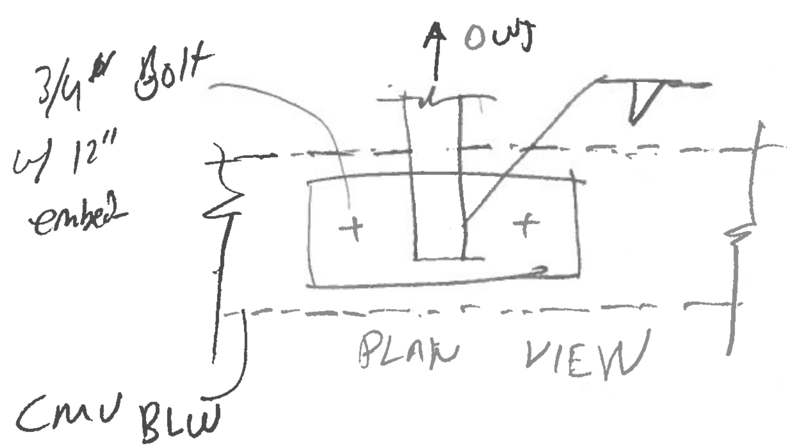
6. STEEL STRENGTH OF ANCHOR IN SHEAR, $\phi n B_{vns}$

No applied shear in X-direction; masonry crushing strength calculation for shear in X-direction is skipped.

Design strength for shear in Y-Direction:
Based on the shear in the most heavily loaded anchor, design strength of the whole group = 23.76 kips

SUMMARY OF DESIGN STRENGTH CALCULATIONS OF THE ANCHOR GROUP:

Shear in Y-direction: 4.06 kips



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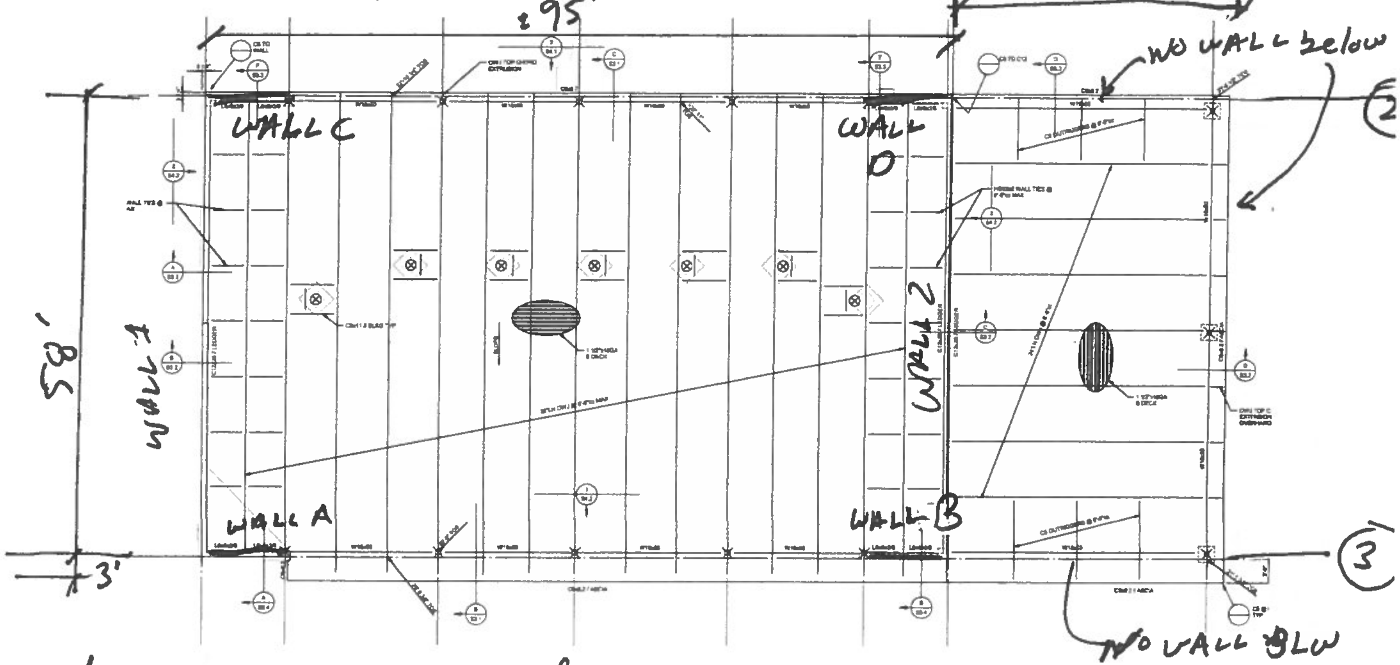
Lateral Tributary Mass

High Roof : flexible diaph. by inspection



Roof seismic mass: 25 psf

N/S

TRIB TO WALL 1: $25psf((95')/2)(58') = 69^k$

$+$ WALL ⊥ : $2(100psf)(23'/2)(95'/2) = 109^k$

178^k

TRIB TO WALL 2: $69^k + 109^k$

$+\ 25psf(35')(58') = 51^k$

Total $= 229^k$

E/W

TRIB TO WALL A & WALL C : $25psf(58'/2)(95'/2) = 35^k$ (ROOF)

$+\ 100psf(58'/2)(23'/2) = 33.5^k$ (WALL ⊥)

68.5^k

TRIB TO WALL B & WALL D : $35^k + 25psf(35')(58'/2) = 60.5^k$

$+\ 33.5^k$

Total: $\boxed{94^k}$

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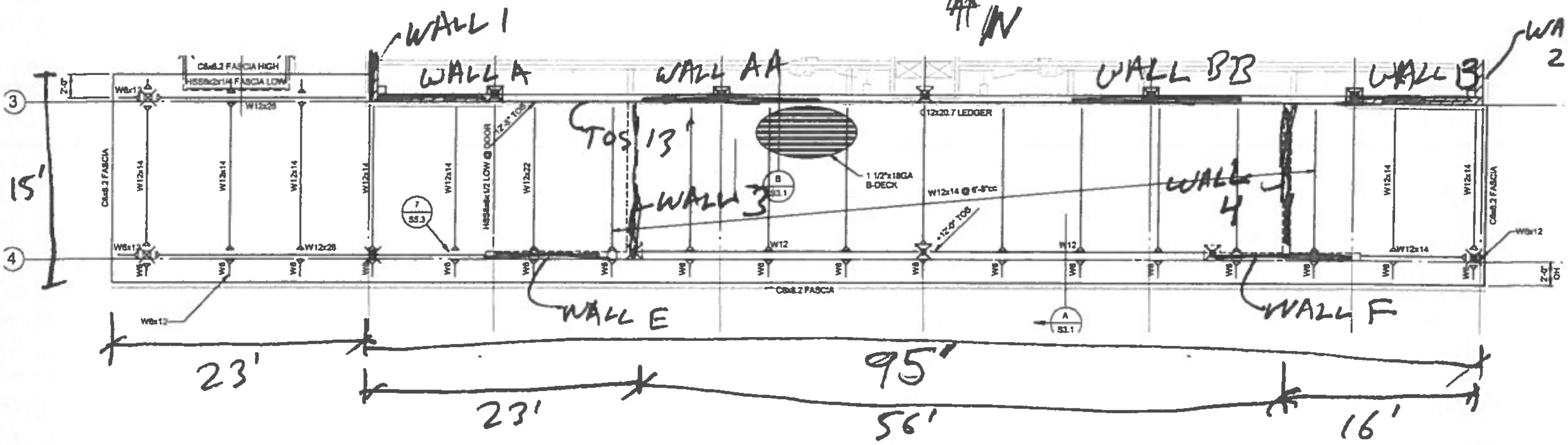
Lateral Tributary Mass

Low Roof - South



N/S : TRIB TO WALL 1: $25psf(15')(23' + 23'/2) = 13^k$

TRIB TO WALL 2: $25psf(15')(16/2) = 3^k$

TRIB TO WALL 3: $30psf(23'/2 + 56'/2)(15') = 18^k$

Add 5psf partitions

$+ 100psf(\frac{95'}{2})(13'/2) = 31^k \times 2 = 62^k$

80^k

TRIB TO WALL 4: $30psf(16/2 + 56/2)(15') = 16.2^k$

$+100(95/2)(13/2)(2) = 62^k$

78.2^k

E/W : TRIB TO LINE 3: Distribute to shear walls based on rigidity

: $30psf(15'/2)(95' + 23') = 22^k$

$+ 100psf(13'/2)(15')(2) = 20^k$

42^k

TRIB TO LINE 4: ~42K

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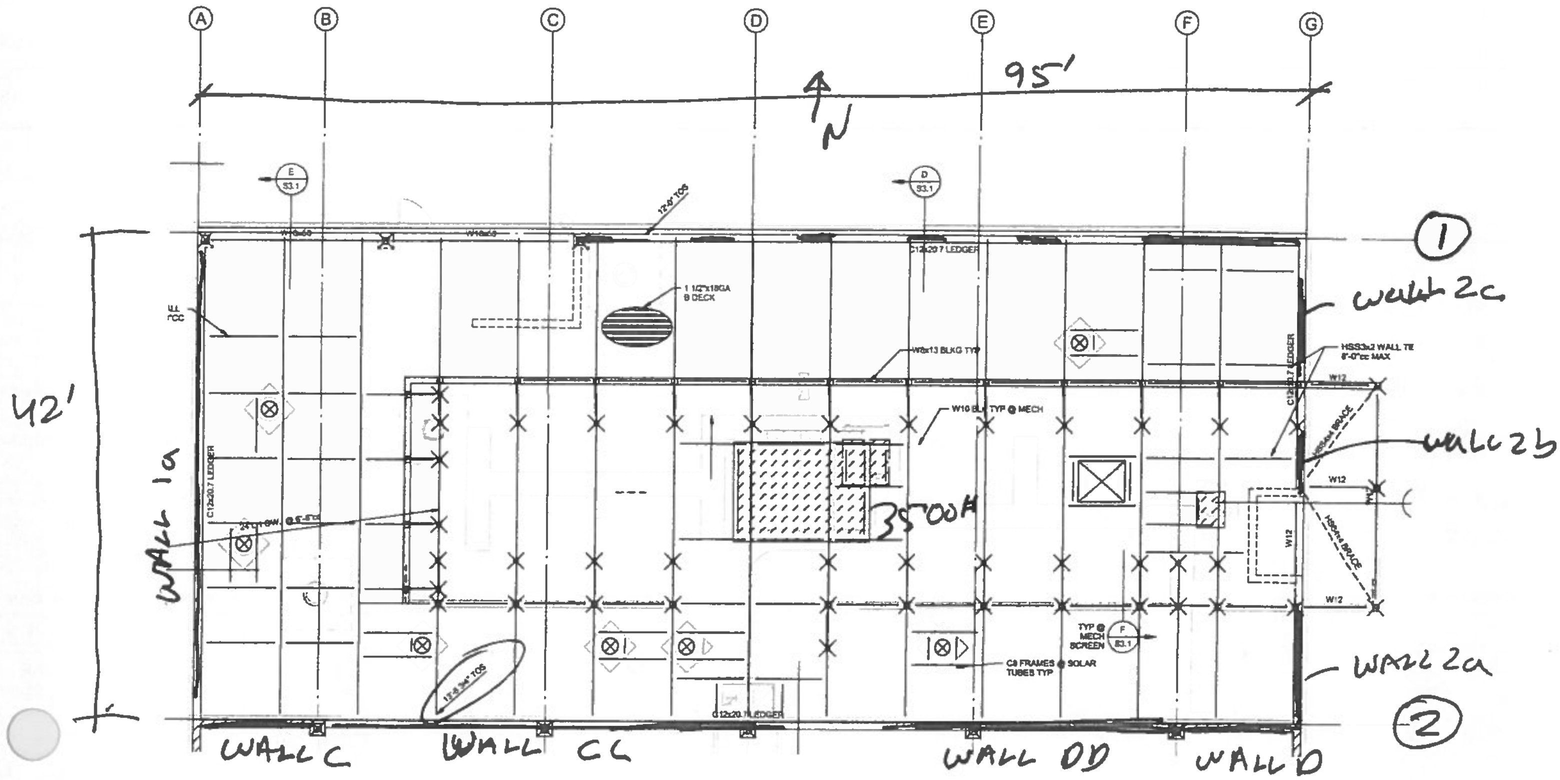
Lateral Tributary Mass

Low Roof : North



N/S Direction: TRIB TO WALL 1a: $25psf(42')(95'/2) = 50^k$

& WALL line G $+ (2)100psf(13'/2)(95'/2) = 62^k$

112^k

E/W Direction: TRIB TO LINE 1 & 2

$25psf(42'/2)(95') = 50^k$

$100psf(13'/2)(42'/2)(2) = 27.5^k$

77.5^k

Add 5 psf for partitions: $5(42'/2)(95'/2) = 5^k$ ea. wall line.

$+$ 3500# mech unit $+ 50plf(80') = 7.5^k$

Screen

12.5^k

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SEISMIC FORCES (ASCE 7-16)

Tedds calculation version 3.1.00

Site parameters

Site class D

Mapped acceleration parameters

at short periods S_S = **0.723** at 1 sec period S_1 = **0.272**

Site coefficientat short periods F_a = **1.2** at 1 sec period F_v = **1.9**

Spectral response acceleration parameters

at short period S_{MS} = **0.883** at 1 sec period S_{M1} = **0.505**

Design spectral acceleration parameters

at short period S_{DS} = **0.589** at 1 sec period S_{D1} = **0.337**

Seismic design category

Occupancy category II

Seismic design category D

Approximate fundamental period

Height above base to highest level of building h_n = **10.00** ft

Building period parameter C_t C_t = **0.02** Building period parameter x x = **0.75**

Building fundamental period T = T_a = **0.112** sec Long-period transition period T_L = **12** sec

Limiting period T_S = **0.572** sec

Seismic response coefficient

Seismic force resisting system:A. Bearing_Wall_Systems

7. Special reinforced masonry shear walls

Response modification factor R = **5**

Seismic importance factor I_e = **1.000** Seismic response coefficient C_s = **0.118**

Seismic base shear

Total Roof wt. : 25 psf (95')(114') + 25 (35')(58')

= 321.5k (Roof)

100psf (114' x 2)$\left(\frac{23+14'}{2}\right)$ = 422k (E/W wall. wt.)

avg wall Ht.

743.5k E/W

100 psf (95')(3)$\left(\frac{23+14}{2}\right)$ = 528k (N/S wall Wt.)

850k (N/S)

Max seismic base shear : 100K

L5

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Cs **0.12** w Rho= **1.00** Sds 0.606

N/S Direction Wall Stress Check

Forces High Roof		Low Roof		Check overall base shear	
WALL 1	**21.0** kips	**WALL 1**	**1.53** kips	**Seismic Base Shear**	94.9 kips
WALL 2	**27.0** kips	**WALL 1A**	**13.22** kips	**Wall Shears**	24.1 kips
		WALL LINE G	**13.22** kips	**Total Seismic Story Shear**	119.0 kips
		WALL 3	**9.44** kips		
		WALL 4	**9.44** kips		

Tributary Mass: N/S	
High Roof	
WALL 1	**178.0** kips
WALL 2	**229.0** kips
Low Roof	
WALL 1	**13.0** kips
WALL 1A	**112.0** kips
WALL LINE G	**112.0** kips
WALL 3	**80.0** kips
WALL 4	**80.0** kips

Note: Wall Weight is excluded from these masses and added below

Masonry Properties		**Steel Properties**		Fixity:
Em	900 psi			
G = 0.4 Em	360 psi	Fy	60 ksi	1 = Fixed-Fixed
F'm =	2000 psi	Horizonal Bars	4 @ 24	4 = Fixed-Pinned
8" CMU Wall	100 **psf**			

	Wall Name	Ftotal	Heigh of Wall	Wall Length	Wall Thick	Self Weigth	Force from Self Weight	Fixity	Regidity	Shear 2.5*fv	M	M/Vd	Fv TMS 9.3.4.1	Shear Check	Moment from DL (0.9-0.2*Sds)	Trim Steel T = M/(d-8")	As rqd	# of bars	H/L
		Max	Ft	ft	in	kips	kips			psi	kft		psi		kft	k			
Grid A-High	**1**	**8.6**	**23.0**	**25.0**	**7.625**	**57.5**	3.39	**4.0**	1168	13.1	198.6	0.92	135.7	**OK**	**515.0**	-13.0	no uplift	**2-#5**	0.92
	1	**12.4**	**23.0**	**30.0**	**7.625**	**69.0**	4.07	**4.0**	1673	15.0	284.5	0.77	147.7	**OK**	**618.0**	-11.4	no uplift	**2-#5**	0.77
								Sum	**2841**										
Grid A-Low	**1a**	**13.2**	**23.0**	**41.0**	**7.625**	**94.3**	5.56	**4.0**	2872	12.5	304.0	0.56	174.1	**OK**	**844.6**	-13.4	no uplift	**2-#5**	0.56
Grid G-High	**2**	**11.3**	**23.0**	**18.0**	**7.625**	**41.4**	2.44	**4.0**	564	20.9	260.3	1.28	118.9	**OK**	**370.8**	-6.4	no uplift	**2-#5**	1.28
	2	**1.3**	**23.0**	**8.0**	**7.625**	**18.4**	1.09	**4.0**	66	8.2	30.6	2.88	94.9	**OK**	**164.8**	-18.3	no uplift	**2-#5**	2.88
	2	**13.6**	**23.0**	**19.5**	**7.625**	**44.9**	2.65	**4.0**	679	22.8	313.8	1.18	122.5	**OK**	**401.7**	-4.7	no uplift	**2-#5**	1.18
	2	**0.7**	**23.0**	**6.5**	**7.625**	**15.0**	0.88	**4.0**	37	6.8	16.9	3.54	91.3	**OK**	**133.9**	-20.1	no uplift	**2-#5**	3.54
								Sum	**1346**										
Grid G-Low	**2a**	**8.1**	**13.5**	**12.5**	**7.625**	**16.9**	1.00	**4.0**	829	20.0	109.9	1.08	105.7	**OK**	**88.7**	1.8	0.1	**2-#5**	1.08
	2b	**1.4**	**13.0**	**6.0**	**7.625**	**7.8**	0.46	**4.0**	145	8.6	18.6	2.17	90.1	**OK**	**39.5**	-3.9	no uplift	**2-#5**	2.17
	2c	**3.7**	**12.5**	**8.3**	**7.625**	**10.3**	0.61	**4.0**	372	14.1	45.6	1.52	95.5	**OK**	**50.2**	-0.6	no uplift	**2-#5**	1.52
								Sum	**1346**										
Grid B/F	**3.0**	**9.4**	**13.0**	**13.0**	**7.625**	**16.9**	1.00	**4.0**	980	21.9	122.7	1.00	106.9	**OK**	**85.6**	3.0	0.1	**2-#5**	1.00
	4.0	**9.4**	**13.0**	**13.0**	**7.625**	**16.9**	1.00	**4.0**	980	21.9	122.7	1.00	106.9	**OK**	**85.6**	3.0	0.1	**2-#5**	1.00

Redudancy Check			
	total rigidity	8693	
Remove most Rigid Wall		980	Per ASCE Table 12.3-3, remove only wall with H/L greater than 1
		11%	**<33% therefore us Rho = 1.0**

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JOB	DMV Delano		
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Tributary Mass: E/W

High Roof		Low Roof	
WALL A	**68.5** kips	**WALL 1**	**77.5** kips
WALL B	**94.0**	**WALL 2(+Wall C & D Abv)**	**240.0** kips
WALL C	**68.5**	**WALL LINE 3 +Wall A & B Abv)**	**204.5** kips
WALL D	**94.0** kips	**WALL LINE 4**	**42.0** kips

Note: Wall Weight is excluded from these masses and added below

Cs **0.12** w Rho= **1.00** Sds 0.606

E/W Direction Wall Stress Check

Forces High Roof		Low Roof		Check overall base shear	
WALL A	**8.1** kips	**WALL 1**	**9.15** kips	**Seismic Base Shear**	104.9 kips
WALL B	**11.1** kips	**WALL 2**	**28.32** kips	**Wall Shears**	20.2 kips
WALL C	**8.1** kips	**WALL 3**	**24.13** kips	**Total Seismic Story Shear**	125.1 kips
WALL D	**11.1** kips	**WALL LINE 4**	**4.96** kips		

Masonry Properties		**Steel Properties**		Fixity:
Em	900 psi			
G = 0.4 Em	360 psi	Fy	60 ksi	1 = Fixed-Fixed
F'm =	2000 psi	Horizonal Bars	4 @ 24	4 = Fixed-Pinned
8" CMU Wall	100 **psf**			

										Shear						Trim Steel			
	Wall Name	Ftotal	Heigh of Wall	Wall Length	Wall Thick	Self Weigth	Force from Self Weight	Fixity	Regidity	2.5*fv	M	M/Vd	Fv TMS 9.3.4.1	Shear Check	Resisting Moment from DL	T = M/(d-8")	As rqd	# of bars	H/L
		Max	Ft	ft	in	kips	kips			psi	kft		psi		kft	k			
Grid 1	1-A	**8.3**	**12.0**	**16.5**	**7.625**	**19.8**	1.17	**4.0**	1845	15.8	100.2	0.73	115.3	OK	**92.5**	0.5	no uplift	**1#5**	0.73
	1-B	**0.4**	**12.0**	**4.7**	**7.625**	**5.6**	0.33	**4.0**	91	4.3	4.9	2.57	86.9	OK	**26.2**	-5.3	no uplift	**1#5**	2.57
	1-C	**0.2**	**12.0**	**3.3**	**7.625**	**4.0**	0.24	**4.0**	35	3.2	1.9	3.59	83.7	OK	**18.7**	-6.3	no uplift	**1#5**	3.59
	1-D	**0.2**	**12.0**	**3.3**	**7.625**	**4.0**	0.24	**4.0**	35	3.2	1.9	3.59	83.7	OK	**18.7**	-6.3	no uplift	**1#5**	3.59
	1-D	**0.1**	**12.0**	**2.5**	**7.625**	**3.0**	0.18	**4.0**	15	2.7	0.8	4.80	81.7	OK	**14.0**	-7.2	no uplift	**1#5**	4.80
								Sum	**2020**										
Grid 2 - High	C	**8.1**	**23.0**	**11.0**	**7.625**	**25.3**	1.49	**4.0**	160	23.8	185.9	2.09	102.1	OK	**226.6**	-3.9	no uplift	**1#5**	2.09
	D	**11.1**	**23.0**	**11.0**	**7.625**	**25.3**	1.49	**4.0**	160	31.3	255.1	2.09	102.1	OK	**226.6**	2.8	0.1	**1-#5**	2.09
Grid 2 - Low	C	**1.6**	**15.0**	**11.0**	**7.625**	**16.5**	0.97	**4.0**	482	6.3	23.3	1.36	102.1	OK	**96.4**	-7.1	no uplift	**1#5**	1.36
	Cc	**17.6**	**15.0**	**42.0**	**7.625**	**63.0**	3.72	**4.0**	5474	13.9	264.2	0.36	176.5	OK	**368.0**	-2.5	no uplift	**1#5**	0.36
	Dd	**7.6**	**15.0**	**23.8**	**7.625**	**35.6**	2.10	**4.0**	2364	11.2	114.1	0.63	132.7	OK	**208.1**	-4.1	no uplift	**1#5**	0.63
	D	**1.6**	**15.0**	**11.0**	**7.625**	**16.5**	0.97	**4.0**	482	6.3	23.3	1.36	102.1	OK	**96.4**	-7.1	no uplift	**1#5**	1.36
								Sum	**8803**									**1-#5**	

Grid 3 - High	**A**	**8.1**	**23.0**	**11.0**	**7.625**	**25.3**	1.49	**4.0**	160	23.8	185.9	2.09	102.1	**OK**	**226.6**	-3.9	no uplift	**1#5**	2.09
	B	**11.1**	**23.0**	**11.0**	**7.625**	**25.3**	1.49	**4.0**	160	31.3	255.1	2.09	102.1	**OK**	**226.6**	2.8	0.1	**1-#5**	2.09
Grid 3 - Low	**A**	**2.9**	**15.0**	**11.0**	**7.625**	**16.5**	0.97	**4.0**	482	9.6	43.1	1.36	102.1	**OK**	**96.4**	-5.2	no uplift	**1#5**	1.36
	Aa	**9.0**	**15.0**	**18.5**	**7.625**	**27.8**	1.64	**4.0**	1503	15.6	134.4	0.81	120.1	**OK**	**162.1**	-1.6	no uplift	**1#5**	0.81
	Bb	**9.4**	**15.0**	**19.0**	**7.625**	**28.5**	1.68	**4.0**	1582	16.0	141.4	0.79	121.3	**OK**	**166.5**	-1.4	no uplift	**1#5**	0.79
	B	**2.9**	**15.0**	**11.0**	**7.625**	**16.5**	0.97	**4.0**	482	9.6	43.1	1.36	102.1	**OK**	**96.4**	-5.2	no uplift	**1#5**	1.36
								Sum	**4050**										
Grid 4	**E**	**2.5**	**12.0**	**13.0**	**7.625**	**15.6**	0.92	**4.0**	1160	7.1	29.7	0.92	106.9	**OK**	**72.9**	-3.5	no uplift	**1#5**	0.92
	F	**2.5**	**12.0**	**13.0**	**7.625**	**15.6**	0.92	**4.0**	1160	7.1	29.7	0.92	106.9	**OK**	**72.9**	-3.5	no uplift	**1#5**	0.92
								Sum	**2320**										

Redudancy Check			
	total rigidity	17834	
Remove most Rigid Wall		482	Per ASCE Table 12.3-3, remove only wall with H/L greater than 1
		3%	**<33% therefore us Rho = 1.0**

$G = \frac{1}{2(1+\mu)}E$; $\mu_{conc} = 0.15 \therefore G = 0.44E$

$I = \frac{t d^3}{12}$ $A = td$

$\Delta_{FF} = \frac{P}{Et}\left[\left(\frac{H}{d}\right)^3 + 2.8\left(\frac{H}{d}\right)\right]$; $\boxed{R_F = \frac{1}{\Delta_{FF}}}$

$\Delta_C = \frac{P}{Et}\left[4\left(\frac{H}{d}\right)^3 + 2.8\left(\frac{H}{d}\right)\right]$, $\boxed{R_C = \frac{1}{\Delta_C}}$

FIXED-FIXED $\Delta_F = \frac{PH^3}{12E_cI} + \frac{1.2PH}{GA}$ (1)

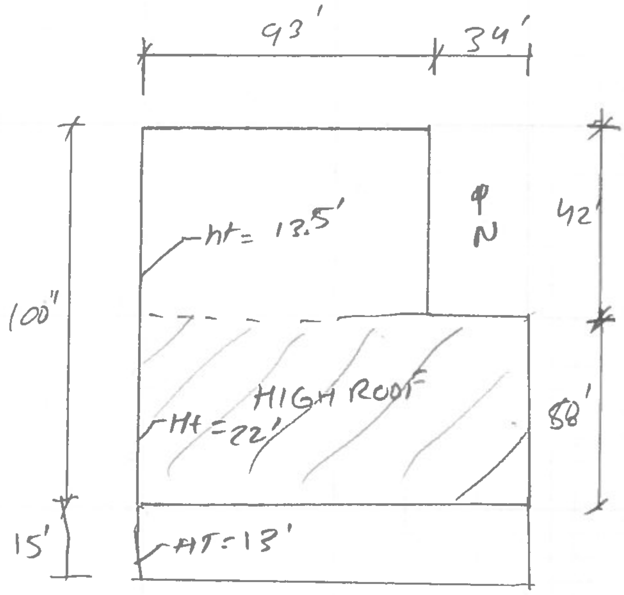
CANTILEVER $\Delta_C = \frac{PH^3}{3E_cI} + \frac{1.2PH}{GA}$ (2)

BUEHLER

PROJECT DMV Delano
PROJECT NO.
DATE
CLIENT
BY
SHEET NO. L8

WIND LOADING

- Check Base shear E/W to determine if wind governs design (90°)



W @ High roof = 18 psf + 7.4 psf
= 25.5 psf

w = 25.5 psf (22'/2) = 281 plf

W @ Low Roof : 17 psf + 13 psf
= 30 psf

w = 30 psf (13.5'/2) = 202.5 plf

$V_{E/W\ wind} = 281(58') + 202.5(42' + 15') = 27.8^{K}$

$\ll 100^{K}$ Seismic Base Shear.

USE 30 psf for O-O-P loading

Seismic governs Design.

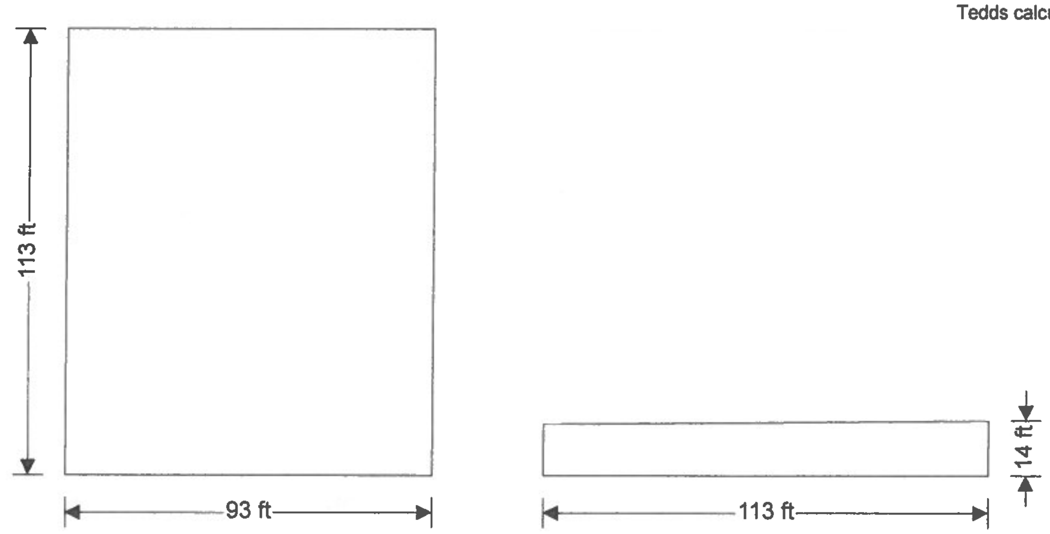
Low Roof

WIND LOADING

In accordance with ASCE7-16

Using the directional design method

Tedds calculation version 2.1.05



Plan Elevation

Building data

Type of roof	Monoslope	Length of building	b = **93.00** ft
Width of building	d = **113.00** ft	Height to eaves	H = **13.00** ft
Pitch of roof	α_0 = **0.5** deg		
Mean height	h = **13.00** ft		

General wind load requirements

Basic wind speed	V = **95.0** mph	Risk category	II
Exponent coef (Table 26.6-1)	K_d = **0.85**	Elevation above sea level	z_{gl} = **0** ft
Ground elevation factor	K_e = **1.00**	Exposure category (cl 26.7.3)	C
Enclosure class (cl.26.12)	Enclosed buildings	Int pres coef +ve	GC_{pi_p} = **0.18**
Int pres coef –ve	GC_{pi_n} = **-0.18**		
Gust effect factor	G_f = **0.85**		
Minimum design wind loading	p_{min_r} = **8** lb/ft^2		

Topography

Topo factor not significant K_{zt} = 1.0

Velocity pressure equation $q = 0.00256 \times K_z \times K_{zt} \times K_d \times V^2 \times 1\text{psf/mph}^2$

Velocity pressures table

z (ft)	K_z (Table 26.10-1)	q_z (psf)
13.00	0.85	16.69
13.99	0.85	16.69

Peak velocity pressure for internal pressure

Peak velocity pressure – int q_i = **16.69** psf

Pressures and forces

Net pressure $p = q \times G_f \times C_{pe} - q_i \times GC_{pi}$

Net force $F_w = p \times A_{ref}$

Roof load case 1 - Wind 0, GC_{pi} 0.18, $-c_{pe}$

Walls load case 1 - Wind 0, GC_{pi} 0.18, $-c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A	13.00	0.80	16.69	8.35	1209.00	10.09
B	13.00	-0.46	16.69	-9.49	1300.71	-12.34
C	13.00	-0.70	16.69	-12.94	1524.72	-19.72
D	13.00	-0.70	16.69	-12.94	1524.72	-19.72

Overall loading

Proj vertical plan area of wall $A_{vert\_w\_0}$ = **1209.00** ft^2

Projected vertical area of roof $A_{vert\_r\_0}$ = **91.71** ft^2

Leeward net force F_l = **-12.3** kips

Overall horizontal loading $F_{w,total}$ = **21.6** kips

Min overall horizontal loading $F_{w,total\_min}$ = **20.08** kips

Windward net force F_w = **10.1** kips

Roof load case 2 - Wind 0, GC_{pi} -0.18, $-0c_{pe}$

Walls load case 2 - Wind 0, GC_{pi} -0.18, $-0c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A	13.00	0.80	16.69	14.36	1209.00	17.36
B	13.00	-0.46	16.69	-3.48	1300.71	-4.53
C	13.00	-0.70	16.69	-6.93	1524.72	-10.56
D	13.00	-0.70	16.69	-6.93	1524.72	-10.56

Overall loading

Proj vertical plan area of wall $A_{vert\_w\_0}$ = **1209.00** ft^2

Projected vertical area of roof $A_{vert\_r\_0}$ = **91.71** ft^2

Leeward net force F_l = **-4.5** kips

Overall horizontal loading $F_{w,total}$ = **21.9** kips

Min overall horizontal loading $F_{w,total\_min}$ = **20.08** kips

Windward net force F_w = **17.4** kips

Roof load case 3 - Wind 90, GC_{pi} 0.18, $-c_{pe}$

Walls load case 3 - Wind 90, GC_{pi} 0.18, $-c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A	13.99	0.80	16.69	8.35	1524.94	12.73
B	13.00	-0.50	16.69	-10.10	1524.72	-15.40
C	13.00	-0.70	16.69	-12.94	1209.00	-15.64
D	13.00	-0.70	16.69	-12.94	1300.71	-16.83

Overall loading

Proj vertical plan area of wall $A_{vert\_w\_90}$ = **1524.72** ft^2

Projected vertical area of roof $A_{vert\_r\_90}$ = **0.00** ft^2

Leeward net force F_l = **-15.4** kips

Overall horizontal loading $F_{w,total}$ = **28.1** kips

Min overall horizontal loading $F_{w,total\_min}$ = **24.40** kips

Windward net force F_w = **12.7** kips

Roof load case 4 - Wind 90, GC_{pi} -0.18, $+c_{pe}$

Walls load case 4 - Wind 90, GC_{pi} -0.18, $+c_{pe}$

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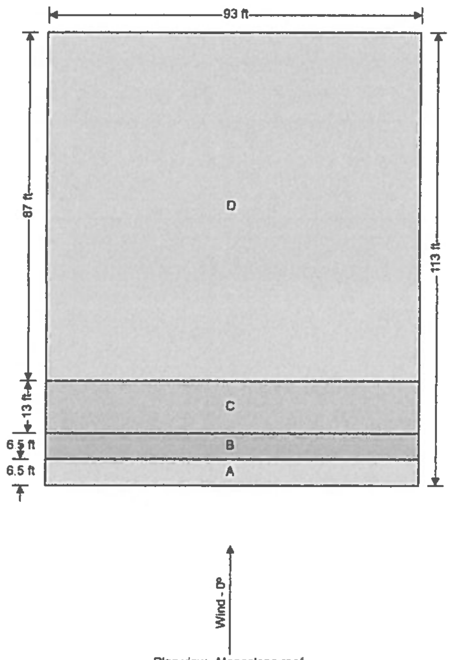
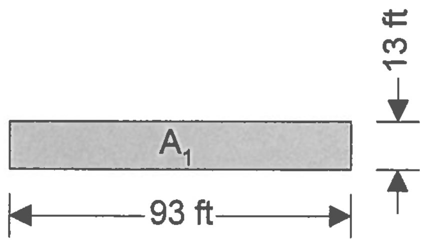
Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A	13.99	0.80	16.69	14.36	1524.94	21.89
B	13.00	-0.50	16.69	-4.09	1524.72	-6.24
C	13.00	-0.70	16.69	-6.93	1209.00	-8.38
D	13.00	-0.70	16.69	-6.93	1300.71	-9.01

Overall loading

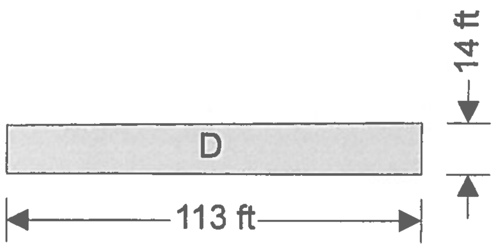
Proj vertical plan area of wall	$A_{vert\_w\_90}$ = **1524.72** ft^2		
Projected vertical area of roof	$A_{vert\_r\_90}$ = **0.00** ft^2	Min overall horizontal loading	$F_{w,total\_min}$ = **24.40** kips
Leeward net force	F_l = **-6.2** kips	Windward net force	F_w = **21.9** kips
Overall horizontal loading	$F_{w,total}$ = **28.1** kips		



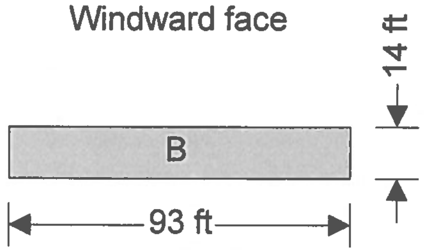
Plan view - Monoslope roof



Windward face



Side face



Leeward face

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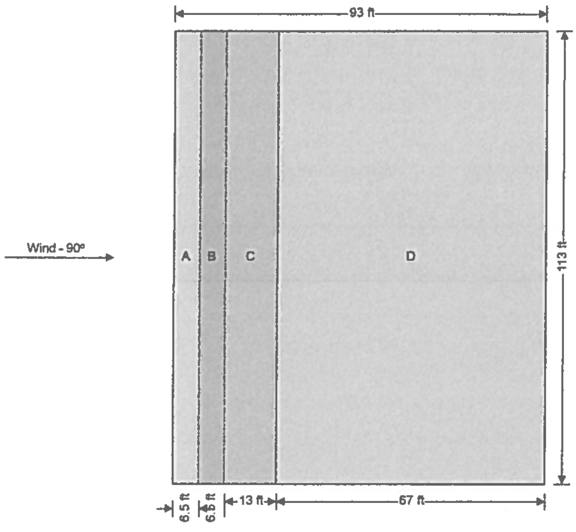
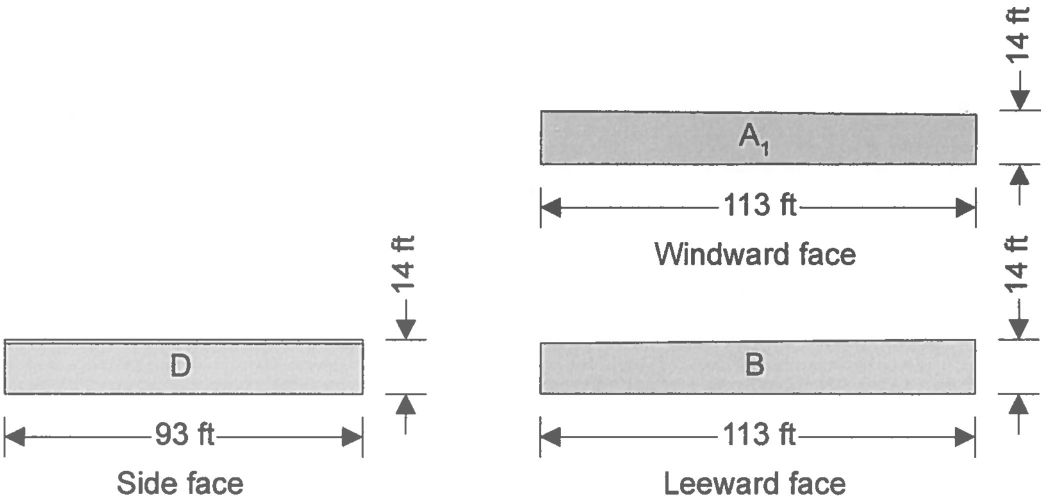


Plan view - Monoslope roof



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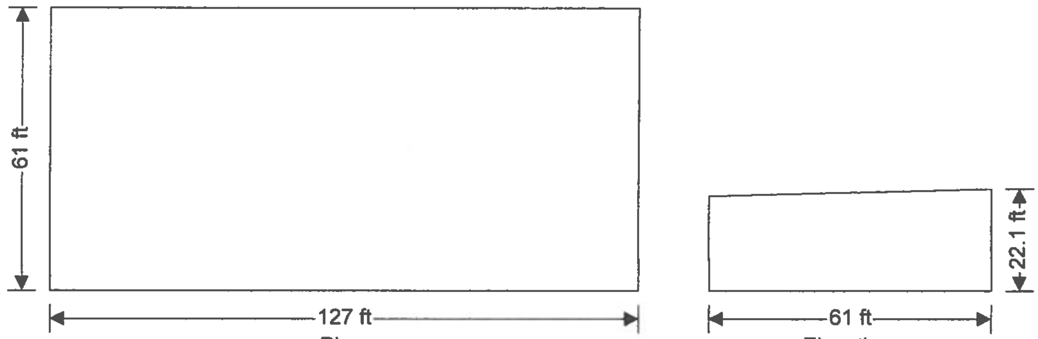
High Roof

WIND LOADING

In accordance with ASCE7-16

Using the directional design method

Tedds calculation version 2.1.05



Plan Elevation

Building data

Type of roof	Monoslope	Length of building	b = **127.00** ft
Width of building	d = **61.00** ft	Height to eaves	H = **20.50** ft
Pitch of roof	α_0 = **1.5** deg		
Mean height	h = **20.50** ft		

General wind load requirements

Basic wind speed	V = **95.0** mph	Risk category	II
Exponent coef (Table 26.6-1)	K_d = **0.85**	Elevation above sea level	z_{gl} = **0** ft
Ground elevation factor	K_e = **1.00**	Exposure category (cl 26.7.3)	C
Enclosure class (cl.26.12)	Enclosed buildings	Int pres coef +ve	GC_{pi_p} = **0.18**
Int pres coef –ve	GC_{pi_n} = **-0.18**		
Gust effect factor	G_f = **0.85**		
Minimum design wind loading	p_{min_r} = **8** lb/ft^2		

Topography

Topo factor not significant K_{zt} = 1.0

Velocity pressure equation $q = 0.00256 \times K_z \times K_{zt} \times K_d \times V^2 \times 1\text{psf/mph}^2$

Velocity pressures table

z (ft)	K_z (Table 26.10-1)	q_z (psf)
15.00	0.85	16.69
15.00	0.85	16.69
20.50	0.90	17.75
22.10	0.92	18.00

Peak velocity pressure for internal pressure

Peak velocity pressure – int q_i = **17.75** psf

Pressures and forces

Net pressure $p = q \times G_f \times C_{pe} - q_i \times GC_{pi}$

Net force $F_w = p \times A_{ref}$

Roof load case 1 - Wind 0, GC_{pi} 0.18, $-c_{pe}$

Walls load case 1 - Wind 0, GC_{pi} 0.18, $-c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A_1	15.00	0.80	16.69	8.16	1905.00	15.54
A_2	15.00	0.80	16.69	8.16	0.00	0.00
A_3	20.50	0.80	17.75	8.88	698.50	6.20
B	20.50	-0.50	17.75	-10.74	2806.36	-30.14
C	20.50	-0.70	17.75	-13.76	1299.22	-17.88
D	20.50	-0.70	17.75	-13.76	1299.22	-17.88

Overall loading

Proj vertical plan area of wall $A_{vert\_w\_0}$ = **2603.50** ft^2

Projected vertical area of roof $A_{vert\_r\_0}$ = **202.86** ft^2

Leeward net force F_l = **-30.1** kips

Overall horizontal loading $F_{w,total}$ = **49.5** kips

Min overall horizontal loading $F_{w,total\_min}$ = **43.28** kips

Windward net force F_w = **21.7** kips

Roof load case 2 - Wind 0, GC_{pi} -0.18, -0c_{pe}

Walls load case 2 - Wind 0, GC_{pi} -0.18, -0c_{pe}

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A_1	15.00	0.80	16.69	14.55	1905.00	27.71
A_2	15.00	0.80	16.69	14.55	0.00	0.00
A_3	20.50	0.80	17.75	15.27	698.50	10.66
B	20.50	-0.50	17.75	-4.35	2806.36	-12.21
C	20.50	-0.70	17.75	-7.37	1299.22	-9.57
D	20.50	-0.70	17.75	-7.37	1299.22	-9.57

Overall loading

Proj vertical plan area of wall $A_{vert\_w\_0}$ = **2603.50** ft^2

Projected vertical area of roof $A_{vert\_r\_0}$ = **202.86** ft^2

Leeward net force F_l = **-12.2** kips

Overall horizontal loading $F_{w,total}$ = **50.7** kips

Min overall horizontal loading $F_{w,total\_min}$ = **43.28** kips

Windward net force F_w = **38.4** kips

Roof load case 3 - Wind 90, GC_{pi} 0.18, -c_{pe}

Walls load case 3 - Wind 90, GC_{pi} 0.18, -c_{pe}

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A_1	15.00	0.80	16.69	8.16	915.00	7.46
A_2	15.00	0.80	16.69	8.16	0.00	0.00
A_3	22.10	0.80	18.00	9.05	384.30	3.48
B	20.50	-0.30	17.75	-7.66	1299.22	-9.95
C	20.50	-0.70	17.75	-13.76	2603.50	-35.82
D	20.50	-0.70	17.75	-13.76	2806.36	-38.61

Overall loading

Proj vertical plan area of wall $A_{vert\_w\_90}$ = **1299.22** ft^2

Projected vertical area of roof $A_{vert\_r\_90}$ = **0.00** ft^2

Min overall horizontal loading $F_{w,total\_min}$ = **20.79** kips

Leeward net force F_l = **-10.0 kips**
Windward net force F_w = **10.9 kips**
Overall horizontal loading $F_{w,total}$ = **20.9 kips**

Roof load case 4 - Wind 90, GC_{pi} -0.18, $+c_{pe}$

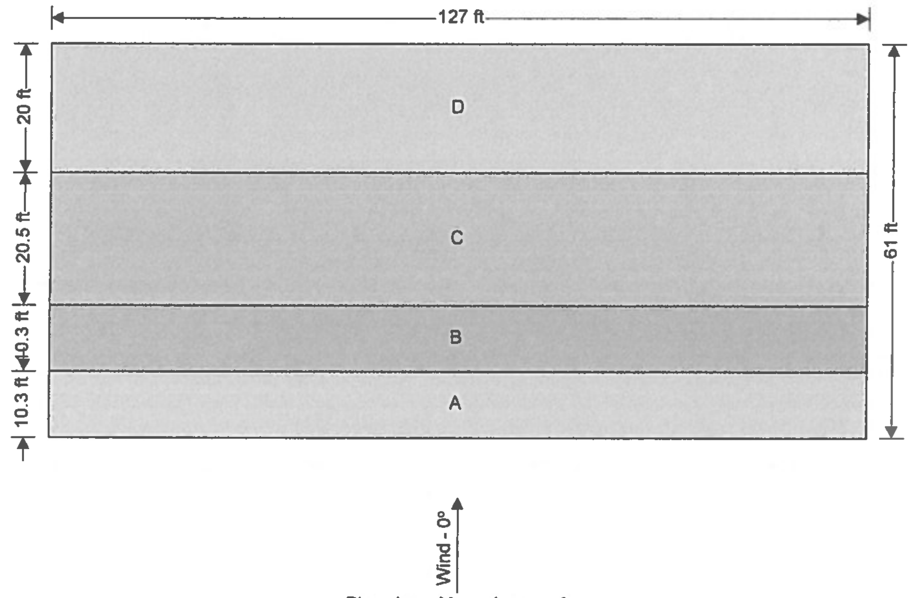
Walls load case 4 - Wind 90, GC_{pi} -0.18, $+c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A_1	15.00	0.80	16.69	14.55	915.00	13.31
A_2	15.00	0.80	16.69	14.55	0.00	0.00
A_3	22.10	0.80	18.00	15.44	384.30	5.93
B	20.50	-0.30	17.75	-1.27	1299.22	-1.65
C	20.50	-0.70	17.75	-7.37	2603.50	-19.18
D	20.50	-0.70	17.75	-7.37	2806.36	-20.68

Fw

Overall loading

Proj vertical plan area of wall $A_{vert\_w\_90}$ = **1299.22** ft^2
Projected vertical area of roof $A_{vert\_r\_90}$ = **0.00** ft^2
Min overall horizontal loading $F_{w,total\_min}$ = **20.79 kips**
Leeward net force F_l = **-1.6 kips**
Windward net force F_w = **19.2 kips**
Overall horizontal loading $F_{w,total}$ = **20.9 kips**



Plan view - Monoslope roof

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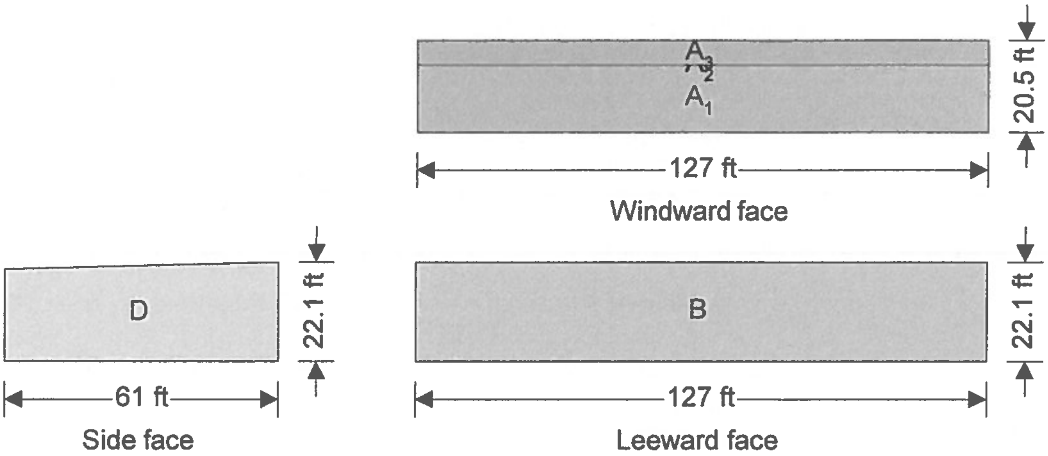
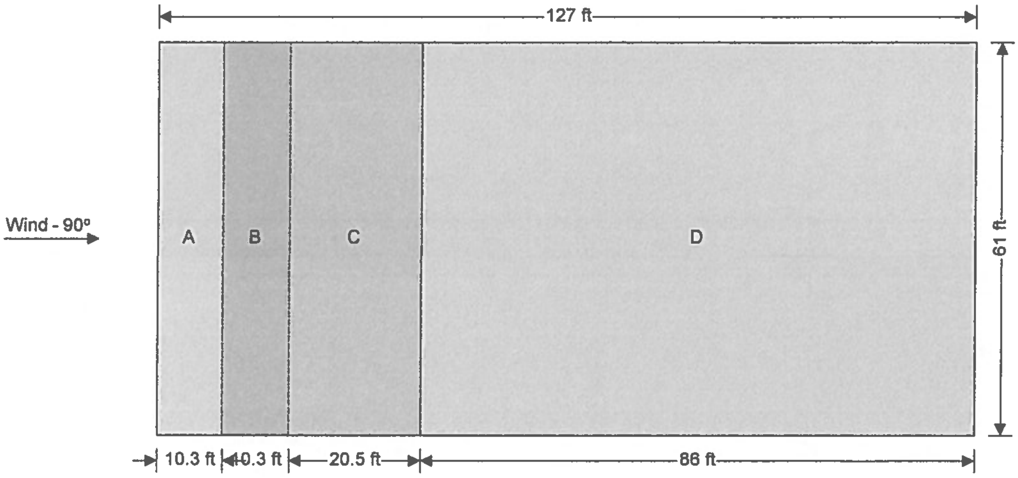
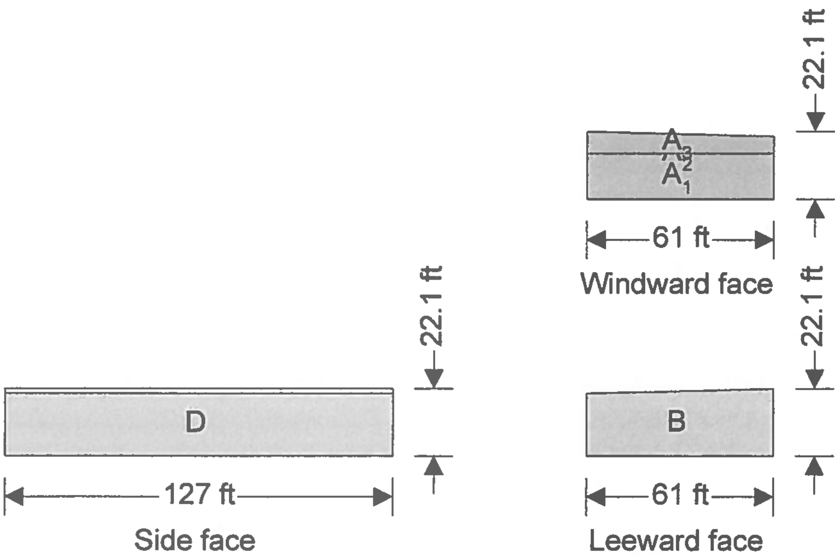




Plan view - Monoslope roof



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PROJECT Delano, DMV

PROJECT NO. ______ DATE ______

CLIENT ______ BY ______ SHEET NO. D1

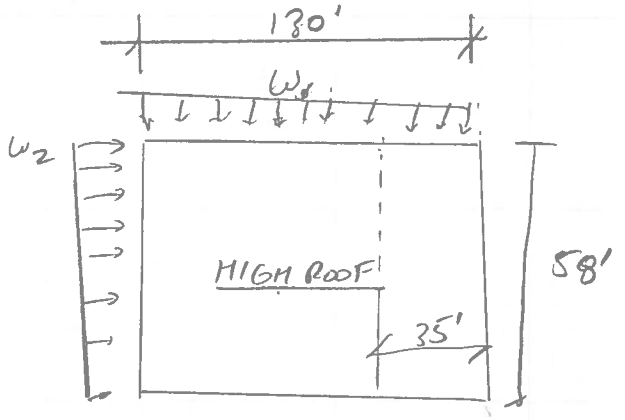
DIAPHGRAM DESIGN

Seismic governs:

$C_s \cong F_p = 0.11$

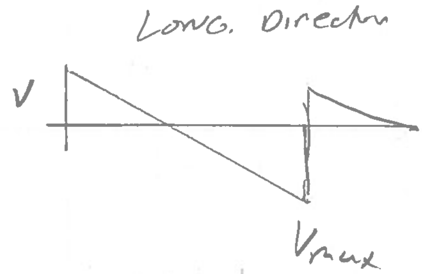
$w_1 = .11(25psf)(58') = 160\ plf$

$w_2 = .11(25psf)(130') = 358\ plf$

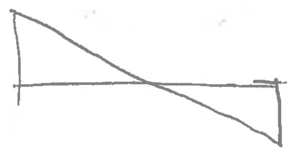


$$V_{max} = \frac{160\ plf(95+35)^2}{2(95)} = 14.2^k/58'$$

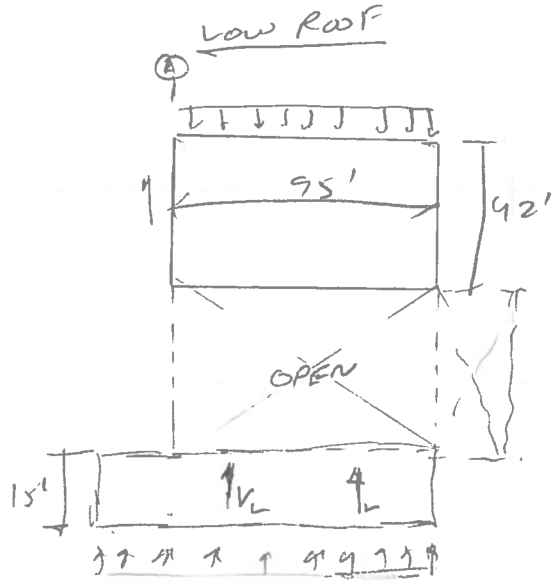
$$= 243\ plf$$



$$V_{max} = \frac{\frac{358\ plf(58')}{2}}{130'} = 80\ plf$$



Low Roof: transverse Direction governs by inspection



$$V_{max}\text{ grid A\&G}: \frac{.11(25psf)(42')(95'/2)}{42'}$$

$$= 130\ plf$$

$$V_L = V_{wall}\ 3: = 0.11(18^k)/15'$$

$$= 132\ plf$$

$\phi V_n = 1574\ plf\ (\phi = 0.55) = 866\ plf.$

Use 36/4 shot pins pattern for deck

D2

Table 10: DGB-36 and DGBF-36 Shear and Flexibility (continued)
DGB-36 and DGBF-36 with DeltaGrip and X-EDN19 / HSN-24 Pins

Gage	Support Fasteners	Seam Attachment	Allowable Shear, qa, Factored Shear, qf and Flexibility Factor, F (10^{-6} in/lbs)																		
			Span (ft)	4		5		6		7		8		9		10		11		12	
18	36/9	4	qa qf	2582	4196	2565	4168	2552	4147	2542	4131	2535	4119	2317	3707	1877	3003	1551	2481	1303	2085
			F, lap	3.8-0.4R		3.9-0.4R		4-0.3R		4-0.3R		4.1-0.3R		4.1-0.3R		4.1-0.3R		4.2-0.3R		4.2-0.2R	
		6	qa qf	2406	3909	2369	3849	2341	3805	2321	3771	2304	3744	2291	3707	1877	3003	1551	2481	1303	2085
			F, lap	4.1-0.5R		4.2-0.6R		4.4-0.5R		4.5-0.5R		4.6-0.5R		4.6-0.5R		4.7-0.5R		4.8-0.5R		4.8-0.4R	
		8	qa qf	2250	3656	2235	3632	2152	3497	2154	3501	2095	3404	2103	3418	1877	3003	1551	2481	1303	2085
			F, lap	4.3-0.7R		4.4-0.7R		4.7-0.7R		4.8-0.7R		5-0.7R		5-0.7R		5.2-0.7R		5.2-0.6R		5.3-0.6R	
		12	qa qf	2011	3268	1924	3126	1859	3021	1809	2940	1770	2876	1738	2825	1712	2782	1551	2481	1303	2085
			F, lap	4.6-0.9R		4.9-1R		5.1-1R		5.3-1.1R		5.5-1.1R		5.7-1.1R		5.9-1.1R		6-1.1R		6-0.9R	
		18	qa qf	1846	3000	1777	2888	1579	2565	1560	2536	1546	2513	1420	2307	1422	2311	1424	2314	1303	2085
			F, lap	4.7-1.1R		5.1-1.1R		5.6-1.4R		5.8-1.4R		6-1.3R		6.4-1.5R		6.5-1.5R		6.6-1.4R		6.9-1.2R	
		24	qa qf	1640	2666	1604	2606	1407	2286	1412	2295	1273	2069	1293	2101	1187	1930	1213	1970	1121	1822
			F, lap	5-1.2R		5.3-1.3R		5.8-1.6R		6-1.6R		6.5-1.7R		6.7-1.7R		7.1-1.9R		7.2-1.8R		7.4-1.5R	
		36	qa qf	1640	2666	1399	2274	1212	1969	1247	2027	1106	1798	971	1578	1031	1676	928	1507	841	1367
			F, lap	5-1.2R		5.5-1.6R		6.1-1.9R		6.3-1.8R		6.9-2R		7.4-2.3R		7.5-2.1R		7.9-2.3R		8.2-1.9R	
	36/7/4	4	qa qf	2062	3351	2055	3340	2050	3332	2047	3326	2044	3322	2042	3318	1877	3003	1551	2481	1303	2085
			F lap	4-0.3R		4-0.3R		4.1-0.3R		4.2-0.3R		4.2-0.2R		4.2-0.2R		4.3-0.2R		4.3-0.2R		4.3-0.2R	
		6	qa qf	1942	3155	1926	3129	1914	3110	1905	3096	1899	3085	1893	3076	1877	3003	1551	2481	1303	2085
			F, lap	4.4-0.5R		4.5-0.5R		4.6-0.5R		4.7-0.5R		4.8-0.5R		4.9-0.4R		4.9-0.4R		5-0.4R		5-0.3R	
		8	qa qf	1824	2964	1828	2971	1778	2889	1788	2906	1752	2847	1764	2866	1735	2820	1551	2481	1303	2085
			F, lap	4.7-0.7R		4.8-0.7R		5.1-0.7R		5.1-0.7R		5.3-0.7R		5.3-0.6R		5.5-0.6R		5.5-0.6R		5.6-0.5R	
		12	qa qf	1624	2639	1577	2562	1543	2507	1517	2466	1497	2433	1481	2407	1468	2386	1457	2368	1303	2085
			F, lap	5.1-1R		5.5-1.1R		5.7-1.1R		5.9-1.1R		6.1-1.1R		6.3-1.1R		6.4-1R		6.6-1R		6.6-0.8R	
		18	qa qf	1472	2393	1447	2352	1294	2103	1300	2113	1305	2120	1205	1958	1219	1980	1230	1999	1157	1880
			F lap	5.4-1.3R		5.8-1.3R		6.4-1.6R		6.6-1.5R		6.7-1.4R		7.3-1.6R		7.4-1.5R		7.4-1.4R		7.8-1.3R	
		24	qa qf	1272	2067	1285	2089	1131	1838	1163	1890	1053	1711	1088	1769	1003	1630	1038	1687	969	1574
			F, lap	5.8-1.6R		6.1-1.6R		6.8-1.9R		7-1.8R		7.6-2R		7.7-1.9R		8.3-2.1R		8.3-1.9R		8.6-1.7R	
		36	qa qf	1272	2067	1084	1762	939	1526	1005	1633	902	1465	816	1326	879	1428	810	1316	744	1209
			F lap	5.8-1.6R		6.6-2R		7.4-2.4R		7.5-2.2R		8.2-2.4R		8.9-2.8R		8.9-2.5R		9.5-2.7R		9.8-2.3R	
	36/4	4	qa qf	1231	2000	1230	1998	1229	1997	1228	1996	1228	1995	1227	1995	1227	1994	1227	1994	1227	1993
			F	4.1-0.3R		4.2-0.3R		4.2-0.2R		4.3-0.2R		4.3-0.2R		4.3-0.2R		4.3-0.2R		4.4-0.2R		4.4-0.1R	
		6	qa qf	1200	1950	1197	1945	1194	1941	1193	1938	1192	1936	1190	1935	1190	1933	1189	1932	1188	1931
			F	4.6-0.5R		4.8-0.5R		4.9-0.4R		4.9-0.4R		5-0.4R		5-0.3R		5.1-0.3R		5.1-0.3R		5.1-0.3R	
		8	qa qf	1165	1893	1169	1899	1154	1876	1159	1883	1149	1867	1153	1874	1145	1861	1149	1867	1142	1856
			F	5.1-0.7R		5.1-0.6R		5.4-0.7R		5.4-0.6R		5.6-0.6R		5.6-0.5R		5.8-0.5R		5.7-0.5R		5.8-0.4R	
		12	qa qf	1093	1776	1080	1754	1070	1738	1062	1726	1057	1717	1052	1709	1048	1703	1045	1698	1042	1693
			F	5.7-1.1R		6.1-1.1R		6.3-1.1R		6.5-1R		6.7-1R		6.8-1R		6.9-0.9R		7-0.9R		7.1-0.8R	
		18	qa qf	1027	1668	1023	1663	954	1550	963	1564	969	1574	920	1495	931	1512	939	1526	901	1465
			F	6.2-1.4R		6.5-1.4R		7.3-1.7R		7.4-1.5R		7.5-1.4R		8.1-1.6R		8.2-1.4R		8.2-1.4R		8.6-1.3R	
		24	qa qf	921	1496	941	1529	861	1399	888	1442	826	1342	853	1386	803	1305	829	1347	788	1280
			F	6.9-1.9R		7.1-1.8R		8-2.1R		8.1-1.9R		8.8-2.2R		8.8-1.9R		9.5-2.1R		9.4-1.9R		9.8-1.7R	
		36	qa qf	921	1496	819	1331	731	1188	788	1280	723	1175	666	1083	718	1167	672	1092	631	1025
			F	6.9-1.9R		7.9-2.4R		9-2.9R		8.9-2.5R		9.8-2.8R		10.7-3.1R		10.4-2.6R		11.2-2.9R		11.7-2.6R	

BUEHLER

PROJECT		
PROJECT NO.		DATE
CLIENT	BY	SHEET NO. C01

Collector Diagram

Level	Low Roof	Line	G

Total Line Length =	41.5ft

Line Load	wall length	average wall load
27409 lb	26.8ft	1022.7plf

V/F	**1.037**
FpMin=	0.1178
Cs	0.1136
Ω	2

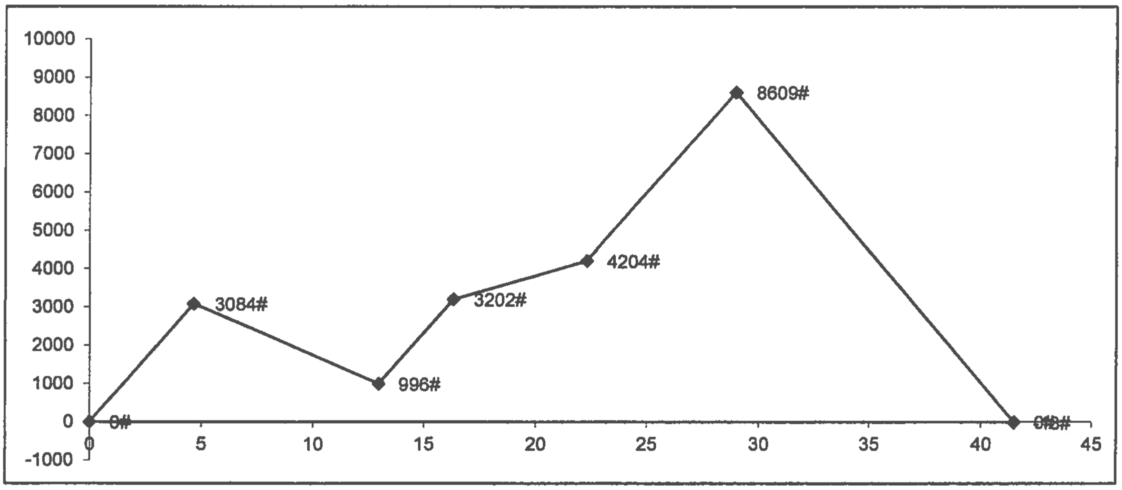
Diaphragm Loads

from	to	load	length	Description
0	41.5ft	660.5plf	41.5ft	
41.5ft	0.0ft	0.0plf	0.0ft	
41.5ft	0.0ft	0.0plf	0.0ft	
41.5ft	0.0ft	0.0plf	0.0ft	
41.5ft	0.0ft	0.0plf	0.0ft	
41.5ft	0.0ft	0.0plf	0.0ft	
41.5ft	0.0ft	0.0plf	0.0ft	

Wall Loads

from	to	load	length	Description
0	4.7ft	0.0plf	0.0ft	
4.7ft	13.0ft	912.0plf	8.3ft	
13.0ft	16.3ft	0.0plf	0.0ft	
16.3ft	22.3ft	493.6plf	6.0ft	
22.3ft	29.0ft	0.0plf	0.0ft	
29.0ft	41.5ft	1350.2plf	12.5ft	
41.5ft	0.0ft	0.0plf	0.0ft	
0.0ft	0.0ft	0.0plf	0.0ft	
0.0ft	0.0ft	0.0plf	0.0ft	
0.0ft	0.0ft	0.0plf	0.0ft	
0.0ft	0.0ft	0.0plf	0.0ft	
0.0ft				

Collector Force Diagram



BUEHLER

PROJECT			
PROJECT NO.		DATE	
CLIENT	BY	SHEET NO.	CD2

Collector Diagram

Level	High	Line	3

Line 2 Similar

Total Line Length =	127.0ft

V/F	**1.037**
FpMin=	0.1178
Cs	0.1136
Ω	2

Line Load	wall length	average wall load
39768 lb	22.0ft	1807.6plf

Diaphragm Loads

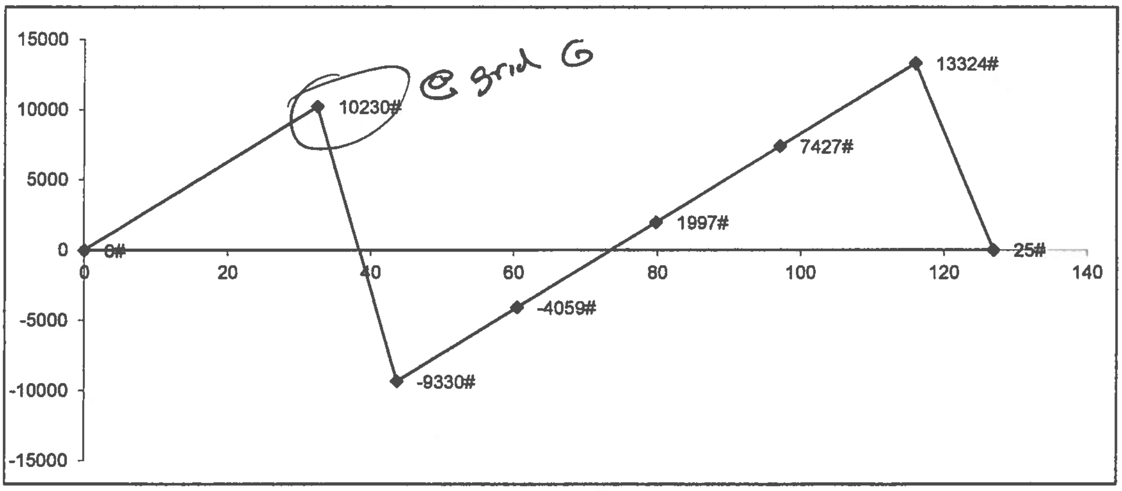
from	to	load	length	Description
0	127.0ft	313.1plf	127.0ft	
127.0ft	0.0ft	0.0plf	0.0ft	
127.0ft	0.0ft	0.0plf	0.0ft	
127.0ft	0.0ft	0.0plf	0.0ft	
127.0ft	0.0ft	0.0plf	0.0ft	
127.0ft	0.0ft	0.0plf	0.0ft	
127.0ft	0.0ft	0.0plf	0.0ft	

Wall Loads

from	to	load	length	Description
0	32.7ft	0.0plf	0.0ft	
32.7ft	43.7ft	2091.3plf	11.0ft	
43.7ft	60.5ft	0.0plf	0.0ft	
60.5ft	79.8ft	0.0plf	0.0ft	
79.8ft	97.2ft	0.0plf	0.0ft	
97.2ft	116.0ft	0.0plf	0.0ft	
116.0ft	127.0ft	1524.0plf	11.0ft	
127.0ft	127.0ft	0.0plf	0.0ft	
127.0ft	0.0ft	0.0plf	0.0ft	
127.0ft	0.0ft	0.0plf	0.0ft	
127.0ft	0.0ft	0.0plf	0.0ft	
127.0ft				

39767.86972
0 lb

Collector Force Diagram



BUEHLER

PROJECT	
PROJECT NO.	DATE
CLIENT BY	SHEET NO. CD3

Collector Diagram

Level	Low Roof	Line	3

Total Line Length =	93.8ft

Line Load	wall length	average wall load
50046 lb	59.5ft	841.1plf

V/F	**1.037**
FpMin=	0.1178
Cs	0.1136
Ω	2

Diaphragm Loads

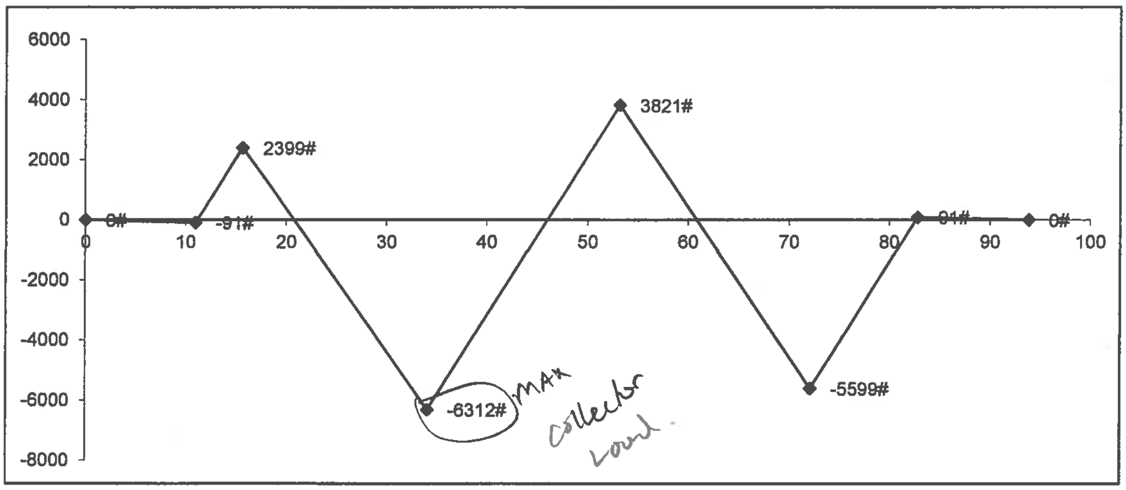
from	to	load	length	Description
0	93.8ft	533.3plf	93.8ft	
93.8ft	0.0ft	0.0plf	0.0ft	
93.8ft	0.0ft	0.0plf	0.0ft	
93.8ft	0.0ft	0.0plf	0.0ft	
93.8ft	0.0ft	0.0plf	0.0ft	
93.8ft	0.0ft	0.0plf	0.0ft	
93.8ft	0.0ft	0.0plf	0.0ft	

Wall Loads

from	to	load	length	Description
0	11.0ft	541.6plf	11.0ft	Wall B
11.0ft	15.7ft	0.0plf	0.0ft	
15.7ft	34.2ft	1004.2plf	18.5ft	
34.2ft	53.2ft	0.0plf	0.0ft	
53.2ft	72.2ft	1029.1plf	19.0ft	
72.2ft	82.8ft	0.0plf	0.0ft	
82.8ft	93.84ft	541.6plf	11.0ft	Wall A
93.8ft	93.84ft	0.0plf	0.0ft	
93.8ft	0.0ft	0.0plf	0.0ft	
93.8ft	0.0ft	0.0plf	0.0ft	
93.8ft	0.0ft	0.0plf	0.0ft	

93.8ft

Collector Force Diagram



PROJECT

PROJECT NO. DATE

CLIENT BY SHEET NO. CD4

Collector Diagram

Level	Low Roof	Line	2

Total Line Length =	94.1ft

Line Load	wall length	average wall load
58734 lb	88.1ft	666.9plf

V/F	**1.037**
FpMin=	0.1178
Cs	0.1136
Ω	2

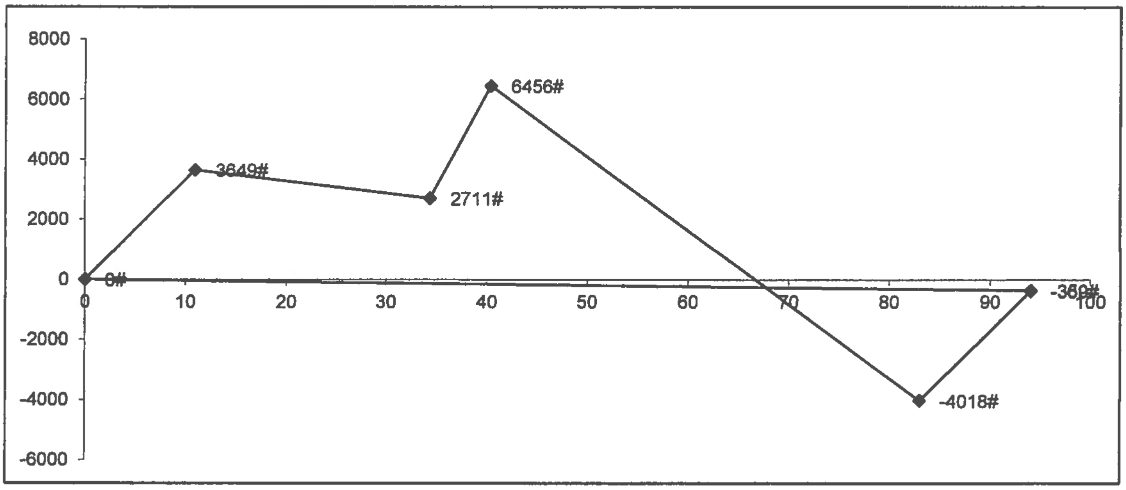
Diaphragm Loads

from	to	load	length	Description
0	94.1ft	624.2plf	94.1ft	
94.1ft	0.0ft	0.0plf	0.0ft	
94.1ft	0.0ft	0.0plf	0.0ft	
94.1ft	0.0ft	0.0plf	0.0ft	
94.1ft	0.0ft	0.0plf	0.0ft	
94.1ft	0.0ft	0.0plf	0.0ft	
94.1ft	0.0ft	0.0plf	0.0ft	

Wall Loads

from	to	load	length	Description
0	11.0ft	292.4plf	11.0ft	
11.0ft	34.4ft	664.2plf	23.4ft	
34.4ft	40.4ft	0.0plf	0.0ft	
40.4ft	83.1ft	869.6plf	42.7ft	
83.1ft	94.1ft	292.4plf	11.0ft	
94.1ft	94.1ft	0.0plf	0.0ft	
94.1ft	0.00ft	0.0plf	0.0ft	
94.1ft	0.00ft	0.0plf	0.0ft	
94.1ft	0.0ft	0.0plf	0.0ft	
94.1ft	0.0ft	0.0plf	0.0ft	
94.1ft	0.0ft	0.0plf	0.0ft	
94.1ft				

Collector Force Diagram



BUEHLER

PROJECT

PROJECT NO. DATE

CLIENT BY SHEET NO. CD5

Collector Diagram

Level	Low Roof	Line	1

Total Line Length =	94.7ft

Line Load	wall length	average wall load
18966 lb	30.4ft	624.9plf

V/F	**1.037**
FpMin=	0.1178
Cs	0.1136
Ω	2

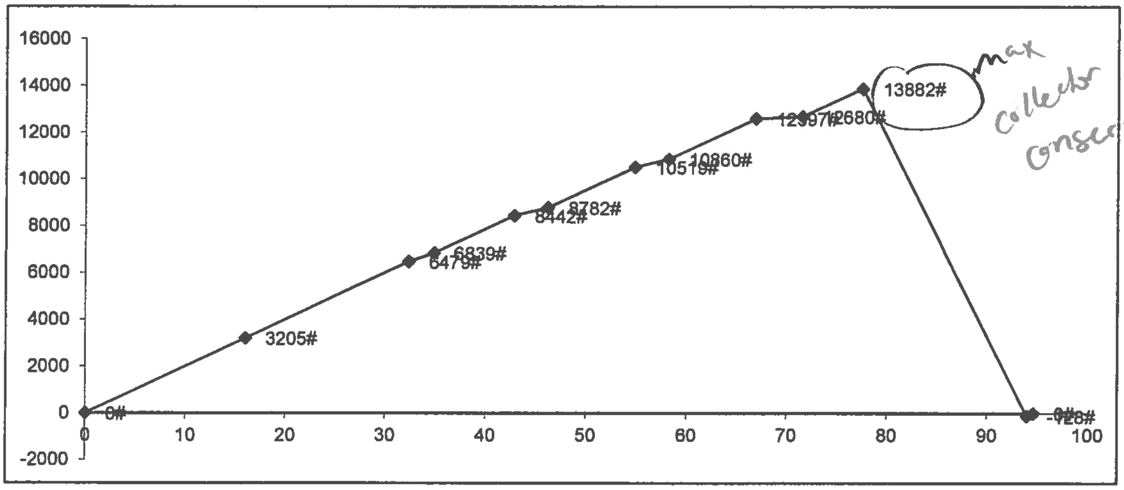
Diaphragm Loads

from	to	load	length	Description
0	94.7ft	200.3plf	94.7ft	
94.7ft	0.0ft	0.0plf	0.0ft	
94.7ft	0.0ft	0.0plf	0.0ft	
94.7ft	0.0ft	0.0plf	0.0ft	
94.7ft	0.0ft	0.0plf	0.0ft	
94.7ft	0.0ft	0.0plf	0.0ft	
94.7ft	0.0ft	0.0plf	0.0ft	

Wall Loads

from	to	load	length	Description
0	16.0ft	0.0plf	0.0ft	
16.0ft	32.3ft	0.0plf	0.0ft	
32.3ft	34.8ft	56.4plf	2.5ft	w
34.8ft	42.8ft	0.0plf	0.0ft	
42.8ft	46.2ft	98.3plf	3.3ft	w
46.2ft	54.9ft	0.0plf	0.0ft	
54.9ft	58.19ft	98.3plf	3.3ft	w
58.2ft	66.86ft	0.0plf	0.0ft	
66.9ft	71.5ft	182.5plf	4.7ft	w
71.5ft	77.5ft	0.0plf	0.0ft	
77.5ft	94.0ft	1049.5plf	16.5ft	w
94.0ft				

Collector Force Diagram



max collector load conservatively

PROJECT		
PROJECT NO.		DATE
CLIENT	BY	SHEET NO. CD6

Collector Diagram

Level	Low Roof	Line	4

Total Line Length =	117.0ft

V/F	1.037
FpMin=	0.1178
Cs	0.1136
Ω	2

Line Load	wall length	average wall load
10278 lb	26.0ft	395.3plf

Diaphragm Loads

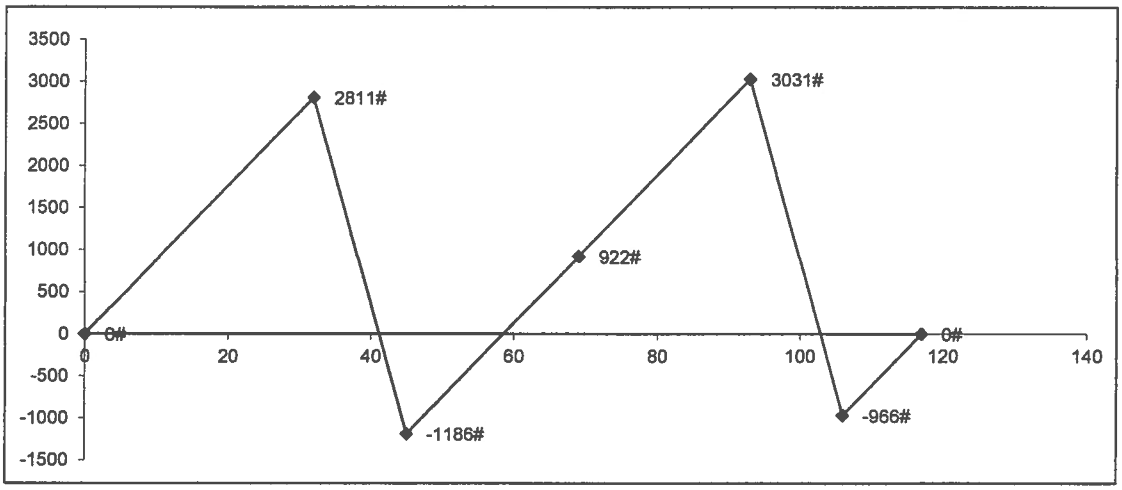
from	to	load	length	Description
0	117.0ft	87.9plf	117.0ft	
117.0ft	0.0ft	0.0plf	0.0ft	
117.0ft	0.0ft	0.0plf	0.0ft	
117.0ft	0.0ft	0.0plf	0.0ft	
117.0ft	0.0ft	0.0plf	0.0ft	
117.0ft	0.0ft	0.0plf	0.0ft	
117.0ft	0.0ft	0.0plf	0.0ft	

Wall Loads

from	to	load	length	Description
0	32.0ft	0.0plf	0.0ft	
32.0ft	45.0ft	395.3plf	13.0ft	
45.0ft	69.0ft	0.0plf	0.0ft	
69.0ft	93.0ft	0.0plf	0.0ft	
93.0ft	106.0ft	395.3plf	13.0ft	
106.0ft	117.0ft	0.0plf	0.0ft	
117.0ft	0.00ft	0.0plf	0.0ft	
117.0ft	0.00ft	0.0plf	0.0ft	
117.0ft	0.0ft	0.0plf	0.0ft	
117.0ft	0.0ft	0.0plf	0.0ft	
117.0ft	0.0ft	0.0plf	0.0ft	

117.0ft

Collector Force Diagram



BUEHLER

JOB Typical Connection Schedule

JOB NO DATE

CLIENT BY SHEET NO

Single Vertical Row-Plate Shear Connection

Connection Summary Table						Extended (WF to WF Col Web) Factored Load Design Capacities Vertical Only			Check	Conventional (WF to WF BEAM and COL Flange) Factored Load Design Capacities Vertical Only			Check	Horizontal Tension Capacity
Beam Size	Shear Plate Thickness	No. Bolts	Bolt Diameter	Bolt Type	Weld 'W'	Coped T&B Kips	Coped Top Only Kips	Uncoped Beams Kips		Coped T&B Kips	Coped Top Only Kips	Uncoped Beams Kips		Single Row Kips
W8	1/4	2	7/8	A325-N	3/16	9.9	17.6	19.6	N/A	9.9	17.6	23.4	N/A	30.5
W10	1/4	2	7/8	A325-N	3/16	12.9	21.9	21.9	N/A	12.9	25.6	26.1	N/A	34.0
W12	1/4	3	7/8	A325-N	3/16	23.6	35.8	35.8	N/A	23.6	36.2	42.4	N/A	53.7
W14	1/4	3	7/8	A325-N	3/16	33.4	40.0	40.0	N/A	33.4	42.4	42.4	N/A	59.9
W16	3/8	4	7/8	A325-N	1/4	57.3	57.3	57.3	N/A	60.6	71.3	83.2	N/A	89.6
W18	3/8	5	7/8	A325-N	1/4	71.9	71.9	71.9	N/A	92.1	101.8	102.8	N/A	121.5
W21	3/8	5	7/8	A325-N	1/4	71.9	71.9	71.9	N/A	102.8	102.8	102.8	N/A	121.5
W24	3/8	6	7/8	A325-N	1/4	86.3	86.3	86.3	N/A	121.0	121.0	121.0	N/A	145.8
W27	3/8	7	7/8	A325-N	1/4	105.7	105.7	105.7	N/A	141.9	141.9	141.9	N/A	170.1
W30	1/2	8	1	A490-N	5/16	202.3	202.3	202.3	OK	202.3	202.3	202.3	OK	306.7
W33	1/2	9	1	A490-N	5/16	226.7	226.7	226.7	OK	226.7	226.7	226.7	OK	345.0
W36	1/2	10	1	A490-N	5/16	251.2	251.2	251.2	OK	251.2	251.2	251.2	OK	383.3
W40	1/2	11	1	A490-N	5/16	275.7	275.7	275.7	OK	275.7	275.7	275.7	OK	421.7

Set Ram Steel Connection capacity for coped T&B. Then check flagged connections for higher capacities where appropriate.

Double Vertical Row-Plate Shear Connection

Connection Summary Table						Extended (WF to WF Col Web) Factored Load Design Capacities Vertical Only			Check	Standard (WF to WF BEAM and COL Flange) Factored Load Design Capacities Vertical Only			Check	Horizontal Tension Capacity
Beam Size	Shear Plate Thickness	No. Bolts	Bolt Diameter	Bolt Type	Weld 'W'	Coped T&B Kips	Coped Top Only Kips	Uncoped Beams Kips		Coped T&B Kips	Coped Top Only Kips	Uncoped Beams Kips		Double Rows Kips
W8	1/4	2	7/8	A325-N	3/16	9.9	16.7	16.7	OK	9.9	16.7	16.7	OK	37.3
W10	1/4	2	7/8	A325-N	3/16	12.9	16.7	16.7	OK	12.9	16.7	16.7	OK	41.7
W12	1/4	3	7/8	A325-N	3/16	23.6	31.8	31.8	N/A	23.6	31.8	31.8	N/A	63.4
W14	1/4	3	7/8	A325-N	3/16	31.8	31.8	31.8	N/A	31.8	31.8	31.8	N/A	70.7
W16	3/8	4	7/8	A325-N	1/4	60.6	73.0	73.0	OK	60.6	73.0	73.0	OK	103.6
W18	3/8	5	7/8	A325-N	1/4	92.1	99.3	99.3	N/A	92.1	99.3	99.3	N/A	153.6
W21	3/8	5	7/8	A325-N	1/4	99.3	99.3	99.3	OK	99.3	99.3	99.3	OK	171.3
W24	3/8	6	7/8	A325-N	1/4	122.3	122.3	122.3	OK	122.3	122.3	122.3	OK	203.9
W27	3/8	7	7/8	A325-N	1/4	141.9	141.9	141.9	OK	141.9	141.9	141.9	OK	236.5
W30	1/2	8	1	A490-N	5/16	202.3	202.3	202.3	OK	202.3	202.3	202.3	OK	337.1
W33	1/2	9	1	A490-N	5/16	226.7	226.7	226.7	OK	226.7	226.7	226.7	OK	377.9
W36	1/2	10	1	A490-N	5/16	251.2	251.2	251.2	OK	251.2	251.2	251.2	OK	418.7
W40	1/2	11	1	A490-N	5/16	275.7	275.7	275.7	OK	275.7	275.7	275.7	OK	459.5

C07

PROJECT	DMV Delano				
PROJECT NO.	2018-0187			DATE	12/2/2019
CLIENT	NLA	BY	GB	SHEET NO.	CD8

Typical Collector Beam at High Room Typicals

W18X50 **Non Slender Steel WF Beam-Column Capacity** **AISC LRFD 15th edition**

L_b = 84 in L_{ux} = 240 in F_y = 50 ksi ϕ_b = 0.9 K_x = 1.0

L_{uy} = 240 in E = 29000 ksi $\phi_c = \phi_t$ = 0.9 K_y = 1.0

Beam-Column Properties:

b_f =	7.50 in	h/t_w =	45.20	A_g =	14.70 in2	r_x =	7.38 in	S_x =	89 in3
t_f =	0.57 in	r_{TS} =	1.98 in	J =	1.24 in4	r_y =	1.65 in	Z_x =	101 in3

Compactness Checks: **Slenderness Check:** **KL/r < 200, OK**

Flexure:

web	$\lambda_p = 3.76(E/F_y)^{1/2}$ =	90.55	OK
flange	$\lambda_p = 0.38(E/F_y)^{1/2}$ =	9.15	OK

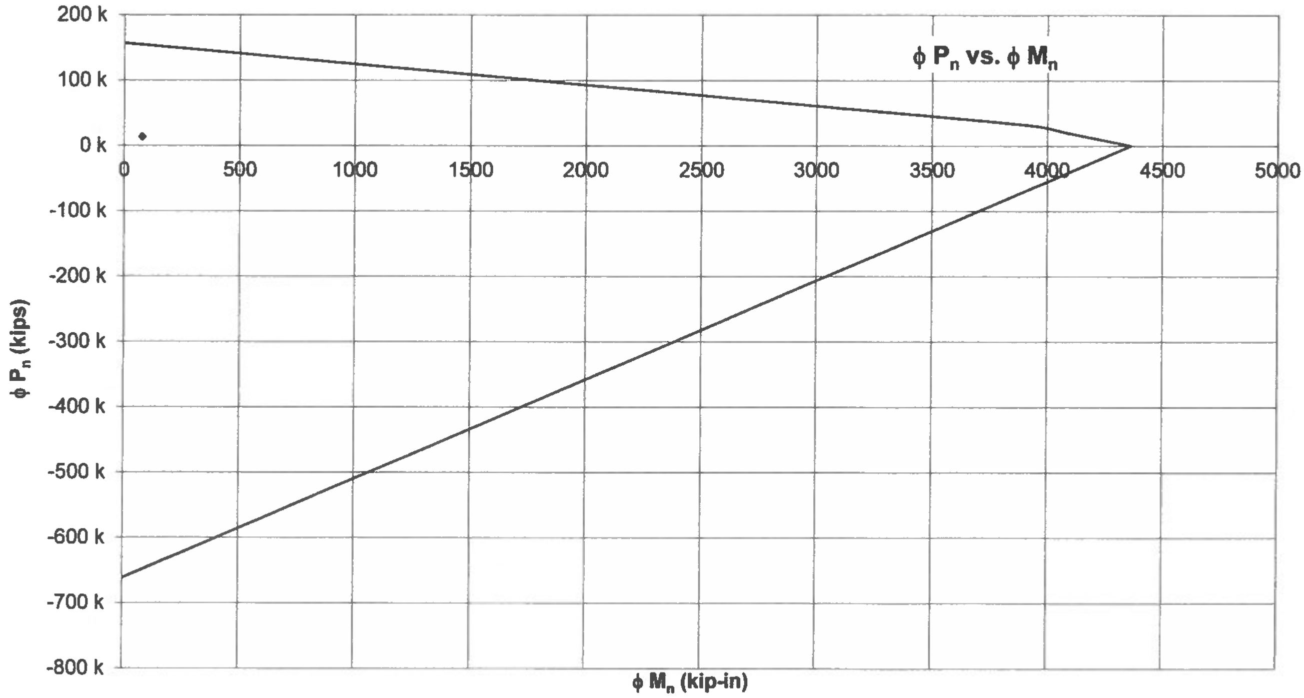
Compression:

web	$\lambda_r = 1.49(E/F_y)^{1/2}$ =	35.88	**Web is slender**
flange	$\lambda_r = 0.56(E/F_y)^{1/2}$ =	13.49	**OK**

Spreadsheet cannot be used to check pure compression for this section

Beam Column Interaction Diagram Values:

ϕMn (k-in)	4361	4089	3925	3067	2453	1840	1227	613	0	Compression
ϕPn (kips)	0 k	20 k	31 k	59 k	78 k	98 k	118 k	137 k	157 k	
ϕPn (kips)	0 k	-83 k	-132 k	-248 k	-331 k	-413 k	-496 k	-579 k	-662 k	Tension
ϕMn (k-in)	4361	3816	3489	2726	2181	1635	1090	545	0	



User input values (factored)

Mu (k-in)	78
Pu (kips)	13

Pu: See Collector Load Diagram Line 3 and 2

Mu Used 1.D+1.6Lr Load Combination Conservatively

F:\2018-018700\04-Calculations\05-Lateral\[05-WF beam column AISC 360-05 v01.1.xlsm]W18x50

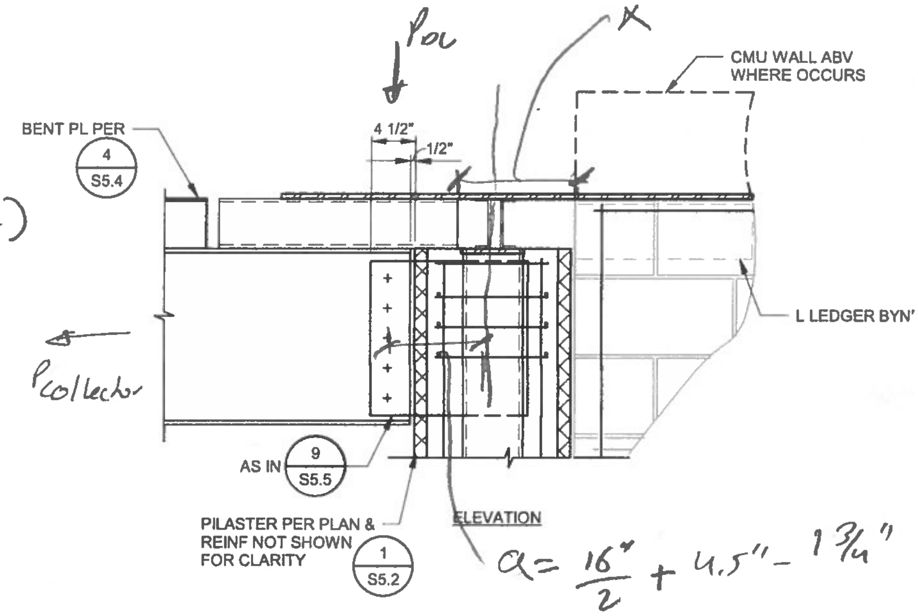
Collector @ Grid 2 & 3 : HIGH ROOF

– Grid 2 & 3 Similar loads

$P = 13.3^k$ @ grid 3
(includes $\Omega_o = 2$)

$P_{DL} = 8.7^k$ (Bm 1)

$a = 10.75''$



$a = \frac{16''}{2} + 4.5'' - 1\frac{3}{4}''$

$= 10.75''$

– See following page.
typ. extended shear tab OK

– PLATE @ TOP

$t_{req} = \frac{13.3^k}{0.9(50ksi)(2.5'')} = 0.118''$ Use 3/8" gr. 50

X = unbraced length. Check ϕP_n

$r_y = d/\sqrt{12} = \frac{0.375''}{\sqrt{12}} = 0.108$ in $\quad \frac{KL}{r_y} = 12''/0.108 = 111 < 2$
OK.

$\phi F_{cr} = 18.3$ ksi $\quad \phi P_n = 18.3(3/8'')(2.5'')$
$= 17^k > 13.3^k$ OK.

– Use 1/4" weld OK by inspection

USE 3/8" x 2.5"
Gr. 50 stitch PL

LEDGER BOLT TO CMU

See following page.

BUEHLER

JOB	DMV Delano		
JOB NO	2018-0187	DATE	2/14/2020
CLIENT	NLA BY GB	SHEET NO	CD10

Steel WF Beam - Single Plate Bolted Connection

AISC LRFD 15th Edition

W18X50

BOLTS

d_b	D_{Hole}	n_x	n_y	s_x	s_y	Bolt Shear ϕr_n	Bolt Bearing & Tearout $\phi r_{n,min}$	Bolt Grade
0.875 in	0.938 in	1	5	3	3	24.3 k	26.6 k	A325N

BEAM

	r	A	S	I	Z	d	k	t_w	T
XX	7.38 in	14.7 in²	88.9 in³	800.0 in4	101.0 in³	18.0 in	0.97 in	0.355 in	16.06 in
YY	1.65 in	-	10.7 in³	40.1 in4	16.6 in³				

F_{yb}	F_{ub}
50 ksi	65 ksi

Connection Type
Collector

Cope Detailing

Support Member		Cope	e_c	c	d_{ct}	d_{cb}
HSS Column		None				

PLATE

L_{eh}	L_{ev}	t_{pl}	h_{pl}	F_{yp}	F_{up}	a	Eccentricity	t_{max}
1.75 in	1.75 in	0.750 in	15.50 in	50 ksi	65 ksi	16.00 in	8.00 in	0.308 in

(Eq. 10-3)

WELD

t_{weld}	$t_{weld,min}$	F_{exx}	l_w	Type
0.5000 in	0.4688 in	70 ksi	15.50 in	2-Fillet

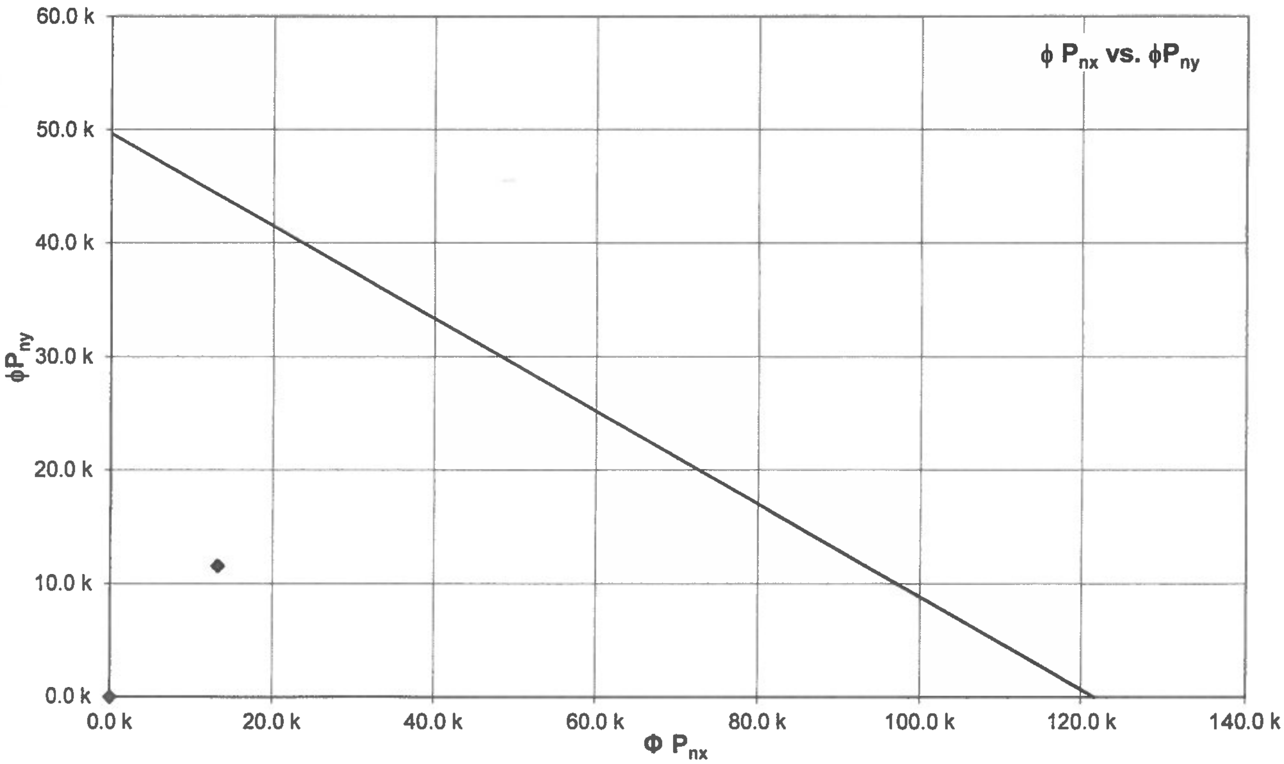
ϕP_{ny}	ϕP_{nx}
49.7 k	0.0 k
0.0 k	121.7 k
Bolts	Bolts

Capacities are based on an Extended Configuration

X-Direction = Axial Loads

Y-Direction = Gravity Loads

Connection Geometry Does Not Match Schedule



P_{ux} - kips	13.3
P_{uy} - kips	11.6

Location

CD11

Project:	
Prepared by:	
Company: *Buehler & Buehler Structural Engineers, Inc.*	
Phone:	**Email:**

PROJECT INFORMATION

Anchor arrangement

Type of arrangement = Grid
Number of anchors = 5
Number of rows = 1
Number of columns = 5 with spacing = 8.0 in.
$\ell_{be,right}$ = 36.0 in., $\ell_{be,left}$ = 8.0 in., $\ell_{be,top}$ = 3.0 in., $\ell_{be,bottom}$ = 36.0 in.
Masonry depth, t_m = 7.6 in.

Base plate properties

Length of plate in X-direction = 33.0 in.
Length of plate in Y-direction = 1.0 in.
Plates's right edge distance = 7.5 in.
Plates's left edge distance = 35.5 in.
Plates's top edge distance = 2.5 in.
Plates's bottom edge distance = 35.5 in.

Loading point eccentricities from the center of the anchors

Eccentricity in X-direction = 0.0 in.
Eccentricity in Y-direction = 0.0 in.

Masonry and Anchor properties

----

f'_m = 2000 psi
Anchor used: Headed Bar of AWS D1.1 Grade B steel
f_y = 50000 psi
Anchor diameter, d_b = 0.75 in.
Anchor effective c/s area, A_b = 0.44 in.^{2}
Anchor embedment depth, ℓ_b = 5.0 in.

Applied loads

Tension (tension is positive):
 N_u = 0 kips
Moment about x-axis (positive moment causes compression at the top edge of the plate):
 M_{ux} = 0 kips-in.

CD12

Moment about y-axis (positive moment causes compression at the right edge of the plate):
M_{uy} = 0 kips-in.
Shear in x-direction (shear towards right edge is positive):
V_{ux} = 13.3 kips
Shear in y-directionv (shear towards top edge is positive):
V_{uy} = 0 kips
Moment about z-axis [pure torsion] (counterclockwise is positive):
M_{uz} = 0 kips-in.

Miscellaneous information

Code used: MSJC 2013
Design type: Strength
ϕ for steel yielding = 0.9
ϕ for concrete pullout = 0.65
ϕ for concrete breakout, pryout, or crushing = 0.5

Anchor Forces (kips)

Anchor	Tension	Shear x	Shear y
1	0.00	2.66	0.00
2	0.00	2.66	0.00
3	0.00	2.66	0.00
4	0.00	2.66	0.00
5	0.00	2.66	0.00

Resulting tension force = 0.00 kips acting at (0.00 , 0.00) in. from the centroid of the tension anchors.

NO APPLIED TENSION. THE FOLLOWING CALCULATIONS ARE SKIPPED:
1. MASONRY BREAKOUT STRENGTH IN TENSION; AND
2. STEEL STRENGTH OF ANCHOR IN TENSION.

3. MASONRY BREAKOUT STRENGTH IN SHEAR, ϕB_{vnb}

Design strength: $\phi 4 A_{pv} \sqrt{f'_m}$ [MSJC 2013 Equation 9-6]

CD13

Project:	
Prepared by:	
Company: *Buehler & Buehler Structural Engineers, Inc.*	
Phone:	**Email:**

Shear in X-Direction (positive direction)

$\phi B_{vnb} = 0.5x4\ x297.37x\sqrt{2000} = 26.60$ kips

Shear in Y-Direction

No applied shear in Y-direction; masonry breakout strength calculation for shear in Y-direction is skipped.

4. MASONRY CRUSHING STRENGTH IN SHEAR, ϕnB_{vnc}

Design strength of a single anchor: $\phi 1050(f'_m\ A_b)^{1/4}$ [MSJC 2013 Equation 9-7]
$\phi B_{vnc} = 0.5x1050\ x(2000x0.44)^{1/4} = 2.86$ kips

Shear in X-Direction (positive direction)

Based on the most heavily loaded anchor:
Anchor No. 1 carries 2.66 kips which is equal to 20.00% of the total shear load.
Design strength of the whole group = 2.86/0.20 = 14.30 kips

Shear in Y-Direction

No applied shear in Y-direction; masonry crushing strength calculation for shear in Y-direction is skipped.

5. PRYOUT STRENGTH OF ANCHOR IN SHEAR, ϕB_{vpry}

Design strength: $\phi 2B_{anb} = \phi 8A_{pt}\sqrt{f'_m}$ [MSJC 2013 Equation 9-8]

Shear in X-Direction (positive direction)

$\phi B_{vpry} = 0.5x8\ x304.37x\sqrt{2000} = 54.45$ kips

Shear in Y-Direction

No applied shear in Y-direction; masonry crushing strength calculation for shear in Y-direction is skipped.

6. STEEL STRENGTH OF ANCHOR IN SHEAR, ϕnB_{vs}

Design strength of a single anchor: $\phi 0.6A_b\ f_y$ [MSJC 2013 Equation 9-9]
$\phi B_{vns} = 0.9x0.6\ x0.44x50000 = 11.88$ kips

Project:	
Prepared by:	
Company: *Buehler & Buehler Structural Engineers, Inc.*	
Phone:	**Email:**

Shear in X-Direction (positive direction)

Based on the most heavily loaded anchor:
Anchor No. 1 carries 2.66 kips which is equal to 20.00% of the total shear load.
Design strength of the whole group = 11.88/0.20 = 59.40 kips

Shear in Y-Direction

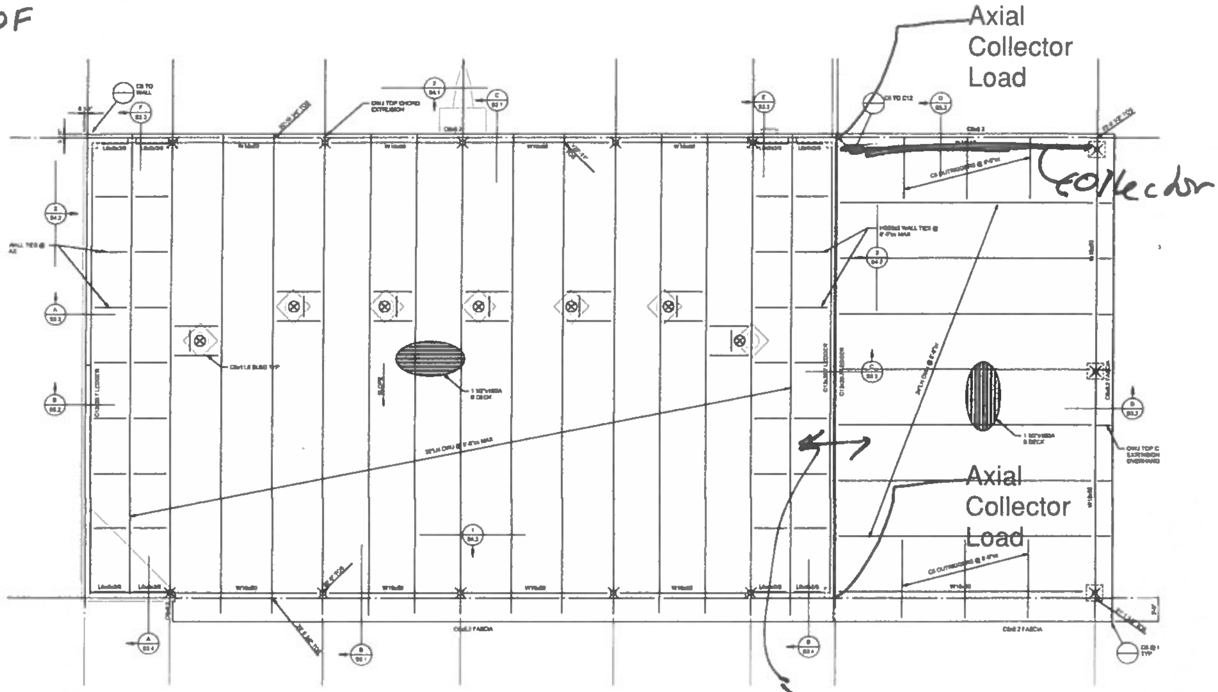
No applied shear in Y-direction; masonry crushing strength calculation for shear in Y-direction is skipped.

SUMMARY OF DESIGN STRENGTH CALCULATIONS OF THE ANCHOR GROUP:

Shear in X-direction: 14.30 kips

PROJECT	DMV Delano		
PROJECT NO.	2018-0187	DATE	11/11/2019
CLIENT	NLA	BY GB	PAGE NO. CD15

CARPORT SUB-DIAPH TO MAIN ROOF

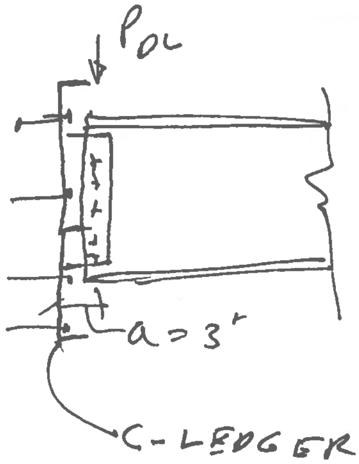


Axial = 10.2kips (includs omega)
See Collector diagram for grid 3.

Gravity Reaction:
25psf * 7ft * 35ft * 0.5 = 3.1kips

(1.2+0.2Sds) * D = 4.5kips

$M_{PL} = 4.5^k(3") = 13.5\ k\text{-}in$



See attached anchor bolt check

(8) Bolts w/ 5" embed ok.

For ALT approach of Load applied @ cant. cols, w/ T/C resisted @ grids 2 & 3 Same Bolts ok.

CD16

Project:	
Prepared by:	
Company: *Buehler & Buehler Structural Engineers, Inc.*	
Phone:	**Email:**

PROJECT INFORMATION

Bm to cmu along grid 6 @ 2 & 3

Anchor arrangement

Type of arrangement = Grid
Number of anchors = 8
Number of rows = 4 with spacing = 4.0 in.
Number of columns = 2 with spacing = 16.0 in.
$\ell_{be,right}$ = 12.0 in., $\ell_{be,left}$ = 12.0 in., $\ell_{be,top}$ = 5.0 in., $\ell_{be,bottom}$ = 36.0 in.
Masonry depth, t_m = 7.6 in.

Base plate properties

Length of plate in X-direction = 17.0 in.
Length of plate in Y-direction = 13.0 in.
Plates's right edge distance = 11.5 in.
Plates's left edge distance = 11.5 in.
Plates's top edge distance = 4.5 in.
Plates's bottom edge distance = 35.5 in.

Loading point eccentricities from the center of the anchors

Eccentricity in X-direction = 0.0 in.
Eccentricity in Y-direction = 0.0 in.

Masonry and Anchor properties

---

f'_m = 2000 psi
Anchor used: Headed Bar of AWS D1.1 Grade B steel
f_y = 50000 psi
Anchor diameter, d_b = 0.75 in.
Anchor effective c/s area, A_b = 0.44 in.^{2}
Anchor embedment depth, ℓ_b = 5.0 in.

Applied loads

Tension (tension is positive):
N_u = 10.2 kips
Moment about x-axis (positive moment causes compression at the top edge of the plate):
M_{ux} = 13.5 kips-in.

Project:	
Prepared by:	
Company: *Buehler & Buehler Structural Engineers, Inc.*	
Phone:	**Email:**

Moment about y-axis (positive moment causes compression at the right edge of the plate):
M_{uy} = 0 kips-in.
Shear in x-direction (shear towards right edge is positive):
V_{ux} = 0 kips
Shear in y-directionv (shear towards top edge is positive):
V_{uy} = -4.5 kips
Moment about z-axis [pure torsion] (counterclockwise is positive):
M_{uz} = 0 kips-in.

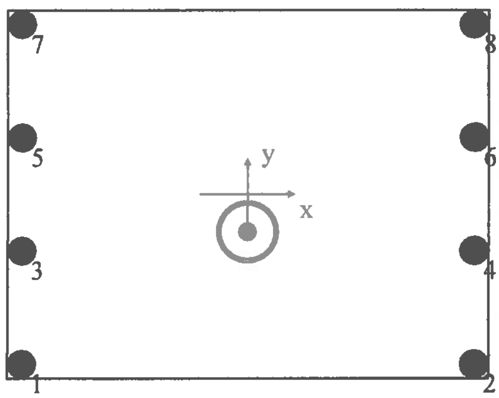
Miscellaneous information

Code used: MSJC 2013
Design type: Strength
ϕ for steel yielding = 0.9
ϕ for concrete pullout = 0.65
ϕ for concrete breakout, pryout, or crushing = 0.5

Anchor Forces (kips)

Anchor	Tension	Shear x	Shear y
1	1.79	0.00	-0.56
2	1.79	0.00	-0.56
3	1.45	0.00	-0.56
4	1.45	0.00	-0.56
5	1.11	0.00	-0.56
6	1.11	0.00	-0.56
7	0.77	0.00	-0.56
8	0.77	0.00	-0.56

Resulting tension force = 10.23 kips acting at (0.00 , -1.32) in. from the centroid of the tension anchors.



1. MASONRY BREAKOUT STRENGTH IN TENSION, ϕB_{ab}

Design strength: $\phi 4 A_{pt} \sqrt{f'_m}$ [MSJC 2013 Equation 9-1]
ϕB_{anb} = 0.5x4 x390.90x$\sqrt{2000}$ = 34.96 kips

CD18

Project:	
Prepared by:	
Company: *Buehler & Buehler Structural Engineers, Inc.*	
Phone:	**Email:**

2. STEEL STRENGTH OF ANCHOR IN TENSION, ϕnB_{as}

Design strength of a single anchor: $\phi A_b f_y$ [MSJC 2013 Equation 9-2]
ϕB_{ans} = 0.9 x0.44x50000 = 19.80 kips

Based on the most heavily loaded anchor:
Anchor No. 1 carries 1.79 kips which is equal to 17.46% of the total tension load.
Design strength of the whole group = 19.80/0.17 = 113.38 kips

3. MASONRY BREAKOUT STRENGTH IN SHEAR, ϕB_{vnb}

Design strength: $\phi 4A_{pv}\sqrt{f'_m}$ [MSJC 2013 Equation 9-6]

Shear in X-Direction

No applied shear in X-direction; masonry breakout strength calculation for shear in X-direction is skipped.

Shear in Y-Direction (negative direction)

ϕB_{vnb} = 0.5x4 x305.00x$\sqrt{2000}$ = 27.28 kips

4. MASONRY CRUSHING STRENGTH IN SHEAR, ϕnB_{vnc}

Design strength of a single anchor: $\phi 1050(f'_m A_b)^{1/4}$ [MSJC 2013 Equation 9-7]
ϕB_{vnc} = 0.5x1050 x(2000x0.44)$^{1/4}$ = 2.86 kips

Shear in X-Direction

No applied shear in X-direction; masonry crushing strength calculation for shear in X-direction is skipped.

Shear in Y-Direction (negative direction)

Based on the most heavily loaded anchor:
Anchor No. 8 carries 0.56 kips which is equal to 12.50% of the total shear load.
Design strength of the whole group = 2.86/0.13 = 22.88 kips

5. PRYOUT STRENGTH OF ANCHOR IN SHEAR, ϕB_{vpry}

Design strength: $\phi 2B_{anb} = \phi 8A_{pt}\sqrt{f'_m}$ [MSJC 2013 Equation 9-8]

CD19

Project:	
Prepared by:	
Company: *Buehler & Buehler Structural Engineers, Inc.*	
Phone:	**Email:**

Shear in X-Direction

No applied shear in X-direction; masonry crushing strength calculation for shear in X-direction is skipped.

Shear in Y-Direction (negative direction)

ϕB_{vpry} = 0.5x8 x390.90x$\sqrt{2000}$ = 69.93 kips

6. STEEL STRENGTH OF ANCHOR IN SHEAR, ϕnB_{vs}

Design strength of a single anchor: $\phi 0.6 A_b f_y$ [MSJC 2013 Equation 9-9]
ϕB_{vns} = 0.9x0.6 x0.44x50000 = 11.88 kips

Shear in X-Direction

No applied shear in X-direction; masonry crushing strength calculation for shear in X-direction is skipped.

Shear in Y-Direction (negative direction)

Based on the most heavily loaded anchor:
Anchor No. 8 carries 0.56 kips which is equal to 12.50% of the total shear load.
Design strength of the whole group = 11.88/0.13 = 95.04 kips

SUMMARY OF DESIGN STRENGTH CALCULATIONS OF THE ANCHOR GROUP:

Tension: 34.96 kips
Shear in Y-direction: 22.88 kips

Interaction:

$b_{af} / \phi B_{an} + b_{vfy} / \phi B_{vny}$
= (10.23/34.96) + (4.50 /22.88)
= 0.49 $\leq$ 1.0 .... OK [MSJC 2013 Equation 9-10]

PROJECT	DMV Delano
PROJECT NO.	2018-0187
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BY	GB
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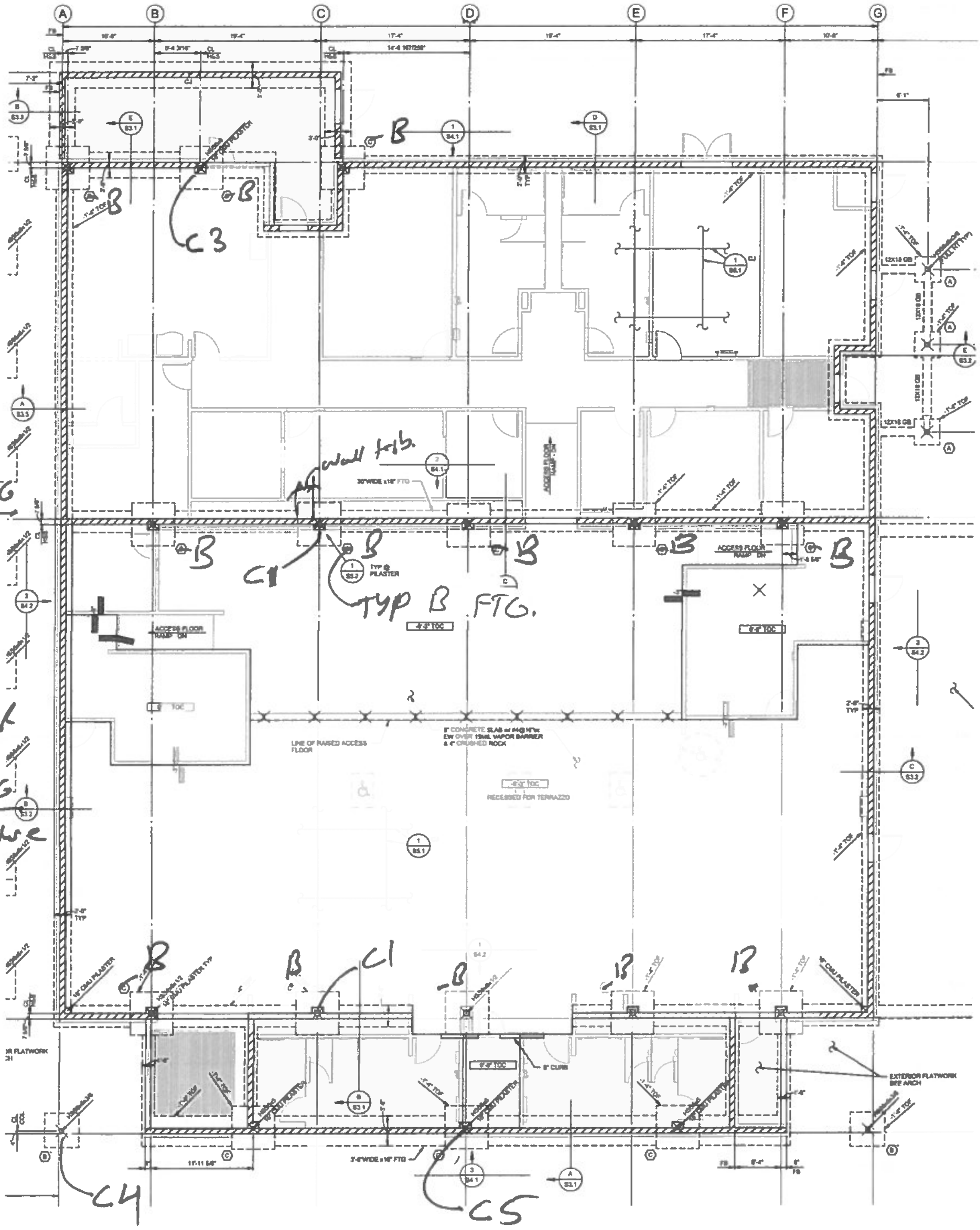
Foundation Design



C1: $DL = 14.2^k$

$L_r = 7.4^k$

WALL = 23'(80psf)(5')

$= 9.2^k$

$P = 31^k$

$SP = 31^k / 16\,sf$

$= 1940\,psf$

USE 4' □ FTG

C3: $P_u = 20^k$

$SP = 20^k / 9\,sf$

$= 2222\,psf$

Use 3' □ FTG

conservative due to factored Loads

C4: $P_u = 8^k$

$SP = 8^k / 4\,sf$

$= 2000\,psf$

Use 2' □ FTG

$P = 10.2^k$

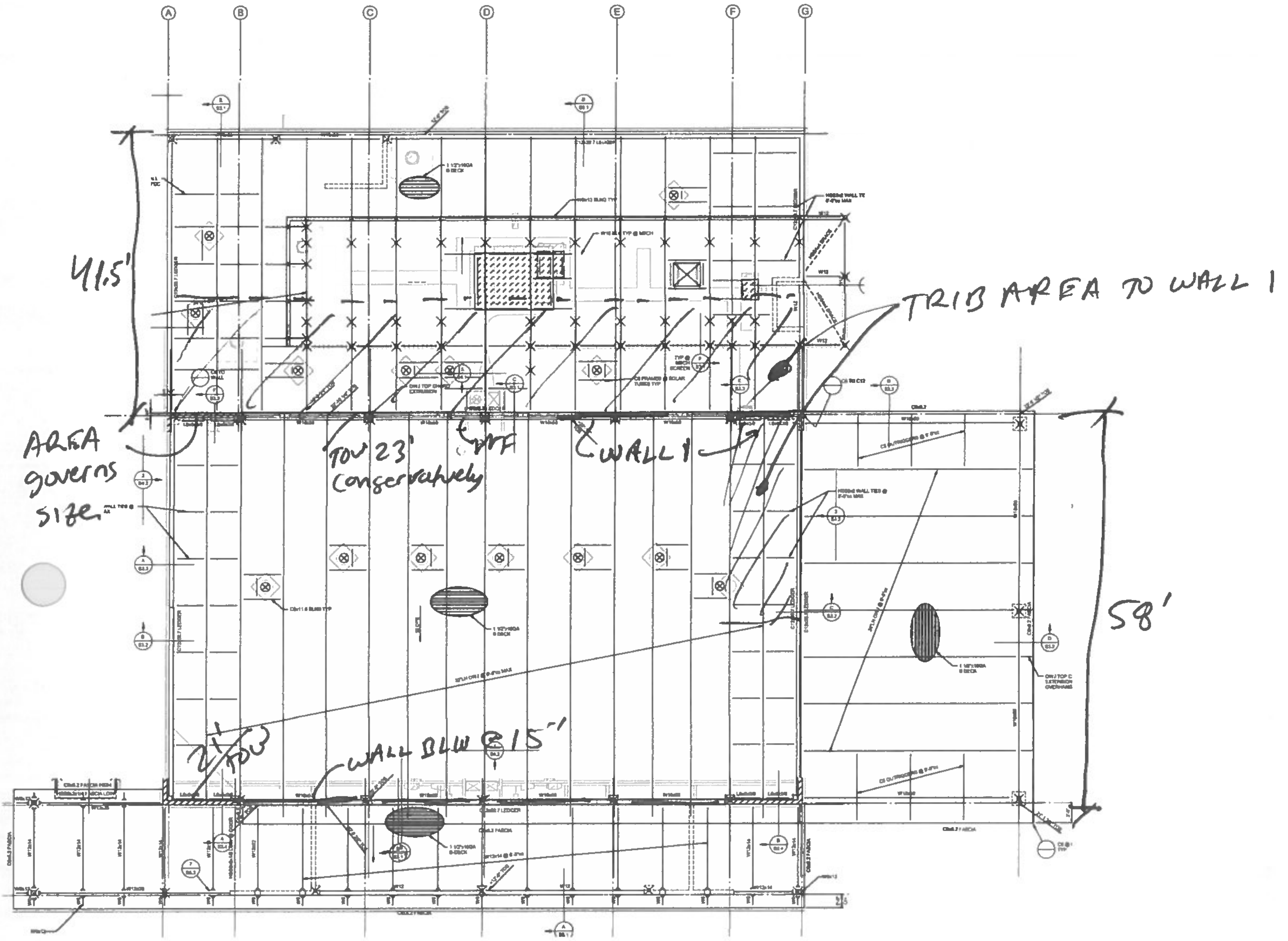
$SP = 10.2^k / 4\,sf = 2550\,psf$

use 2' □ FTG

BUEHLER

PROJECT	DMV Delano		
PROJECT NO.	2018-0187	DATE	11/11/2019
CLIENT	NLA	BY GB	PAGE NO. FD2

CONT. FTG DESIGN: WALL 1



WALL 1: $w_{DL} = 25\,psf(41.5'/2) + 85\,psf(23') = 2475\,plf$

$+25\,psf(58'/2) = 725\,plf$

Total $w_{DL} = 3200\,plf$

$w_{Lr} = 20\,psf(41.5'/2) + 20\,psf(58'/2) = 995\,plf$

$w_{DL+Lr} = 4200\,plf$

$SP = 4200\,plf/(2'\text{-}6") = 1680\,psf < 3000\,psf$

Use 2'-6" cont ftg

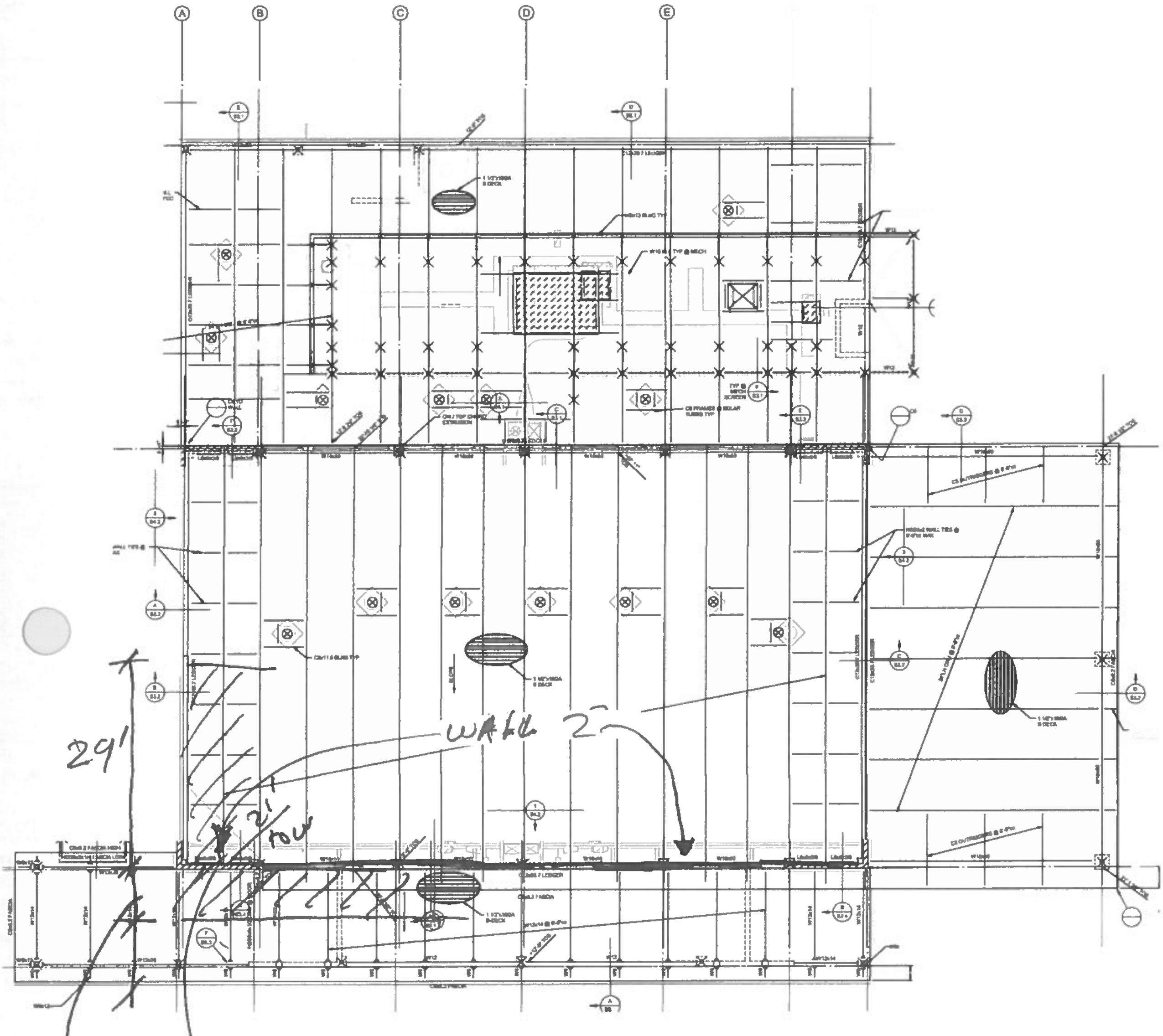
Seismic checked separately

PROJECT DMV Delano

PROJECT NO. 2018-0187 DATE 11/11/2019

CLIENT NLA BY GB PAGE NO. F03

CONT. FTG DESIGN : WALL 2



8'

Area governs min width: seismic checked seperately

$W_{DL} = 25psf(29' + 8') + 85psf(21') = 2710\ plf$

$W_{Lr} = 20(29'+8) = 588\ plf$

Per geotech report, disregard ftg self weight

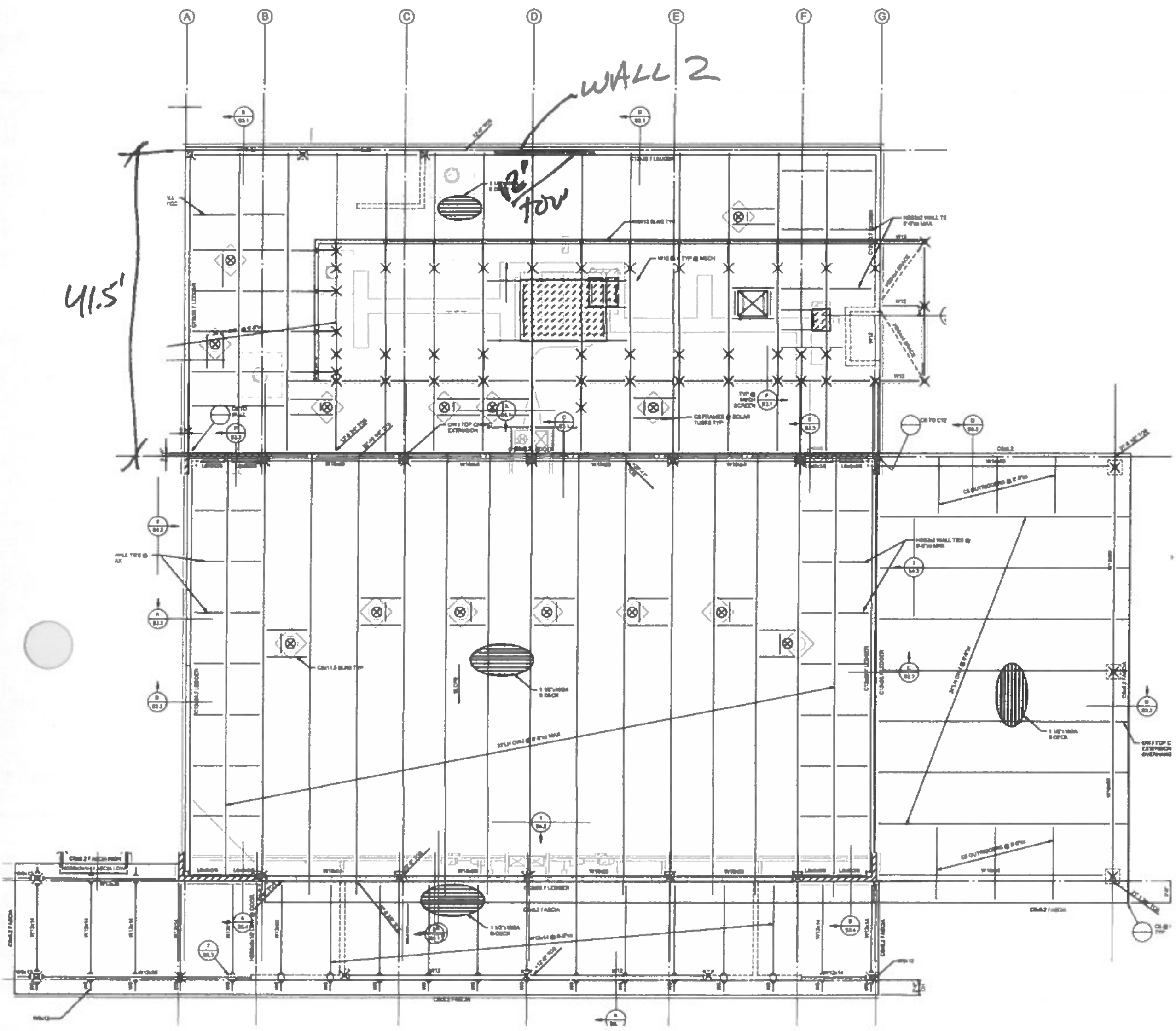
$W_{DL+Lr} = 3300\ plf$

$SP = 3300plf / 2'\text{-}6" = 1320\ psf$

Use 2'-6" min FTG

PROJECT	DMV Delano		
PROJECT NO.	2018-0187	DATE	11/11/2019
CLIENT	NLA	BY GB	PAGE NO. FD4

CONT. FTG DESIGN : WALL 3



WALL 2 : $w_{DL} = 25\,psf(41.5/2) + 100\,psf(12'\ Ht) = 1720\,plf$

$w_{Lr} = 20(41.5'/2) = 415\,plf$

Add $1000\#/6'\text{-}7''$ spacin $= 147\,plf$

Screen DL

Conservative to use for full length of wall

Add $3500\#/6.7' = 522\,plf$

Mech'l unit.

Total $= 2804\,plf$

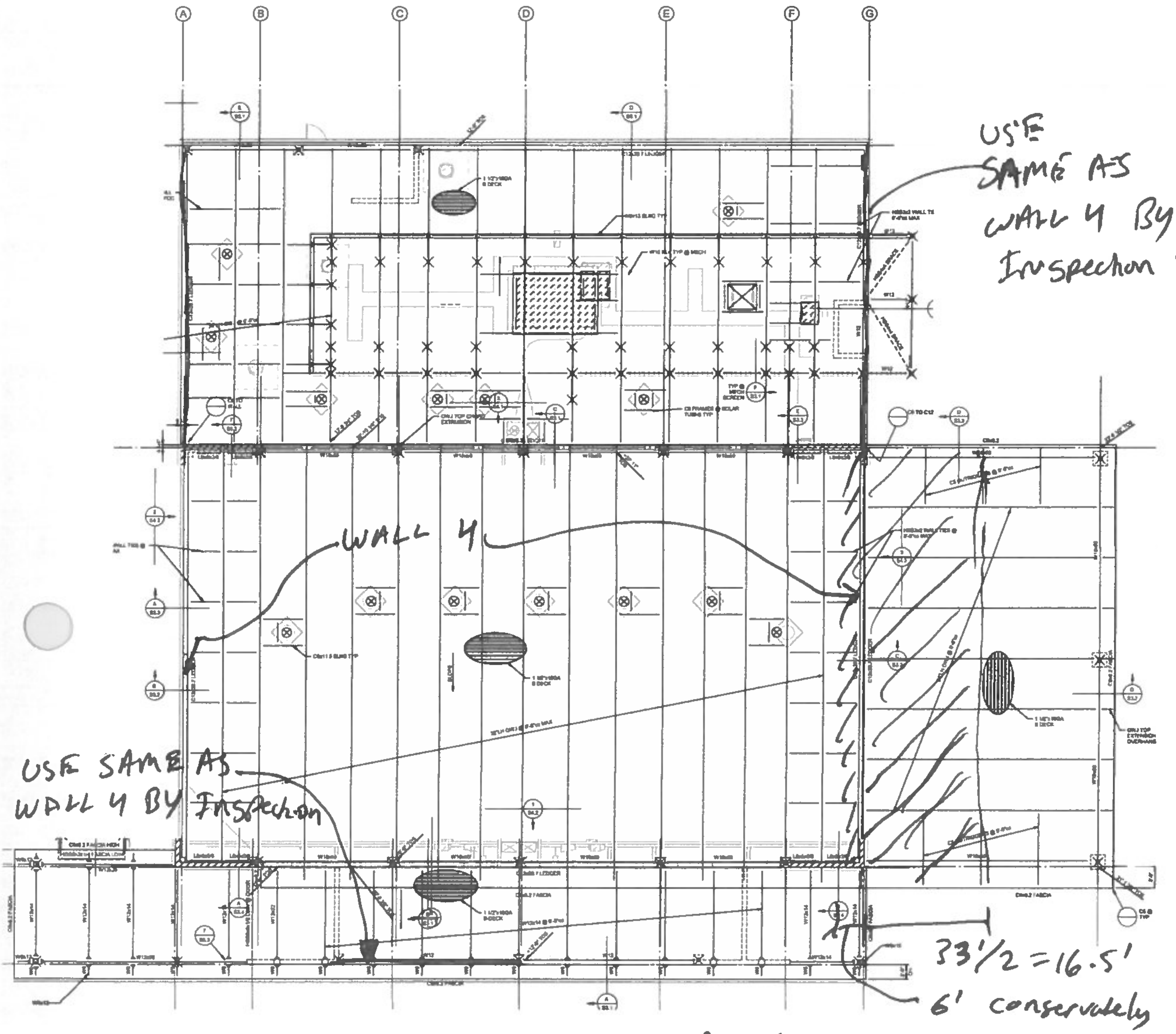
$SP = 2804\,plf/2' = 1402\,psf$

USE 2' STRIP FTG

BUEHLER

PROJECT	DMV Delano		
PROJECT NO.	2018-0187	DATE	11/11/2019
CLIENT	NLA	BY GB	PAGE NO. FD5

CONT. FTG DESIGN: WALL 4, WALL 5



$w_{DL} = 25\,psf(22.5') + 100\,psf(23'\ ht.) = 2863\,plf$

$w_{LL} = 20\,psf(22.5') = 450\,plf$

$w_{DL+LL} = 3313\,plf$

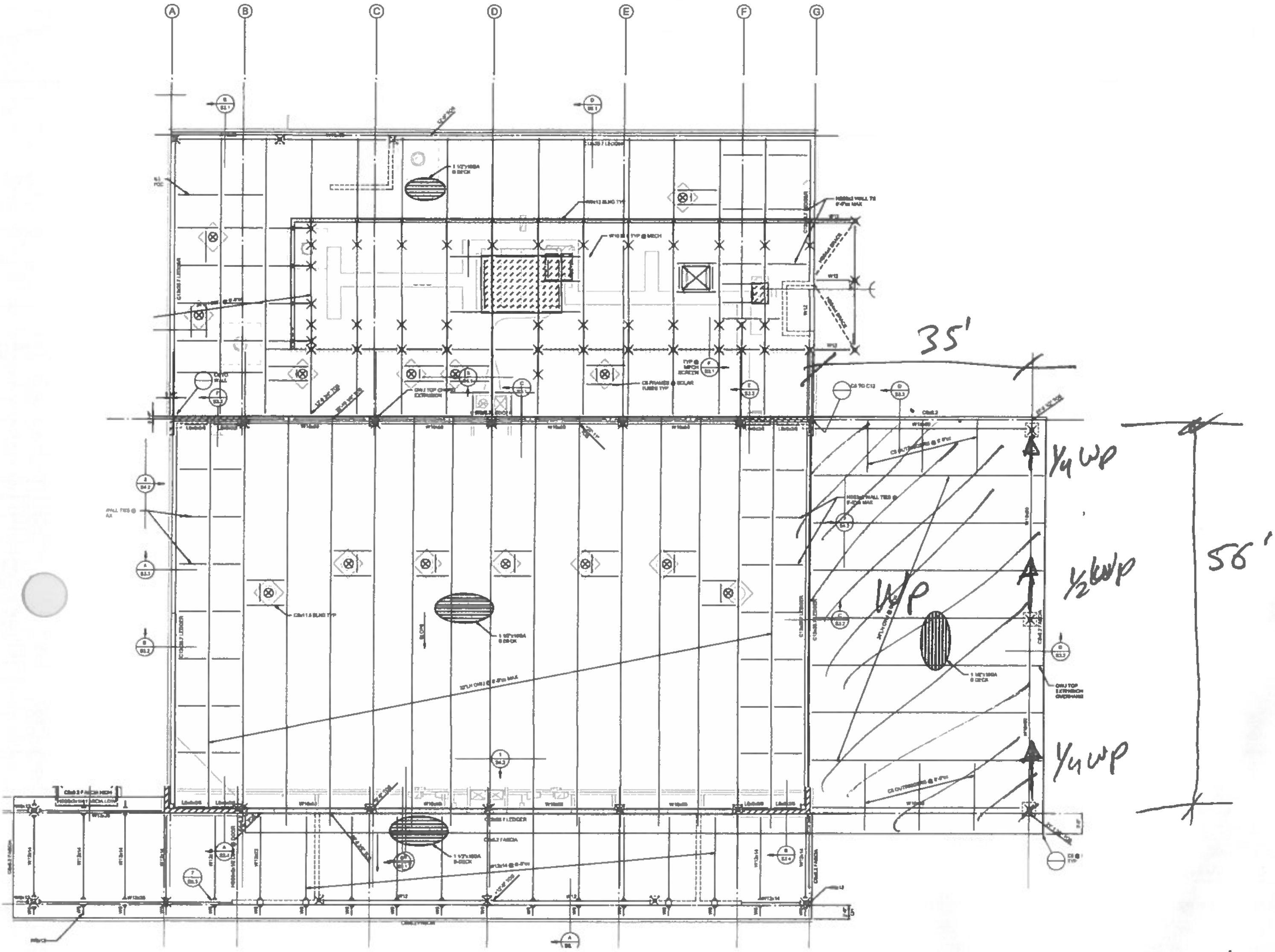
$SP = 3313/2' = 1660\,psf$

WALL 4
$\boxed{\text{USE 2' strip FTG}}$ min

BUEHLER

PROJECT	DMV Delano		
PROJECT NO.	2018-0187	DATE	11/11/2019
CLIENT	NLA	BY GB	PAGE NO. M1

CANTILEVER COLUMN'S @ CARPORT:



Seismic wt. of shaded area will be resolved into grid G, 2 & 3
→ To envelope design, design col's as cantilevers for some nominal load per ASCE chp. 13.

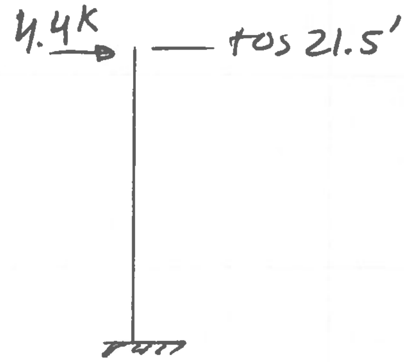
$$F_p = \frac{0.4(0.6g)(a_p = 1.0)}{R = 2.5 / I = 1.0}\left(1 + 2(0)\right) = 0.1\,W_p$$

$F_{p\,min} = 0.3(0.6)1.0\,W_p = 0.18\,W_p$ ← governs

$W_p = 25\,psf(35')(56') = 49^k$

$F_p = .18(49^k) = 8.8^k$ $\frac{1}{2}(8.8) = 4.4^k$ – Design middle col

– cantilever Col design



$M_{@base} = 4.4^k(21.5') = 94.6$ k-ft

$V_{base} = 4.4^k$

$M_{@base}$ w/ $\Omega_0 = 1.25$ = 118 k-ft

Drift not considered @ this location

USE HSS 8×8×1/2

$\phi M_n = 129$ k-ft.

– connection @ base.

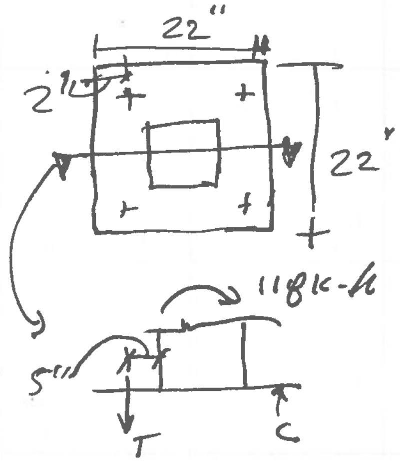
CJP @ base.

– BASE PL

$M = 79^k(5") = 395$ k-in

$\phi M_n = 0.9(50ksi)\left(\frac{22"(t)^2}{4}\right)$

w/ $t = 1\,3/8$ = 466 k-in



$T/C = \frac{118 \times 12"}{18"}$

$= 79^k$

USE 1 3/8" Gr. 50 Base PL

– Anchorage design, see following pg.

1½" gr. 55 F1554 Bolts w/ 24" embed.

M3

SIMPSON Strong-Tie® Anchor Designer™ Software
Version 2.8.7094.7

Company:		Date:	11/13/2019
Engineer:		Page:	1/6
Project:			
Address:			
Phone:			
E-mail:			

1.Project information

Customer company:
Customer contact name:
Customer e-mail:
Comment:

Project description:
Location:
Fastening description:

2. Input Data & Anchor Parameters

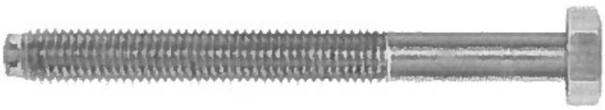
General
Design method:ACI 318-14
Units: Imperial units

Anchor Information:
Anchor type: Cast-in-place
Material: F1554 Grade 55
Diameter (inch): 1.500
Effective Embedment depth, h_{ef} (inch): 25.000
Anchor category: -
Anchor ductility: Yes
h_{min} (inch): 27.25
C_{min} (inch): 9.00
S_{min} (inch): 9.00

Base Material
Concrete: Normal-weight
Concrete thickness, h (inch): 30.00
State: Cracked
Compressive strength, f'_c (psi): 3000
$\Psi_{c,V}$: 1.0
Reinforcement condition: B tension, B shear
Supplemental reinforcement: Not applicable
Reinforcement provided at corners: No
Ignore concrete breakout in tension: No
Ignore concrete breakout in shear: No
Ignore 6do requirement: No
Build-up grout pad: Yes

Base Plate
Length x Width x Thickness (inch): 22.00 x 22.00 x 0.13

Recommended Anchor
Anchor Name: Heavy Hex Bolt - 1 1/2"Ø Heavy Hex Bolt, F1554 Gr. 55



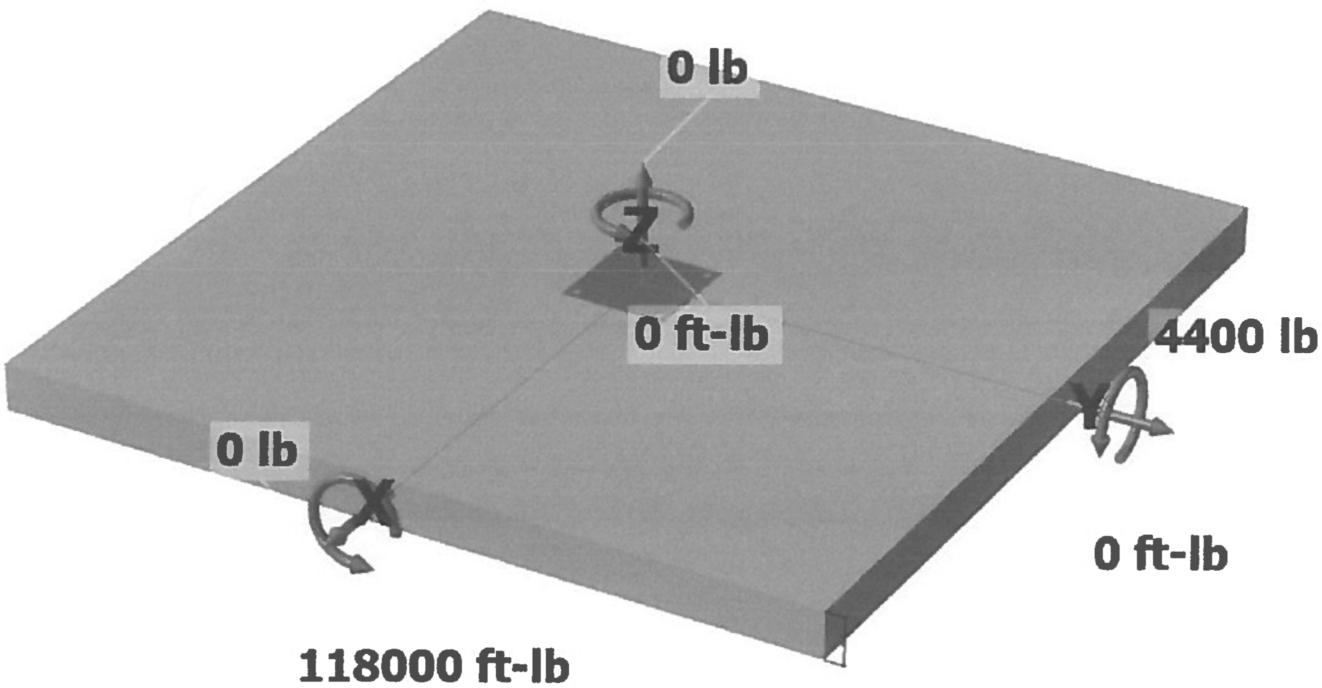
MH

Load and Geometry
Load factor source: ACI 318 Section 5.3
Load combination: not set
Seismic design: Yes
Anchors subjected to sustained tension: Not applicable
Ductility section for tension: 17.2.3.4.3 (d) is satisfied
Ductility section for shear: 17.2.3.5.3 (c) is satisfied
Ω_0 factor: not set
Apply entire shear load at front row: No
Anchors only resisting wind and/or seismic loads: Yes

Strength level loads:

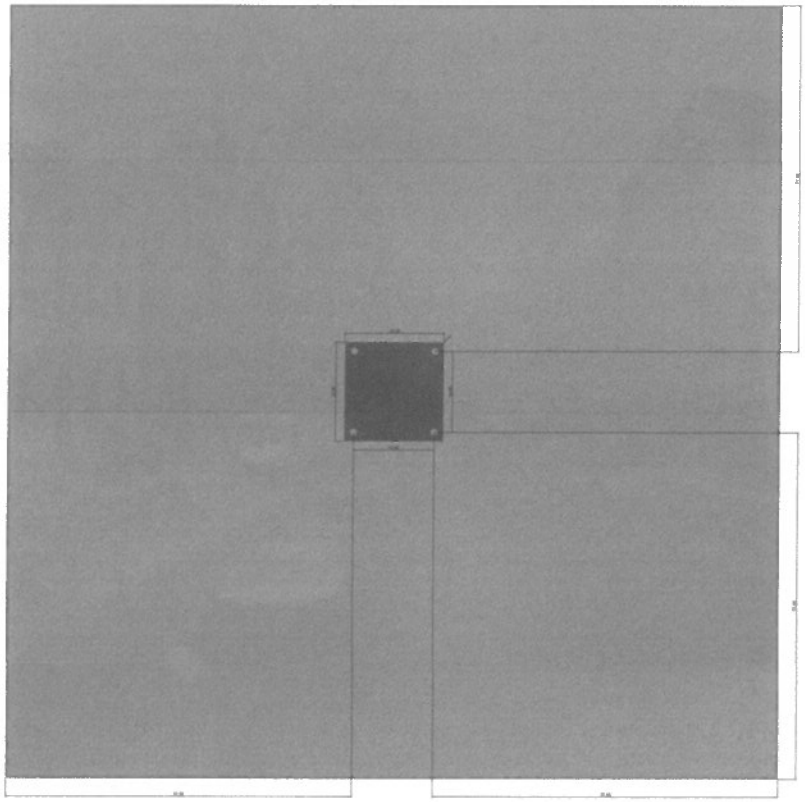
N_{ua} [lb]: 0
V_{uax} [lb]: 0
V_{uay} [lb]: 4400
M_{ux} [ft-lb]: 118000
M_{uy} [ft-lb]: 0
M_{uz} [ft-lb]: 0

<Figure 1>



MS

<Figure 2>



M6

SIMPSON Strong-Tie Anchor Designer™ Software
Version 2.8.7094.7

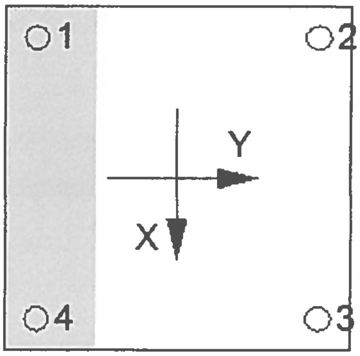
Company:		Date:	11/13/2019
Engineer:		Page:	4/6
Project:			
Address:			
Phone:			
E-mail:			

3. Resulting Anchor Forces

Anchor	Tension load, N_{ua} (lb)	Shear load x, V_{uax} (lb)	Shear load y, V_{uay} (lb)	Shear load combined, $\sqrt{(V_{uax})^2+(V_{uay})^2}$ (lb)
1	0.0	0.0	1100.0	1100.0
2	39189.5	0.0	1100.0	1100.0
3	39189.5	0.0	1100.0	1100.0
4	0.0	0.0	1100.0	1100.0
Sum	78379.0	0.0	4400.0	4400.0

Maximum concrete compression strain (‰): 0.28
Maximum concrete compression stress (psi): 1228
Resultant tension force (lb): 78379
Resultant compression force (lb): 78379
Eccentricity of resultant tension forces in x-axis, e'_{Nx} (inch): 0.00
Eccentricity of resultant tension forces in y-axis, e'_{Ny} (inch): 0.00
Eccentricity of resultant shear forces in x-axis, e'_{Vx} (inch): 0.00
Eccentricity of resultant shear forces in y-axis, e'_{Vy} (inch): 0.00

<Figure 3>



4. Steel Strength of Anchor in Tension (Sec. 17.4.1)

N_{sa} (lb)	ϕ	ϕN_{sa} (lb)
105375	0.75	79031

5. Concrete Breakout Strength of Anchor in Tension (Sec. 17.4.2)

$N_b = 16\lambda_a\sqrt{f'_c}h_{ef}^{5/3}$ (Eq. 17.4.2.2b)

λ_a	f'_c (psi)	h_{ef} (in)	N_b (lb)
1.00	3000	25.000	187318

$0.75\phi N_{cbg} = 0.75\phi\,(A_{Nc}/A_{Nco})\Psi_{ec,N}\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b$ (Sec. 17.3.1 & Eq. 17.4.2.1b)

A_{Nc} (in^2)	A_{Nco} (in^2)	$c_{a,min}$ (in)	$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	$0.75\phi N_{cbg}$ (lb)
6975.00	5625.00	77.00	1.000	1.000	1.00	1.000	187318	0.70	121944

6. Pullout Strength of Anchor in Tension (Sec. 17.4.3)

$0.75\phi N_{pn} = 0.75\phi\Psi_{c,P}N_p = 0.75\phi\Psi_{c,P}8A_{brg}f'_c$ (Sec. 17.3.1, Eq. 17.4.3.1 & 17.4.3.4)

$\Psi_{c,P}$	A_{brg} (in^2)	f'_c (psi)	ϕ	$0.75\phi N_{pn}$ (lb)
1.0	3.12	3000	0.70	39287

8. Steel Strength of Anchor in Shear (Sec. 17.5.1)

V_{sa} (lb)	ϕ_{grout}	ϕ	$\phi_{grout}\phi V_{sa}$ (lb)
63225	0.8	0.65	32877

9. Concrete Breakout Strength of Anchor in Shear (Sec. 17.5.2)

Shear perpendicular to edge in y-direction:

$V_{by} = \min|7(l_e/d_a)^{0.2}\sqrt{d_a}\lambda_a\sqrt{f'_c}c_{a1}^{1.5}; 9\lambda_a\sqrt{f'_c}c_{a1}^{1.5}|$ (Eq. 17.5.2.2a & Eq. 17.5.2.2b)

l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{a1} (in)	V_{by} (lb)
12.00	1.500	1.00	3000	51.33	181302

$\phi V_{cbgy} = \phi\ (A_{Vc}/A_{Vco})\Psi_{ec,V}\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_{by}$ (Sec. 17.3.1 & Eq. 17.5.2.1b)

A_{Vc} (in^2)	A_{Vco} (in^2)	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_{by} (lb)	ϕ	ϕV_{cbgy} (lb)
5160.00	11858.00	1.000	1.000	1.000	1.602	181302	0.70	88476

Shear parallel to edge in y-direction:

$V_{bx} = \min|7(l_e/d_a)^{0.2}\sqrt{d_a}\lambda_a\sqrt{f'_c}c_{a1}^{1.5}; 9\lambda_a\sqrt{f'_c}c_{a1}^{1.5}|$ (Eq. 17.5.2.2a & Eq. 17.5.2.2b)

l_e (in)	d_a (in)	λ_a	f'_c (psi)	c_{a1} (in)	V_{bx} (lb)
12.00	1.500	1.00	3000	51.33	181302

$\phi V_{cbgy} = \phi\ (2)(A_{Vc}/A_{Vco})\Psi_{ec,V}\Psi_{ed,V}\Psi_{c,V}\Psi_{h,V}V_{bx}$ (Sec. 17.3.1, 17.5.2.1(c) & Eq. 17.5.2.1b)

A_{Vc} (in^2)	A_{Vco} (in^2)	$\Psi_{ec,V}$	$\Psi_{ed,V}$	$\Psi_{c,V}$	$\Psi_{h,V}$	V_{bx} (lb)	ϕ	ϕV_{cbgy} (lb)
5160.00	11858.00	1.000	1.000	1.000	1.602	181302	0.70	176951

10. Concrete Pryout Strength of Anchor in Shear (Sec. 17.5.3)

$\phi V_{cpg} = \phi k_{cp}N_{cbg} = \phi k_{cp}(A_{Nc}/A_{Nco})\Psi_{ec,N}\Psi_{ed,N}\Psi_{c,N}\Psi_{cp,N}N_b$ (Sec. 17.3.1 & Eq. 17.5.3.1b)

k_{cp}	A_{Nc} (in^2)	A_{Nco} (in^2)	$\Psi_{ec,N}$	$\Psi_{ed,N}$	$\Psi_{c,N}$	$\Psi_{cp,N}$	N_b (lb)	ϕ	ϕV_{cpg} (lb)
2.0	8649.00	5625.00	1.000	1.000	1.000	1.000	187318	0.70	403229

11. Results

Interaction of Tensile and Shear Forces (Sec. 17.6.)

Tension	Factored Load, N_{ua} (lb)	Design Strength, ϕN_n (lb)	Ratio	Status
Steel	39190	79031	0.50	Pass
Concrete breakout	78379	121944	0.64	Pass
Pullout	**39190**	**39287**	**1.00**	**Pass (Governs)**

Shear	Factored Load, V_{ua} (lb)	Design Strength, ϕV_n (lb)	Ratio	Status
Steel	1100	32877	0.03	Pass
T Concrete breakout y+	**4400**	**88476**	**0.05**	**Pass (Governs)**
|| Concrete breakout x-	**2200**	**176951**	**0.01**	**Pass (Governs)**
Pryout	4400	403229	0.01	Pass

Interaction check	$N_{ua}/\phi N_n$	$V_{ua}/\phi V_n$	Combined Ratio	Permissible	Status
Sec. 17.6..1	1.00	0.00	99.8%	1.0	Pass

1 1/2"Ø Heavy Hex Bolt, F1554 Gr. 55 with hef = 25.000 inch meets the selected design criteria.

12. Warnings

- Per designer input, ductility requirements for tension have been determined to be satisfied – designer to verify.

- Per designer input, ductility requirements for shear have been determined to be satisfied – designer to verify.

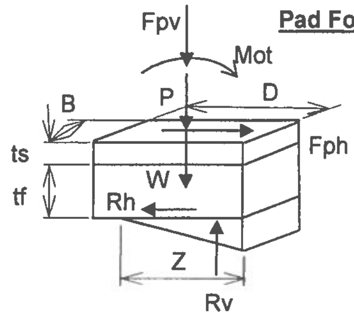
- Designer must exercise own judgement to determine if this design is suitable.

BUEHLER

PROJECT		
PROJECT NO.		DATE
CLIENT	BY	SHEET NO. M9

PAD FOOTING WITH ONE WAY OVERTURNING MOMENT

Pad Footing at Carport



FOOTING GEOMETRY AND LOADING

B =	84 in
D =	84 in
tf =	30 in
ts =	16 in

ρ soil =	110 pcf
P_{DL} =	12,250 lb
P_{LL} =	9,800 lb

A = B * D / 144 =	49 sft
kern = D / 6 =	14 in
S = B * D^2 / 6 / 1728 =	57 cft
Vol = A * tf / 12 =	123 cft

Wf = Vol * 150 pcf =	18,375 lb	
Ws = A * ts / 12 * ρ soil =	7,187 lb	
Rv_{DL} = P_{DL} + Wf + Ws =	37,812 lb	DL
qf = (Wf + Ws) / A =	522 psf	footing self

SEISMIC LOAD - Alternate ASD Load Combinations, CBC-2019 Section 1605.3.2

(1.0E) Fph =	4,400 lb
(1.0E) Fpv =	1,470 lb
(1.0E) Mot =	82,600 lb-ft

Eqn 16-21
(D + L + E/1.4)

Rv = Rv_{DL} + P_{LL} + Fpv =	48,641 lb	
e = Mot * 12 / (1.4 * Rv) =	15 in	outside kern
Z = 3 *(D / 2 - e) =	82 in	
qmax = 2 * Rv / (B * Z) =	2026 psf	

Eqn 16-22
(0.9D - E/1.4)

Rv = .9 * Rv_{DL} -Fpv =	33,002 lb	
e = Mot * 12 / (1.4 * Rv) =	21 in	outside kern
Z = 3 *(D / 2 - e) =	62 in	
qmax = 2 * Rv / (B * Z) =	1836 psf	

WIND LOAD - Alternate ASD Load Combinations, CBC-2019 Section 1605.3.2: **Does not Govern**

(1.0W) Fph =	0 lb
(1.0W) Fpv =	0 lb
(1.0W) Mot =	0 lb-ft

Eqn 16-18
(D + L + 0.6*1.3*W)

Rv = Rv_{DL} + P_{LL} + 0.6 * 1.3 * Fpv =	47,612 lb	
e = 0.6 * 1.3 * Mot * 12 / Rv =	0 in	inside kern
Z = D =	84 in	
qmax = Rv / A + 1.3 * Mot / S =	972 psf	

Eqn 16-18
(2/3*D - 0.6*1.3*W)

Rv = 2 / 3 * Rv_{DL} - 0.6 * 1.3 * Fpv =	25,208 lb	
e = 0 .6 * 1.3 * Mot *12 / Rv =	0 in	inside kern
Z = D =	84 in	
qmax = Rv / A + 1.3 * Mot / S =	514 psf	

RESULTING SOIL PRESSURES - ASD

Use: 84 in x 84 in x 30 in pad

	Capacity	Demand	D/C	Check
Dead	3000 psf	250 psf	**0.08**	OK
Total	3000 psf	450 psf	**0.15**	OK
Seismic	4000 psf	2026 psf	**0.51**	OK
Wind	4000 psf	972 psf	**0.24**	OK

MECHANICAL SCREEN WALL DESIGN

Screen Geometry

Ht = 8 ft OH = 3.5ft OS = 1.5 ft

Post_spacing = 7 ft max

Screen_Weight = 10psf (3psf deck plus, 12plf 4" std pipe at 8.5ft, 12plf 4" std pipe brace at 8.5ft,)

SEISMIC LOADS

Unit Weight W_p = 10psf

Seismic Load Parameters (ASCE 7-10, Ch 13, Tables 13.5-1)

a_p = **2.5** R_p = **2.5** I_p = **1.0**

S_{ds} = **0.589** z = **1.0** ft h = **1.0** ft Ω_0 =**2**

Seismic Forces

$F_{pcalc} = 0.4^* a_p{}^* S_{ds}{}^* I_p{}^*(1+2^*(z/h)) / R_p =$ **0.707** $* W_p$

$F_{pmin} = 0.3^* S_{ds}{}^* I_p =$ **0.2** $* W_p$ $F_{pmax} = 1.6^* S_{ds}{}^* I_p =$ **0.9** $* W_p$

Controlling Horizontal Force F_p = **0.71** $* W_p$

$Fp = F_p * W_p =$ **7.1** psf

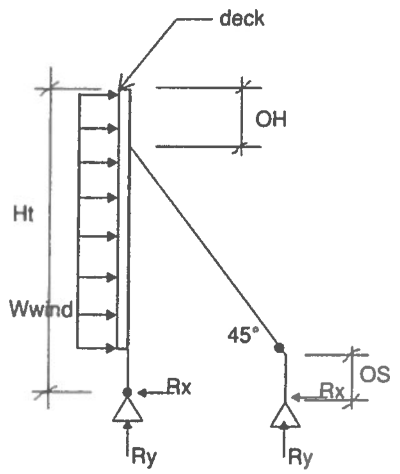
WIND LOADS SHALL BE DETERMINED BY ASCE 7-16

H = 30ft	height of attachment
V = 95	basic wind speed
EXP=C	exposure type ... section 26.7.3
Kz = 1.0	velocity pressure exposure coeff. ... table 29.3-1
Kzt = 1	topographic factor ... section 26.8.2
Kd = 0.9	wind directionality factor ... table 26.6-1
qh = 0.00256*Kz*Kzt*Kd*V^2 * 1 psf =**20.8** psf	velocity pressure ... eqn 29.3-1

ROOFTOP STRUCTURES ON BUILDINGS W/ H<60FT

GCr=1.9	EQN 29.5-2 (Vertical projection normal to wind direction is small)
Fwh = qh*GCr =**39.5** psf	design wind lateral force
$F_p = F_p * W_p =$**7.1** psf	design seismic lateral force

If(Fwh > F_p, "Wind Governs Of course", "Seismic") = **"Wind Governs Of course"**



Wall Panel Design

Panel spans vertically

Panel Type: 24 ga corrigated metal panel

SECTION PROPERTIES								ALLOWABLE UNIFORM LOADS, psf For various fastener spacings											
Ga	Width in	Yield ksi	Weight psf	Top In Compression		Bottom In Compression		Inward Load						Outward Load					
				Ixx in⁴/ft	Sxx in³/ft	Ixx in⁴/ft	Sxx in³/ft	5'	6'	7'	8'	9'	10'	5'	6'	7'	8'	9'	10'
26	36	50	0.88	0.0653	0.0678	0.0667	0.0748	65	46	34	26	21	16	59	42	31	24	19	16
24	36	50	1.15	0.0977	0.1082	0.0953	0.1122	101	71	52	40	29	21	98	69	51	39	29	21
22	36	50	1.51	0.1367	0.1562	0.1333	0.1623	148	104	76	54	38	28	143	100	74	54	38	28
20	36	33	1.85	0.1867	0.2227	0.1833	0.2320	140	98	72	55	44	34	135	94	69	53	42	34
18	36	33	2.45	0.2633	0.3230	0.2567	0.3253	196	137	101	78	61	45	195	136	100	77	61	45

1. Theoretical section properties have been calculated per AISI 2012 'North American Specification for the Design of Cold-Formed Steel Structural Members'. Ixx and Sxx are effective section properties for deflection and bending.
2. Allowable load is calculated in accordance with AISI 2012 specifications considering bending, shear, combined bending and shear and deflection. Allowable load considers the 3 or more equal spans condition. Allowable load does not address web crippling, fasteners, support material or load testing. Panel weight is not considered.
3. Deflection consideration is limited by a maximum deflection ratio of L/180 of span.
4. Allowable loads do not include a 1/3 stress increase for wind.

Fwh_ASD = 0.6 * Fwh = **23.7** psf

Span2 = (Ht – OS)/2 = **3.3** ft Span = 7 ft S_minus = 0.1082 in^3 S_plus = 0.1122 in^3

M_deck = Fwh_ASD * 1 ft * Span * Span * 0.125 = **145.2** lbs_ft

Allowable_Inward_Load = 50ksi * S_plus = **467.5** lbs_ft

Allowable_outward_Load = 50ksi * S_minus = **450.8** lbs_ft

If(and(M_deck < Allowable_Inward_Load, M_deck < Allowable_outward_Load), "Deck Span OK", "Deck NG") = **"Deck Span OK"**

Deflt_deck = (5* 0.75*Fwh_ASD*1ft* Span^4)/(384*29000ksi*0.0953in^4) = **0.35** in

Span/Deflt_deck = **241.7**

Horizontal Spaning Member

W_wind = Fwh * Span = **237.0** plf

W_DL = Screen_Weight * Span = **60.0** plf

Mu = (W_wind + 1.2 * W_DL) * Post_spacing^2 * 0.125 = **1.9** kip_ft

V = (W_wind + 1.2 * W_DL) * Post_spacing * 0.5 = **1.1** kips

Ok for Shear by Inspection

HSS 6x3x1/4

A = **3.84** in^2

I_x = **17.00** in^4

Z_x = **7.19** in^3

E = 29000 ksi

b_f = **3.00** in

I_y = **5.70** in^4

r_x = **2.10** in

r_y = **1.22** in

Fy = 42 ksi

J = **14.20** in^4

t = **0.23** in

b_t = b_f/t =**12.876**

Flexural Load

λ_p = 1.2 * sqrt(E/Fy) = **31.5**
λ_r = 1.4 * sqrt(E/Fy) = **36.8**

Compactness = if(b_t < λ_p , "Compact", "Non-Compact") = **"Compact"**
Slenderness = if(b_t < λ_r , "Compact", "Slender") = **"Compact"**

Mp = Fy * Z_x = **302** kip_in

ϕMn = 0.9 * Mp = **272** kip_in
DCM = Mu / ϕMn = **0.08** Flexural demand-capacity ratio

Deflection

Δ = (5 * (W_wind + 0.6* W_DL)* Post_spacing^4) / (384 * E * I_x) = **0.03** in

Δ_allowable = Post_spacing / 180 = **0.47** in

DC_Δ = Δ / Δ_allowable = **0.06** Deflection demand-capacity ratio

<u>Vertical Spaning Member</u>

Mu = (W_wind + 1.2 * W_DL) * Post_spacing * OH = **7.6** kip_ft Moment at overhang governs my inspection

HSS 6x3x1/4

A = **3.84** in^2 r_x = **2.10** in
I_x = **17.00** in^4 r_y = **1.22** in
Z_x = **7.19** in^3 Fy = 42 ksi
E = 29000 ksi J = **14.20** in^4
b_f = **3.00** in t = **0.23** in
I_y = **5.70** in^4 b_t = b_f/t =**12.876**

Flexural Load

λ_p = 1.2 * sqrt(E/Fy) = **31.5**
λ_r = 1.4 * sqrt(E/Fy) = **36.8**

Compactness = if(b_t < λ_p , "Compact", "Non-Compact") = **"Compact"**
Slenderness = if(b_t < λ_r , "Compact", "Slender") = **"Compact"**

Mp = Fy * Z_x = **302** kip_in

ϕMn = 0.9 * Mp = **272** kip_in
DCM = Mu / ϕMn = **0.33** Flexural demand-capacity ratio

BUEHLER	JOB		
	JOB NO.	DATE	11/11/2019
	CLIENT	BY GB	PAGE NO. 4 M13

Connection to Structure

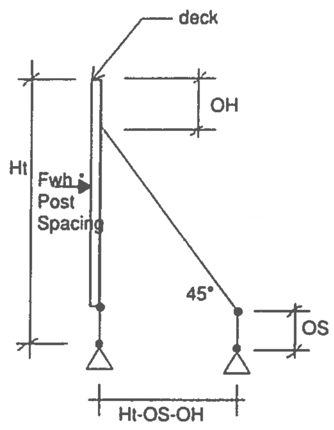
Calculate global reactions

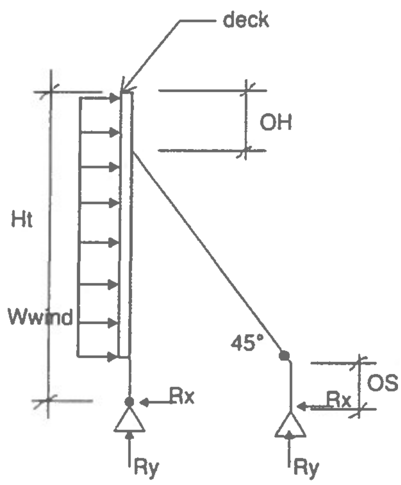
Ry = (Fwh * Post_spacing * Ht * Ht / 2) / (Ht – OS-OH) = **2949.9** lbs

Rx = (Fwh * Post_spacing * Ht)/ 2 = **1106.2** lbs

Ry_DL = (Screen_Weight * Ht * Post_spacing) = **560.0** lbs

Indicate reaction on drawings. OWJ to be designed by others



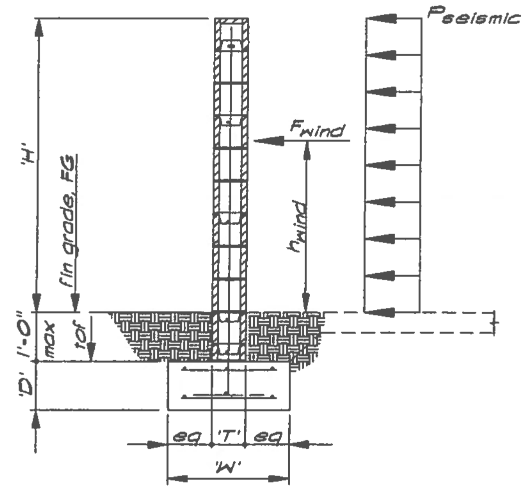


PROJECT	DMV Delano				
PROJECT NO.	2018-0187			DATE	11/12/2019
CLIENT	NLA	BY	GB	SHEET NO.	MH

CANTILEVERED MASONRY SCREEN WALL DESIGN

ASCE 7-16, TMS 402/602-16

Typical Screen Wall



Screen Wall Geometry

ASCE 7-16, CH 29

height of wall, h = 8.00 ft
vertical dim. of wall, s = 8.00 ft
clearance ratio s/h = 1.0
wall length, B = 11.0 ft
Aspect Ratio, B/s= 1.38
FG to tof = 1.0 ft

Return Wall? No

Wind Design Inputs

ASCE 7-16, CH 26 & 29

Occupancy Category = II
Exposure Category = C (26.7.3)
Exposure Coeff., K_z = 0.85 (26.10.1)
Topo. Factor, K_{zt} = 1.0 (26.8.2)
Dir. Factor, K_d = 0.85 (26.6.)
Ground Elev Fact, K_e = 1.0 (26.9.)
basic wind speed, V = 95 mph (Figures 26.5-1A, B, C)

$C_{f\,case\,A/B}$ = 1.43 Case A and B (29.3-1)
$C_{f\,case\,C}$ = 2.25 Case C (29.3-1)
G = 0.85 gust-effect factor (26.11.1)
A_s = 8.0 sq ft based on 1'-0" strip

q_h = 16.7 psf $= 0.00256\ K_z K_{zt} K_d K_e V^2\ I\ (lb/ft^2)$ [Eqn 26.10-1]

Wind Design Force

ASCE 7-16, 29.3

$F = q_h G C_f A_s$ (lb) [eqn: 29.3-1]

Case A & B:
f = **20.3 psf** Case A and B (typical wall)
F = **162.5 lb** Case A and B (typical wall)

Case C:
reduction factor, C = **0.8** Table 29.3-1, ftnote 4
f = **25.5 psf** Case C (end wall)
F = **204.3 lb** Case C (end wall)

Seismic Design Force

ASCE 7-16, 13.3.1

S_{ds} = 0.589
actual t_{wall} = 7.625 in.
W_p(psf) = 80 psf
I_p = 1.0 (13.1.3)
a_p = 2.50 (Table 13.5-1, cantilever elements)
R_p = 2.50 (Table 13.5-1, cantilever elements)

$F_p = [0.4a_p(S_{ds})(I_p)(1+2(z/h))]/(R_p/I_p)$ W_p = 0.236 Wp Eqn: 13.3-1 **Governs**
$F_{p\,max} = 1.6\,(S_{ds})(I_p)$ W_p = 0.942 Wp Eqn: 13.3-2
$F_{p\,min} = 0.3(S_{ds})(I_p)$ W_p = 0.177 Wp Eqn: 13.3-3

F_p = 0.236 Wp (@ strength)
F_p = **18.8 psf** (@ strength)

Force Summary

F_{wind} = **162.5 lb** Case A and B (typical wall) (1.0W)
F_{wind} = **204.3 lb** Case C (end wall) (1.0W)
h_{wind} = **4.40 ft** (distance from Finshed Grade, FG)
$F_{p\,seismic}$ = **18.8 psf** (1.0E)

CASE C GOVERNS

PROJECT	DMV Delano		
PROJECT NO	2018-0187	DATE	11/12/2019
CLIENT	NLA	BY GB	SHEET NO M15

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CANTILEVERED MASONRY SCREEN WALL DESIGN

Typical Screen Wall

CMU Wall Design Criteria:

f'_m = 2000 psi	bar loc: centered	$\phi_{bending}$ = 0.9	# of bars = 1	A_s [in^2/ft] = 0.15
f_r = 133 psi		E_s [psi] = 29000000	bar size = #4	ρ = 0.0033
actual t_{wall} = 7.625 in.	d = 3.813 in.	E_m [psi] = 1800000	bar spc'g = 16 cc	ε_{mu} = 0.0025
	b = 12.0 in.	n = 16.1	f_y = 60.0 ksi	ε_s = 0.0138
	a = 0.47 in.	ρ*n= 0.05282	ε_y = 0.0021	ρ_{max} = 0.0095
	c = 0.59 in.			

CMU Wall Bending @ Base (Strength Design - TMS 402-16)

Case C (1.0W) P = 204.3 lb   ht to force from FG = 4.40 ft

				Check if Mn > 1.3*Mcr
Load case: 1.2*D + 1.0*W		ϕM_n = 29.0 k-in.		S_n = 116.3 in.^3
$M_{max\ @\ base\ wind}$ = 13.2 k-in.	(factored)	M_u = 13.2 k-in.	(wind)	M_{cr} = 15.5 k-in.
Load case: 1.2*D + 1.0*E		D/C = 0.46		M_n / M_{cr} = 2.1
$M_{max\ @\ base\ seismic}$ = 9.16 k-in.	(factored)	Check: OK		Check: OK

Ftg Stability/Bearing Pressure Design (ASD Design-Alternative Basic Load Combinations - CBC 1605.3.2)

footing width, W = 2.50 ft			**DL Weights**
footing thickness, D = 1.00 ft	wall wt / ft = 80.0 psf		Wall = 720 lb
W/3 = 0.83 ft			Footing = 375 lb
Allowable Soil Pressure:			Soil = 205 lb
D + L = 3000 psf	seismic =	4000 psf	Total = 1300 lb

Case C Wind Design	Seismic Design	
DL factor = 0.66	DL factor = 0.90	Ev=0 (CBC 1605.3.2)
ω = 1.0		
0.6*OTM = 785 ft-lb	OTM/1.4 = 666 ft-lb	
0.66*DLRM = 1073 ft-lb	0.9*DLRM 1463 ft-lb	
a = 0.34 ft	a = 0.68 ft	
location = Outside Third	location = Outside Third	
SP_{max} = 1704 psf	SP_{max} = 1146 psf	
Check: OK	Check: OK	

Concrete Footing Design (Strength Design)

$f'_{c\ ftg}$ = 3.000 ksi	bar size = #5	ACI Table 24.4.3.2	d = 8.7 in.
f_y = 60 ksi	bar spc'g = 12 cc	reinf ratio, ρ > 0.0018	a = 0.61 in.
ϕ = 0.9	A_s [in^2/ft] = 0.31	ρ = 0.0022	ϕM_n = 11.7 k-ft.
1'-0" strip, b = 1.0 ft	bars t&b: no	Min reinf: OK	M_u = 1.2 k-ft.
			D/C = 0.10
			Check: OK

Conclusions / Summary

Wall	Footing
f'_m [psi] = 2000	$f'_{c\ ftg}$ = 3.0 ksi
t [in.] = 7.625	FG to tof = 1.0 ft
bar loc: centered	footing width, W = 2.5 ft
d [in] = 3.813 in.	footing thickness, D = 1.0 ft
bar size = #4	transverse reinf = #5
bar spc'g = 16 cc	bar spc'g = 12 cc
	bars t&b: no

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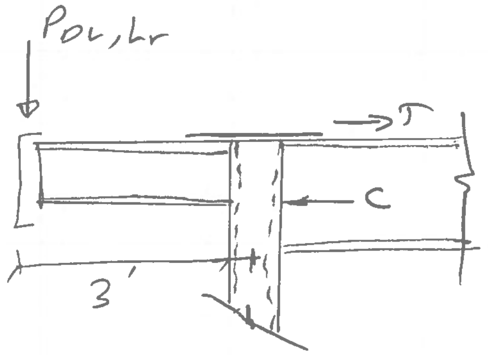
PROJECT ______

PROJECT NO. ______ DATE ______

CLIENT ______ BY ______ SHEET NO. M16

2-WAY MOMENT CONNE @ ENTRY

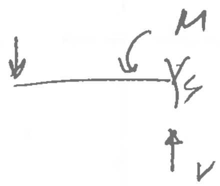


$P_{DL} = 25psf(2')(8') = 400\#$

$P_{Lr} = 20(2')(8') = 320\#$

$M_u = (1.2(400)+1.6(320))(3')$
$= 2976\#\text{-ft}$

$V_u = 992\#$



$\Delta = 992\#(3' \times 12")^3 / 3(29000)(I = 22.1)$
$= 0.024" < L/240 = 0.15"$ ok.

$T/C = 2976\#\text{-ft} / 6" = 5952\#$

$\phi T_n = 0.9(36)(3")(0.25") = 24^k$

$\phi P_n =$ OK by inspection
reduce unbraced length by
welding all around.

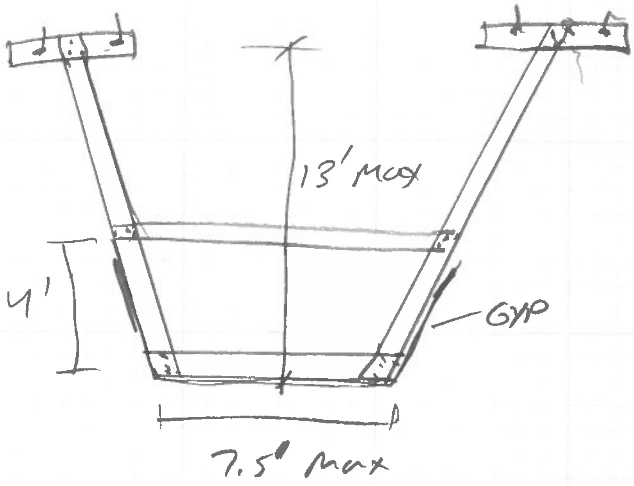
USE 1/4" PL x 3" WIDE
w/ 2" MIN weld.

PROJECT DMV Delano

PROJECT NO. 	DATE

CLIENT 	BY 	SHEET NO. M17

SOFFIT FRMG



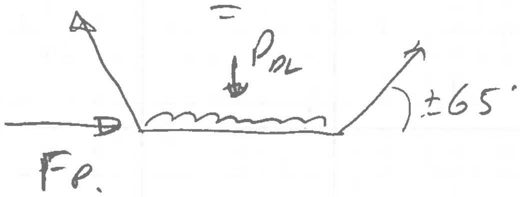
$P_{DL} = 5psf(4' \times 2 + 7.5')(1.34')$
$= 104^{\#}/2\text{ side} = 52^{\#}$

$P_{DL\text{-}vertical} = \dfrac{52^{\#}}{\sin 65} = 58^{\#}$

Hanger & Deck OK by inspection.

$F_p = \dfrac{.4(1)(0.6)}{2.5/1.0}(1+2)$
$= 0.29 W_p > F_{p\,min} = 0.18$

$F_p = .29(5psf)(1.34')(8' + 7.5')$
$= 30^{\#}$ / frame.



USE 3 5/8" x 18ga studs typ. Frame OK by inspection.

PROJECT DMV Delano

PROJECT NO. DATE

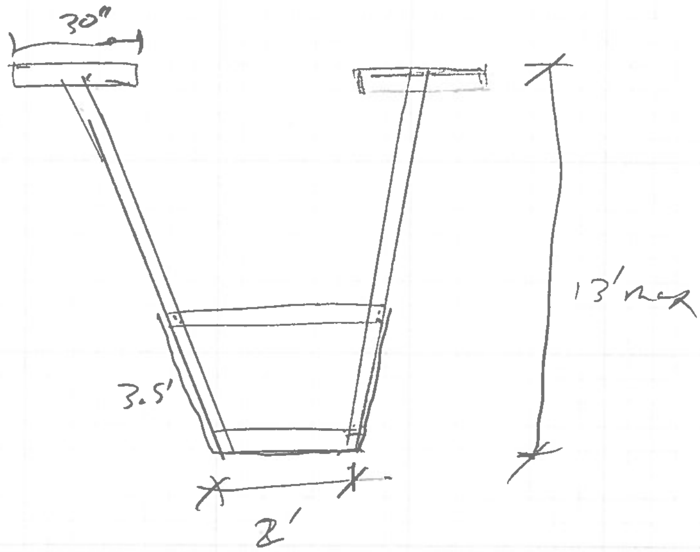
CLIENT BY SHEET NO. M18

SOFFIT FRAMING

hangers @ 48" cc

$P_{DL} = 5psf(3.5' \times 2 + 2')(4')$

$= 180\# / 2 \text{ sides} = 90\#$



Deck Allowable Load: 63 psf

−20 psf Lr

−10 psf mis

33 psf.

Using 30" Long hanger & 48" spacing Area = 10 sf:

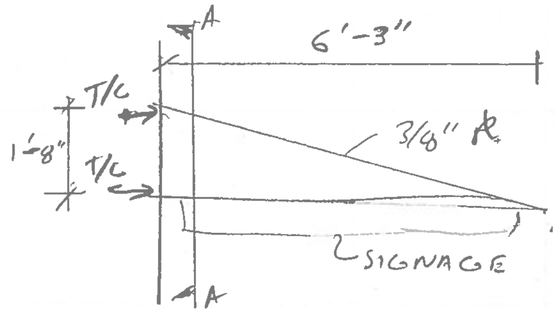
33 (10 sf) = 330# allowable

Deck ok.

3 5/8" x 18 ga dsp framing

o/k by inspection

SIGN ATTACHMENT



VERTICAL LOADS

$$wt. = \frac{480pcf(.375" \times 6.25' \times 20")}{12^3}$$

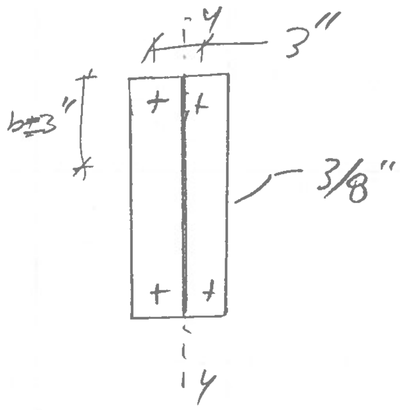
$= 156\#$ + signage – USE $\underline{200\#}$

$$T/C = \underbrace{1.4D(6.25'/2)/20"}_{\text{conservative}} = 1.4(200\#)(6.25'/2)/20"$$

$= 525\# / 2$ bolts $= 262\#$

HORIZONTAL WIND LOADS

$$P_{wall\ Zone\ 5} = \underset{windward}{18\ psf} + \underset{leeward}{19.5\ psf} = 37.5\ psf$$



$$M_{y-y} = 37.5\ psf\ (20"/12")(6.25'/2)$$

$= 195\ \#\text{-ft} = 2344\ \#\text{-in}$

$$T/C_{BOLTS} = \frac{2344\ \#\text{-in}}{2 bolts(3")} = 390\#$$

CHECK PL Bending

$$\phi M_n = 0.9(36ksi)(3" \times 0.375^2/4) = 3.4\ k\text{-in} > 2.3\ k\text{-in}$$

OK.

CHECK EXPANSION ANCHORS

$$V = 200\#/4 = 50\#$$

$$T = 262\# + 390\# = 653\#$$

M20

Hilti PROFIS Engineering 3.0.56

www.hilti.com

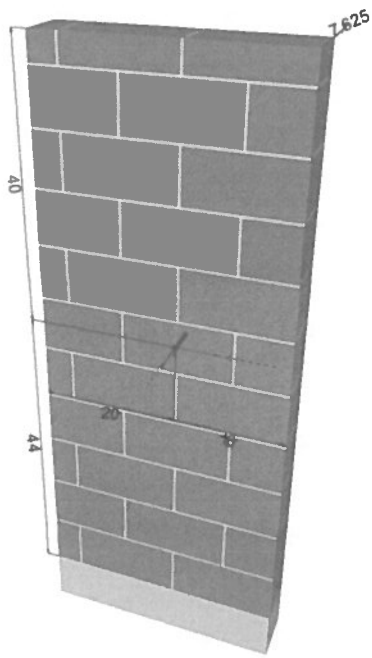
Company:		Page:	1
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Masonry - Feb 6, 2020	Date:	2/6/2020
Fastening point:			

Specifier's comments:

1 Input data

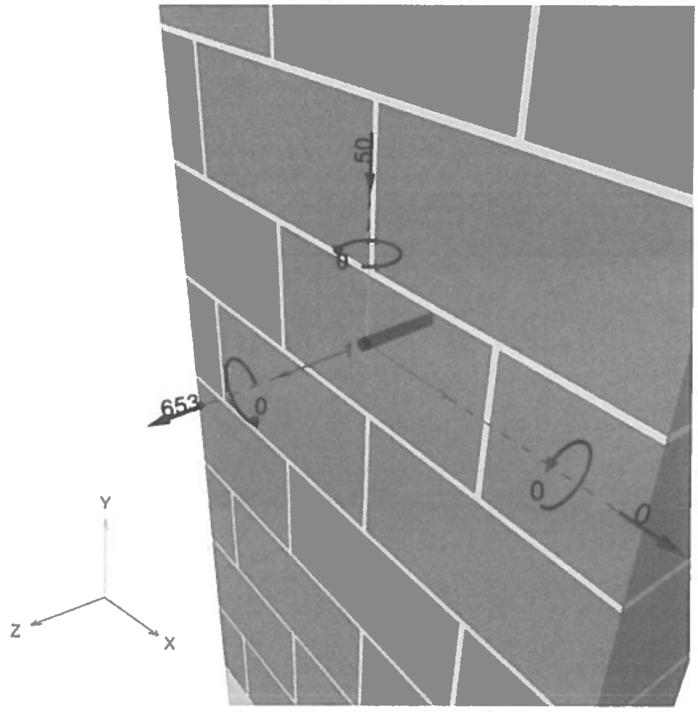
Anchor type and diameter:	**Kwik Bolt TZ - CS 5/8 (4)**
Item number:	not available
Effective embedment depth:	h_{ef} = 4.000 in.
Material:	Carbon Steel
Evaluation Service Report:	ESR-3785
Issued I Valid:	7/1/2019 | 7/1/2020
Proof:	Design Method ASD Masonry
Stand-off installation:	
Profile:	
Base material:	Grout-filled CMU, L x W x H: 16.000 in. x 8.000 in. x 8.000 in.; Joints: vertical: 0.375 in.; horizontal: 0.375 in. Base material temperature: 68 °F
Installation:	Face installation
Seismic loads	no

Geometry [in.]



Company:		Page:	2
Address:		Specifier:	
Phone I Fax:	|	E-Mail:	
Design:	Masonry - Feb 6, 2020	Date:	2/6/2020
Fastening point:			

Geometry [in.] & Loading [lb, in.lb]



1.1 Design results

Case	Description	Forces [lb] / Moments [in.lb]	Seismic	Max. Util. Anchor [%]
1	Combination 1	N = 653; V_x = 0; V_y = -50; M_x = 0; M_y = 0; M_z = 0;	no	76

2 Load case/Resulting anchor forces

Load case: Service loads

Anchor reactions [lb]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	653	50	0	-50

max. compressive strain: - [‰]
max. compressive stress: - [psi]
resulting tension force in (x/y)=(0.000/0.000): 0 [lb]
resulting compression force in (x/y)=(0.000/0.000): 0 [lb]

PROJECT California Department of Motor Vehicles

PROJECT NO. 2018-0187 DATE 10/4/2019

CLIENT BY ICG SHEET NO. CW1

design of top members DMV and columns package

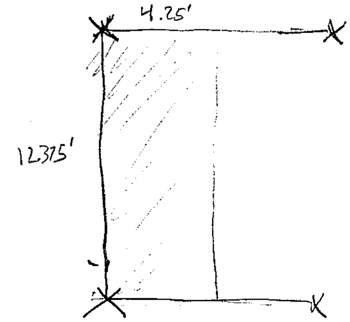
trib length = 4.25' $L_r = 20$ psf $D = 10$ psf

member length = 12.375

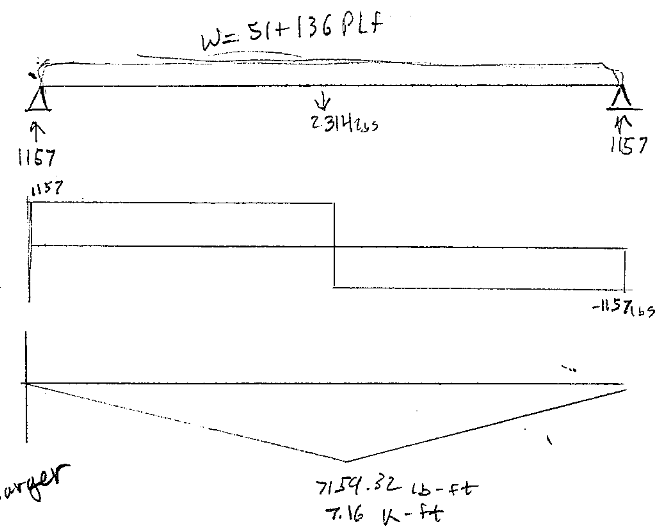
$L_r = 20\text{psf} \cdot 4.25\text{ft} = 85$ plf

$D = 10\text{psf} \cdot 4.25 = 42.5$ plf

$1.2D = 51$ plf $1.6L_r = 136$ plf



(pin, pined, worst case)



or larger

HSS 6X6X$\frac{1}{4}$ — $M_u = 7.16$ k-ft

$S_x = 9.54$ $F_y = 36$ksi (conservitive)

$M_n = S_x \cdot F_y = 9.54 \cdot 36\text{ksi} = 343\ \text{k-in} = 28.62\ \text{k-ft}$

$\phi M_n = 0.9 \cdot 28.62 = 25.76$ k-ft

$\phi M_n > M_u$

(use HSS 6X6X$\frac{1}{4}$ for both top members

Check Shear

$A_w = 5.24\text{in}^2$ $F_y = 36$ksi

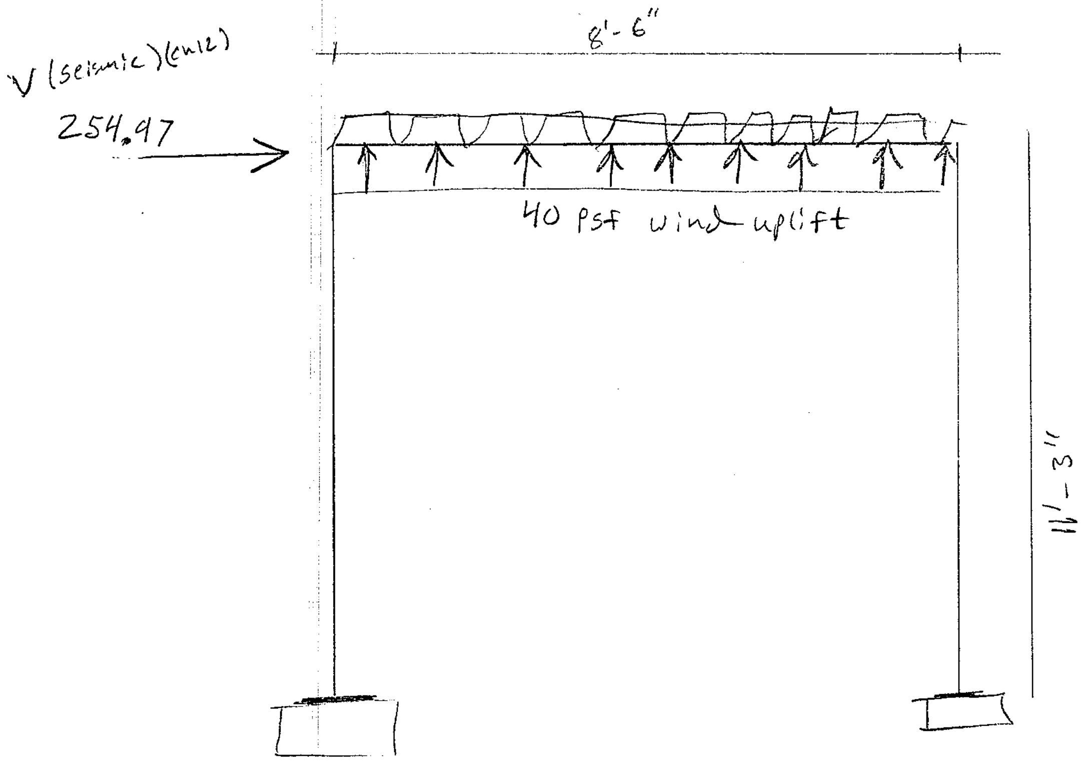
$V_u = 1.157$ kips

$V_n = A_w \cdot f_y \cdot 0.54 = 101$ kips $\phi V_n = 91.7$

$\phi V_n > V_u$

PROJECT			
PROJECT NO.		DATE	
CLIENT	BY	SHEET NO.	CW2

Chapter 15 Seismic Design Requirements for nonbuilding Structures



Dead Load	Wind Load	Seismic
Deck - 3 psf	40 psf uplift	Site Class D
Beams - 1.5 psf		$S_S = 0.761$ $S_{DS} = 0.606$
ect - 5.5		$S_1 = 0.308$ $S_{D1} = 0.366$
DL = 10 psf		$R = ?$ $I_E = 1.0$

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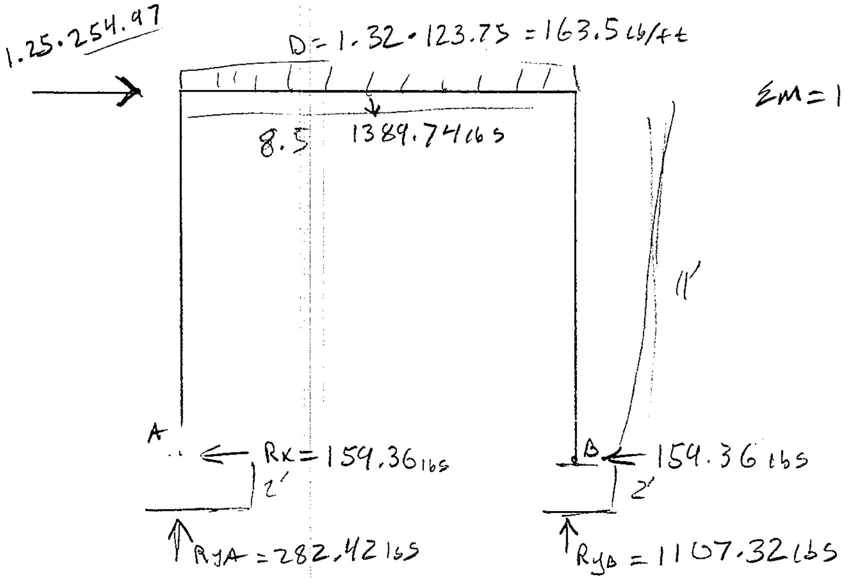
PROJECT

PROJECT NO. DATE

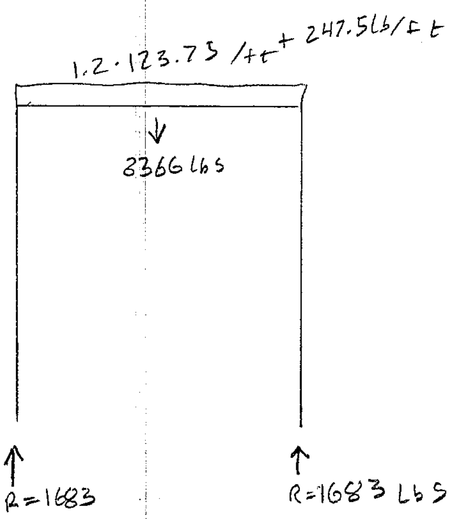
CLIENT BY SHEET NO. CW3

lateral seismic load combination ((1.2 + 0.2SDS) D + $Q_E \Omega_0$ case)



$\Sigma M = 1.25 \cdot 254.97 \cdot 11' + 1389.74 \cdot 4.25' - R_{yB} \cdot 8.5$

Check gravity (1.2D + 1.6 Lr case) $L_r = 20\,psf \times 12.375 = 247.5$ lb/ft



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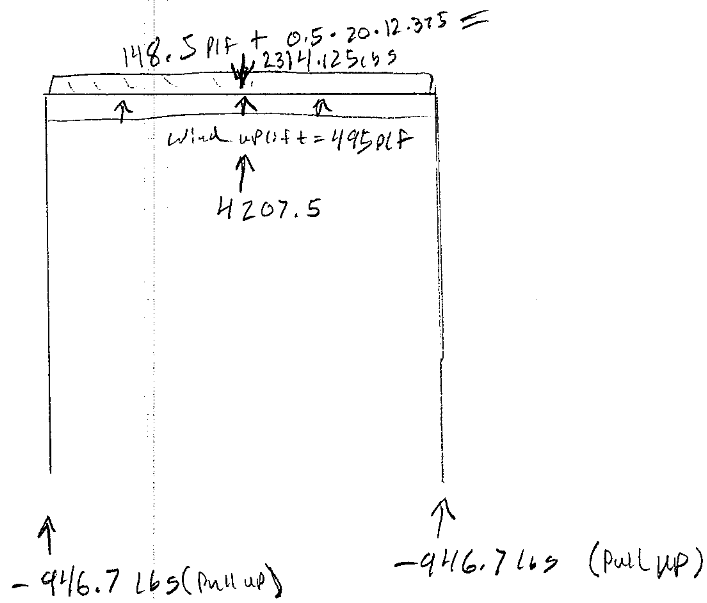
PROJECT ______

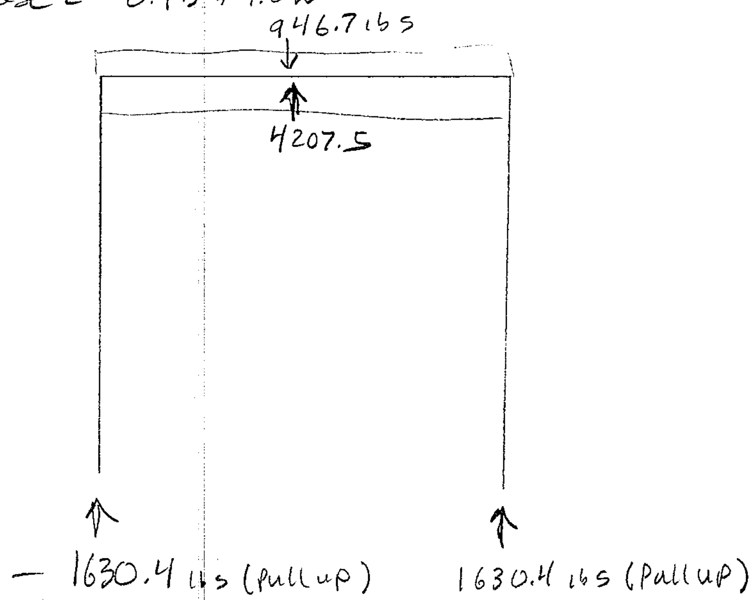
PROJECT NO. ______ DATE ______

CLIENT ______ BY ______ SHEET NO. CW4

Wind loading case 1.2D + 1.0W + 0.5 Lr



Wind case 2 0.9D + 1.0W



Typical Cantileaver Column Lateral design

$P_{DL} = 10\,psf\,(12.375' \cdot 8.5)$ tributary length = 12'-4 1/2"
$= 1051.875$

Table 12.2-1

select steel special cantileaver column system (Detailing 14.1)
$R = 2.5$ $\Omega_0 = 1.25$ $C_d = 2.5$ height limit = 35'
$I_e = 1.0$ (Table 1.5-2) Seismic design category D

12.8.1 Equivilent Lateral force Proceedure

$V = C_s \cdot W$

$C_s = \frac{S_{DS}}{(R/I_e)} = \left(\frac{0.606}{2.5/1}\right) = 0.2424$ 6 x 6 x ½ 2 ft thick

$W = P_{DL} = 1051.875$ (for tributary section)

$\Rightarrow V_{eq} = 0.2424 \cdot 1051.875 = 254.97$ lbs

Check column bending

(Ch 12.5.4) increase V_{Eq} by 30% $V_{eq} = 254.97 \cdot 1.3 = 331.461$ lbs

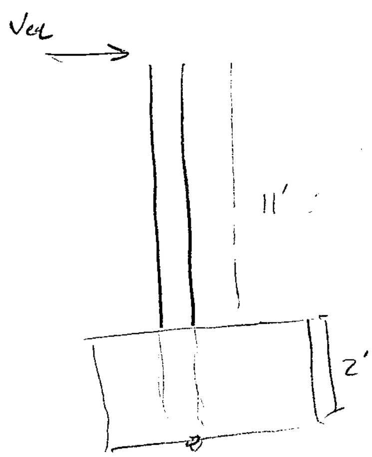
on 1 column $V_{eq}/2 = 165.73$ lbs

$M_{Base} = 165.73 \cdot 13' = 2154.5$ lb-ft or 2.154 kip-ft

HSS 6X6X½ $\phi M_n = 74.3$ kip-ft

$M_u \cdot \Omega_0 = 2.154 \cdot 1.25 = 2.6925$

$M_{base} = 2.6425$ kip-ft



PROJECT

PROJECT NO. DATE

CLIENT BY SHEET NO. CW6

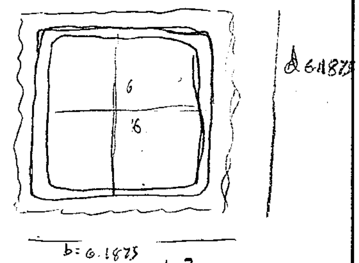
ch 12.8-2
12.12-1

typical cantileaver design, lateral

$V = \Omega \cdot V_{eq} = 1.25 \cdot 165.73\ lbs = 207.16\ lbs$

$\phi V_n = 0.75\,(0.6 \times 70 ksi)\,(3/16)\,(\sqrt{2}/2) \cdot (4 \times 3.75)$

$= 62.64\ kips$



? $\phi M_n = 0.75 \cdot 0.6 \cdot 70 ksi\ (Z_w = 5.22 in^3)$

$= 164.4\ k\text{-}in \qquad 13.7\ K\text{-}ft$

$Z_w = \frac{bd^2}{4} - \frac{b_1 d_1^2}{4}$

$Z_w = 5.22\ in^3$

$DC\text{-}M = \frac{2.1642 k\text{-}ft}{13.7\ k\text{-}ft} = 0.193 < 1$ O.K.

$DC\text{-}V = \frac{207.16\ lbs}{62{,}000\ lbs} = 0.003 < 1$ O.K.

use $\underline{3/16''}$ fillet weld all around base

Drift check

$\Delta_{eq} = \frac{V_{eq} \cdot L}{3\,EI} = \frac{0.2 k \cdot (11'^3)}{3 \cdot 29{,}000 ksi \cdot 48.3 in^4} = 0.11''$

$\delta_1 = \frac{C_d\,\Delta_{eq}}{I_e} = \frac{2.5 \cdot 0.11''}{1.0} = 0.275''$

Table 12.12-1 Allowable story drift

$\Delta_1 = 0.025 \cdot h = 0.025 \cdot 132'' = 3.3''$

$0.275'' < 3.3''$ OK

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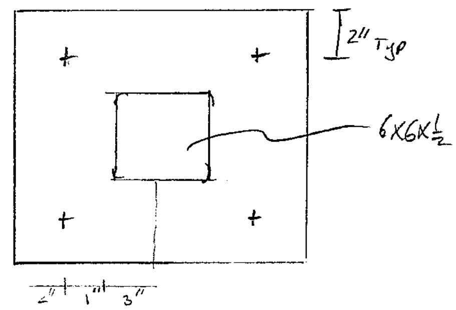
0.0018

PROJECT ______
PROJECT NO. ______ DATE ______
CLIENT ______ BY ______ SHEET NO. CW7

Canopy Base PL design

$V_u\text{-Bolt} = \Omega_o \cdot V_{eq} / 4$

$= 1.25 \cdot \dfrac{127.48\,\text{lbs}}{4} = 40\,\text{lbs/Bolt shear}$

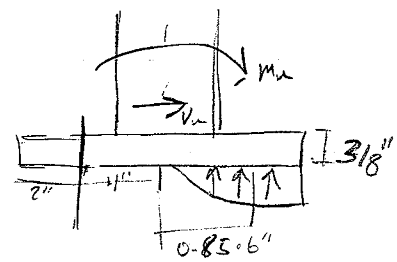


$T_u\text{-Bolt} = (\Omega_o \cdot M_{eq}) / (2 \cdot (4" + 0.85 \cdot 6"))$

$= (1.25 \cdot 2.64\,\text{kip-ft}) / \underbrace{(18.2")/12"}_{1.52}$

$= 2.176\,\text{kips/bolt}$

$M_{u\,plate} = T_u \times 2 \cdot 1.5" = 6.528\,\text{kip-in}$



thickness

$M_u = F_y \cdot Z_{x\,plate} = F_y \cdot \dfrac{b \cdot t^2}{4}$

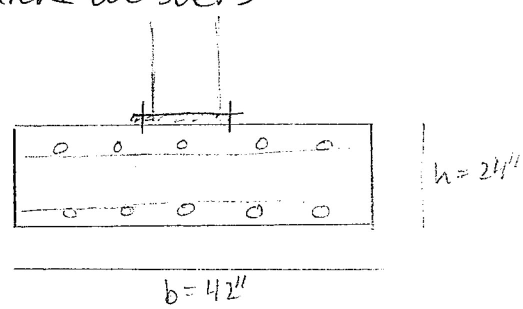
$\sqrt{\dfrac{4 M_u}{F_y \cdot b}} = \cancel{b} \cdot t^2 = t_{min} = \sqrt{\dfrac{4 \cdot M_u}{0.9 \cdot F_y \cdot b}} = \sqrt{\dfrac{4 \cdot 6.528\,\text{kip-in}}{0.9 \cdot 36\,\text{ksi} \cdot 12"}}$

$\Rightarrow t_{min} = 0.259"$ $\boxed{\text{~~3/8" plate o.k. to use~~}}$ grade 36

use 3/4" (constructability)

— use 4 3/4 Ø x 24" F1554 GR 36 Anchor rods w/ 6" projection
4-1 5/16" Ø Holes and 2" Ø 1/4" thick washers

— use 3.5' x 3.5' x 2' footing



use reinforcement ratio 0.0018

$A_s = 0.0018 A_g = 0.0018 \cdot 42" \cdot 24" = 1.8144\,\text{in}^2$

Need $0.9072\,\text{in}^2$ top and bottom, each direction

use 5-#4 top and bottom each direction

PROJECT	Delano DMV		
PROJECT NO.	2018-0187	DATE	4/1/2020
CLIENT	NLA	BY ICG	SHEET NO. E1

Equipment Schedule

Designation	Description	a_p	R_p	LxWxH	Weight (lbs)	Mechanical Detail
AHU-1	Outdoor air handler	2.5	6	157"x64"x59"	3500	6/M5.02
VRF-1	Outdoor VRF Heat Pump (on anti vibration rubber pad)	2.5	2.5	36"x29.5"x65"	450	9/M5.02
CU-1	Outdoor Air Conditioning System (on anti vibration rubber pad)	2.5	6	36.5"x10"x12"	250	9/M5.02
EF-1	Roof Exhaust Fan (Upblast)	2.5	6	22"x22"x25"	100	9/M5.01
EF-2	Suspended Exhaust Fan (in-line spring isolated)	2.5	2.5	21"x17"x17"	100	4/M5.01
EF-3	Roof Exhaust Fan (Upblast)	2.5	6	22"x22"x25"	100	9/M5.01
EF-4	Roof Exhaust Fan (Upblast)	2.5	6	22"x22"x25"	100	9/M5.01
ERV-1	Energy Recovery Ventilator (suspended)	2.5	2.5	48.5"x45"x28"	300	8/M5.02
BC-1	VRF Branch Controller (supsended)	2.5	2.5	11"x43"x17"	150	10/M5.02

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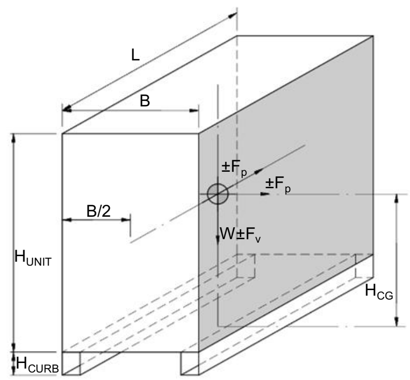
Mechanical Equipment Anchorage Calc
BASE MOUNTED



Unit Identification: **ACH-1**

W_p =	3500 lbs	unit weight
B =	64.00 in	unit width
$d_{B\ dir\ anchors}$ =	60.00 in	dist btwn anchors (B dir.)
L =	157.00 in	unit length
$d_{L\ dir\ anchors}$ =	150.00 in	dist btwn anchors (L dir.)
H_{UNIT} =	59.00 in	unit height
H_{CG_EQUIP} =	40.00 in	center of gravity height (equip)
H_{CURB} =	6.00 in	curb height
H_{CG} =	46.00 in	(includes curb height)

Seismic Load (ASCE 7-16, Ch. 13)

a_p and R_p obtained from ASCE 7-16, Tables 13.5-1 and 13.6-1

a_p =	2.50	R_p =	6.00	I_p =	1.00	S_{DS} =	0.606
z =	12.00 ft	h =	12.00 ft	Ωo =	2.0		

$F_p = (0.4[(a_p)(S_{DS})(I_p)(1+2(z/h)]/R_p) \times W_p =$ 0.303 Wp $F_v = 0.2(S_{DS})W_p =$ 0.121 Wp

$F_{p\text{-}MIN} = 0.3(S_{DS})(I_p) \times W_p =$ 0.182 Wp

$F_{p\text{-}MAX} = 1.6(S_{DS})(I_p) \times (W_p) =$ 0.970 Wp

STRENGTH		SERVICE	
$F_p = V_h =$	**1061 lbs**	**$0.7F_p = V_h =$**	**742 lbs**
$F_v = V_v =$	**424 lbs**	**$0.7F_v = V_v =$**	**297 lbs**

Unit Anchorage:

Fp in "B" Direction

connections shear 10
connections tension 2

STRENGTH @ 1.0E		STRENGTH @ Ω		SERVICE	
$M_{OT} = V_hH_{CG}$ =	4065 lb-ft	$M_{OT} = \Omega V_hH_{CG}$ =	8131 lb-ft	$M_{OT} = V_hH_{CG}$ =	2846 lb-ft
$M_R = (0.9\text{-}0.2S_{ds})W_p(B/2)$ =	7269 lb-ft	$M_R = (0.9\text{-}0.2S_{ds})W_p(B/2)$ =	7269 lb-ft	$M_R = (0.6\text{-}0.14S_{ds})W_p(B/2)$ =	4808 lb-ft
$T = (M_{OT} - M_R) / d_B$ =	no net uplift	$T = (M_{OT} - M_R) / d_B$ =	172 lbs	$T = (M_{OT} - M_R) / d_B$ =	no net uplift
T_{conn} = T / # conn tension =	no net uplift	T_{conn} = T / # conn tension =	86 lbs	T_{conn} = T / # conn tension =	no net uplift
$V_{conn} = V_h$ / # conn shear =	106 lbs	$V_{conn} = \Omega V_h$ / # conn shear =	212 lbs	$V_{conn} = V_h$ / # conn shear =	74 lbs

Fp in "L" Direction

connections shear 10
connections tension 2

STRENGTH @ 1.0E		STRENGTH @ Ω		SERVICE	
$M_{OT} = V_hH_{CG}$ =	4065 lb-ft	$M_{OT} = \Omega V_hH_{CG}$ =	8131 lb-ft	$M_{OT} = V_hH_{CG}$ =	2846 lb-ft
$M_R = (0.9\text{-}0.2S_{ds})W_p(L/2)$ =	17831 lb-ft	$M_R = (0.9\text{-}0.2S_{ds})W_p(L/2)$ =	17831 lb-ft	$M_R = (0.6\text{-}0.14S_{ds})W_p(L/2)$ =	11795 lb-ft
$T = (M_{OT} - M_R) / d_L$ =	no net uplift	$T = (M_{OT} - M_R) / d_L$ =	no net uplift	$T = (M_{OT} - M_R) / d_L$ =	no net uplift
T_{conn} = T / # conn tension =	no net uplift	T_{conn} = T / # conn tension =	no net uplift	T_{conn} = T / # conn tension =	no net uplift
$V_{conn} = V_h$ / # conn shear =	106 lbs	$V_{conn} = \Omega V_h$ / # conn shear =	212 lbs	$V_{conn} = V_h$ / # conn shear =	74 lbs

Max Connection Forces:

STRENGTH @ 1.0E		STRENGTH @ Ω		SERVICE	
Tmax =	0 lbs	Tmax =	86 lbs	Tmax =	0 lbs
Vmax =	106 lbs	Vmax =	212 lbs	Vmax =	74 lbs

Connection/Anchorage Notes:

Seismic governs Tmax = 86 lbs Vmax = 212 lbs
In Channel use galv 3/8" A307 bolt every 48"
Tu= 3712 lbs D/C = 0.02

Check wind loading of Roof mounted Equipment

- ASCE 7-16 - 29.4 Risk category B $V = 95$

$AF < (0.1\ B \times h) \Rightarrow G_{Cr} = 1.9$ and $GCr = 1.5$ for verticle

$F_h = q_h (GC_r) A_F$

$q_h = 0.00256 \cdot 0.57 \cdot 1 \cdot 0.85 \cdot 1 \cdot 95^2 = 11.85\ psf$

$GCr = 1.9$

$F_V = q_h (GC_r) A_r = 11.85 \times 1.5 \times A_r$

Wind Check of AHU-1

$W = 3091$ lbs $d = 54.5$ # connections $= 10$

$F_h = q_h (GC_r) \times \left(\frac{54 \times 157}{144}\right) = 11.85 \times 1.9 \times 64.3\ ft^2 = 1447.7\ lb$

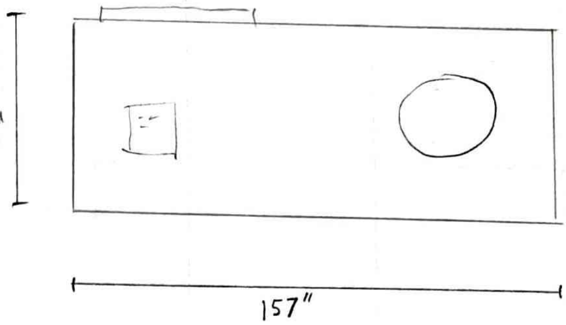
$FV = q_h (GCr) \times \left(\frac{54.5 \times 157}{144}\right) = 11.85 \times 1.5 \times 65\ ft^2 = 1155\ lbs$

$M_{ot} = F_h \times \frac{54}{2} + f_V \times \frac{54.5}{2} - W \times \frac{54.5}{2}$

$= 1447.7 \times \frac{54}{2} + 1155 \times \frac{54.5}{2} - 3091 \times \frac{54.3}{2} = -14886.85$

NO tension uplift

max Shear $= F_h/10 = 144.77$ lbs



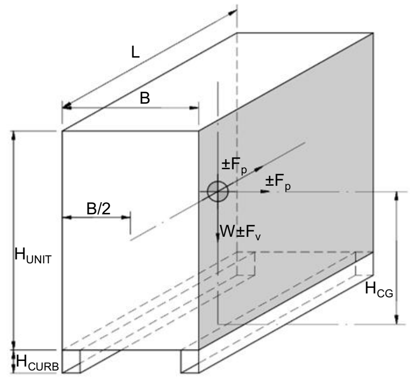
BUEHLER

PROJECT Delano DMV

PROJECT NO. 2018-0187 DATE 4/1/2020 E4

CLIENT NLA BY ICG SHEET NO.

Mechanical Equipment Anchorage Calc
BASE MOUNTED



Unit Identification: **EF-1,3,4**

W_p =	100 lbs	unit weight
B =	22.00 in	unit width
$d_{B\ dir\ anchors}$ =	22.00 in	dist btwn anchors (B dir.)
L =	22.00 in	unit length
$d_{L\ dir\ anchors}$ =	22.00 in	dist btwn anchors (L dir.)
H_{UNIT} =	25.00 in	unit height
H_{CG_EQUIP} =	17.00 in	center of gravity height (equip)
H_{CURB} =	6.00 in	curb height
H_{CG} =	23.00 in	(includes curb height)

Seismic Load (ASCE 7-16, Ch. 13)

a_p and R_p obtained from ASCE 7-16, Tables 13.5-1 and 13.6-1

a_p = 2.50	R_p = 6.00	I_p = 1.00	S_{DS} = 0.606
z = 16.00 ft	h = 16.00 ft	Ωo = 2.0	

$F_p = (0.4[(a_p)(S_{DS})(I_p)(1+2(z/h)]/R_p) \times W_p =$ 0.303 Wp

$F_{p\text{-}MIN} = 0.3(S_{DS})(I_p) \times W_p =$ 0.182 Wp

$F_{p\text{-}MAX} = 1.6(S_{DS})(I_p) \times (W_p) =$ 0.970 Wp

$F_v = 0.2(S_{DS})W_p =$ 0.121 Wp

STRENGTH	
$F_p = V_h$ =	**30 lbs**
$F_v = V_v$ =	**12 lbs**

SERVICE	
$0.7F_p = V_h$=	**21 lbs**
$0.7F_v = V_v$ =	**8 lbs**

Unit Anchorage:

Fp in "B" Direction

connections shear 4

connections tension 2

STRENGTH @ 1.0E		STRENGTH @ W		SERVICE	
$M_{OT} = V_hH_{CG}$ =	58 lb-ft	$M_{OT} = \Omega V_hH_{CG}$ =	116 lb-ft	$M_{OT} = V_hH_{CG}$ =	41 lb-ft
$M_R = (0.9\text{-}0.2S_{ds})W_p(B/2)$ =	71 lb-ft	$M_R = (0.9\text{-}0.2S_{ds})W_p(B/2)$ =	71 lb-ft	$M_R = (0.6\text{-}0.14S_{ds})W_p(B/2)$ =	47 lb-ft
$T = (M_{OT} - M_R) / d_B$ =	no net uplift	$T = (M_{OT} - M_R) / d_B$ =	24 lbs	$T = (M_{OT} - M_R) / d_B$ =	no net uplift
T_{conn} = T / # conn tension =	no net uplift	T_{conn} = T / # conn tension =	12 lbs	T_{conn} = T / # conn tension =	no net uplift
$V_{conn} = V_h$ / # conn shear =	8 lbs	$V_{conn} = \Omega V_h$ / # conn shear =	15 lbs	$V_{conn} = V_h$ / # conn shear =	5 lbs

Fp in "L" Direction

connections shear 4

connections tension 2

STRENGTH @ 1.0E		STRENGTH @ W		SERVICE	
$M_{OT} = V_hH_{CG}$ =	58 lb-ft	$M_{OT} = \Omega V_hH_{CG}$ =	116 lb-ft	$M_{OT} = V_hH_{CG}$ =	41 lb-ft
$M_R = (0.9\text{-}0.2S_{ds})W_p(L/2)$ =	71 lb-ft	$M_R = (0.9\text{-}0.2S_{ds})W_p(L/2)$ =	71 lb-ft	$M_R = (0.6\text{-}0.14S_{ds})W_p(L/2)$ =	47 lb-ft
$T = (M_{OT} - M_R) / d_L$ =	no net uplift	$T = (M_{OT} - M_R) / d_L$ =	24 lbs	$T = (M_{OT} - M_R) / d_L$ =	no net uplift
T_{conn} = T / # conn tension =	no net uplift	T_{conn} = T / # conn tension =	12 lbs	T_{conn} = T / # conn tension =	no net uplift
$V_{conn} = V_h$ / # conn shear =	8 lbs	$V_{conn} = \Omega V_h$ / # conn shear =	15 lbs	$V_{conn} = V_h$ / # conn shear =	5 lbs

Max Connection Forces:

STRENGTH @ 1.0E	
Tmax =	0 lbs
Vmax =	8 lbs

STRENGTH @ W	
Tmax =	12 lbs
Vmax =	15 lbs

SERVICE	
Tmax =	0 lbs
Vmax =	5 lbs

Connection/Anchorage Notes:

Wind governs Tmax = 27.79 lbs Vmax = 18.5 lbs
In 18ga curb use 4 sms#8
Tu= 94 lbs D/C = 0.295

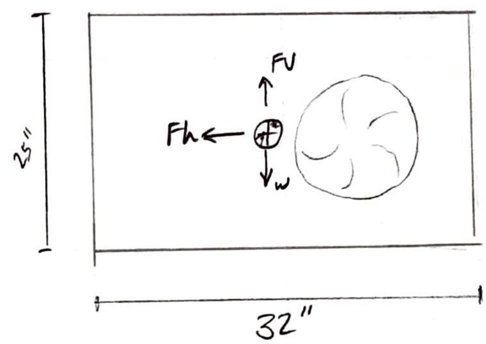
Wind Check of CU-1

$W = 92$ $d = 11.5\,in$ # connections $= 4$

$F_h = q_h (GC_r) \times \left(\frac{32 \times 24}{144}\right) = 11.85\,psf \times 1.9 \times 5.33\,ft^2 = 120.08\,lbs$

$F_V = q_h (GC_r) \times \left(\frac{11.5 \times 32}{144}\right) = 11.85 \times 1.5 \times 2.55\,ft^2$

$F_V = 45.43\,lbs$



$M_{ot} = F_h \cdot \frac{25}{2} + F_V \cdot \frac{11.5}{2} - W \cdot \frac{11.5}{2}$

$= 120 \cdot \frac{25}{2} + 45.43 \cdot \frac{11.5}{2} - 92 \cdot \frac{11.5}{2} = 1232\,lb\text{-}in$

$M_{ot}/d = \frac{1232\,lb\text{-}in}{11.5\,in} = 107.2\,lbs$

w/ 2 connections

ea bolt $= 65.64/2 = \boxed{53.57\,lbs}$ tension

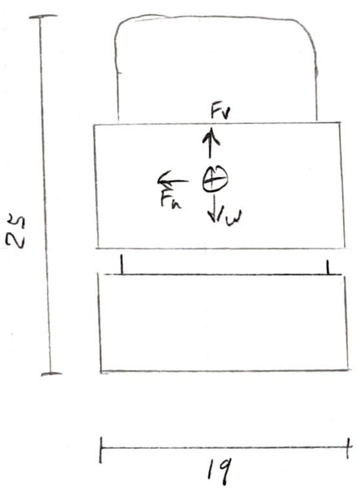
$V = F_h/4 = 120/4 = 30\,lbs$

Wind Check of EF-1,3,4

$W = 31\,lbs$ $d = 19''$ # Connections

$F_h = q_h \times (GC_r) \times \left(\frac{25 \times 19}{144}\right) = 11.85\,psf \times 1.9 \times 3.3\,ft^2 = 74.3\,lbs$

$F_V = q_h \times (GC_r) \times \left(\frac{19 \times 19}{144}\right) = 11.85\,psf \times 1.5 \times 2.5 = 44.4\,lbs$



$M_{ot} = F_h \times \frac{25}{2} + F_V \times \frac{19}{2} - W \times \frac{19}{2}$

$= 74.3 \times \frac{25}{2} + 44.4 \times \frac{19}{2} - 31 \times \frac{19}{2} = 1056.05\,lb\text{-}in$

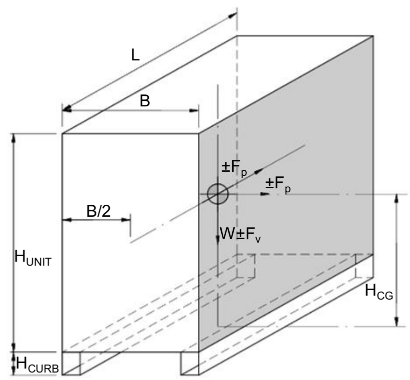
$M_{ot}/d = \frac{1056.05}{19} = 55.58\,lbs$

w/ 2 connections

ea bolt $= 55.58/2 = \boxed{27.79\,lbs}$

$V = F_h/4 = 74.3/4 = 18.5\,lbs$

Mechanical Equipment Anchorage Calc
BASE MOUNTED



Unit Identification: **CU-1**

W_p =	250 lbs	unit weight
B =	11.00 in	unit width
$d_{B\ dir\ anchors}$ =	9.00 in	dist btwn anchors (B dir.)
L =	32.00 in	unit length
$d_{L\ dir\ anchors}$ =	34.00 in	dist btwn anchors (L dir.)
H_{UNIT} =	24.00 in	unit height
H_{CG_EQUIP} =	16.00 in	center of gravity height (equip)
H_{CURB} =	6.00 in	curb height
H_{CG} =	22.00 in	(includes curb height)

Seismic Load (ASCE 7-16, Ch. 13)

a_p and R_p obtained from ASCE 7-16, Tables 13.5-1 and 13.6-1

a_p = 2.50	R_p = 6.00	I_p = 1.00	S_{DS} = 0.606
z = 15.00 ft	h = 16.00 ft	Ωo = 2.0	

$F_p = (0.4[(a_p)(S_{DS})(I_p)(1+2(z/h))]/R_p) \times W_p =$ 0.290 Wp

$F_v = 0.2(S_{DS})W_p =$ 0.121 Wp

$F_{p\text{-}MIN} = 0.3(S_{DS})(I_p) \times W_p =$ 0.182 Wp

$F_{p\text{-}MAX} = 1.6(S_{DS})(I_p) \times (W_p) =$ 0.970 Wp

STRENGTH	
$F_p = V_h$ =	**73 lbs**
$F_v = V_v$ =	**30 lbs**

SERVICE	
$0.7F_p = V_h$=	**51 lbs**
$0.7F_v = V_v$ =	**21 lbs**

Unit Anchorage:

Fp in "B" Direction

\# connections shear 4
\# connections tension 2

STRENGTH @ 1.0E		STRENGTH @ W		SERVICE	
$M_{OT} = V_hH_{CG}$ =	133 lb-ft	$M_{OT} = \Omega V_hH_{CG}$ =	266 lb-ft	$M_{OT} = V_hH_{CG}$ =	93 lb-ft
$M_R = (0.9\text{-}0.2S_{ds})W_p(B/2)$ =	89 lb-ft	$M_R = (0.9\text{-}0.2S_{ds})W_p(B/2)$ =	89 lb-ft	$M_R = (0.6\text{-}0.14S_{ds})W_p(B/2)$ =	59 lb-ft
$T = (M_{OT} - M_R) / d_B$ =	58 lbs	$T = (M_{OT} - M_R) / d_B$ =	236 lbs	$T = (M_{OT} - M_R) / d_B$ =	46 lbs
T_{conn} = T / # conn tension =	29 lbs	T_{conn} = T / # conn tension =	118 lbs	T_{conn} = T / # conn tension =	23 lbs
$V_{conn} = V_h$ / # conn shear =	18 lbs	$V_{conn} = \Omega V_h$ / # conn shear =	36 lbs	$V_{conn} = V_h$ / # conn shear =	13 lbs

Fp in "L" Direction

\# connections shear 4
\# connections tension 2

STRENGTH @ 1.0E		STRENGTH @ W		SERVICE	
$M_{OT} = V_hH_{CG}$ =	133 lb-ft	$M_{OT} = \Omega V_hH_{CG}$ =	266 lb-ft	$M_{OT} = V_hH_{CG}$ =	93 lb-ft
$M_R = (0.9\text{-}0.2S_{ds})W_p(L/2)$ =	260 lb-ft	$M_R = (0.9\text{-}0.2S_{ds})W_p(L/2)$ =	260 lb-ft	$M_R = (0.6\text{-}0.14S_{ds})W_p(L/2)$ =	172 lb-ft
$T = (M_{OT} - M_R) / d_L$ =	no net uplift	$T = (M_{OT} - M_R) / d_L$ =	2 lbs	$T = (M_{OT} - M_R) / d_L$ =	no net uplift
T_{conn} = T / # conn tension =	no net uplift	T_{conn} = T / # conn tension =	1 lbs	T_{conn} = T / # conn tension =	no net uplift
$V_{conn} = V_h$ / # conn shear =	18 lbs	$V_{conn} = \Omega V_h$ / # conn shear =	36 lbs	$V_{conn} = V_h$ / # conn shear =	13 lbs

Max Connection Forces:

STRENGTH @ 1.0E	
Tmax =	29 lbs
Vmax =	18 lbs

STRENGTH @ W	
Tmax =	118 lbs
Vmax =	36 lbs

SERVICE	
Tmax =	23 lbs
Vmax =	13 lbs

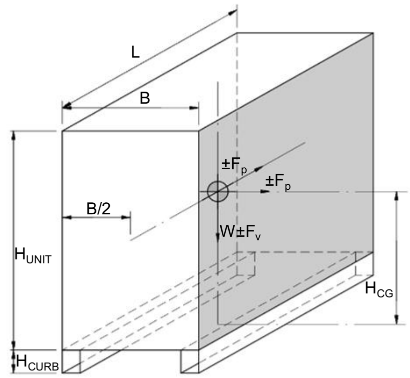
Connection/Anchorage Notes:

Seismic governs Tmax = 118 lbs Vmax = 236 lbs
In Connetion use galv 3/8" A307 bolt at mounting bracket.
Tu= 3712 lbs D/C = 0.032

PROJECT Delano DMV			
PROJECT NO. 2018-0187		DATE 4/1/2020	
CLIENT NLA	BY ICG	SHEET NO.	

E7

Mechanical Equipment Anchorage Calc
BASE MOUNTED



Unit Identification: **VRF-1**

W_p =	450 lbs	unit weight
B =	29.00 in	unit width
$d_{B\ dir\ anchors}$ =	27.00 in	dist btwn anchors (B dir.)
L =	37.00 in	unit length
$d_{L\ dir\ anchors}$ =	35.00 in	dist btwn anchors (L dir.)
H_{UNIT} =	65.00 in	unit height
H_{CG_EQUIP} =	43.00 in	center of gravity height (equip)
H_{CURB} =	6.00 in	curb height
H_{CG} =	49.00 in	(includes curb height)

Seismic Load (ASCE 7-16, Ch. 13)

a_p and R_p obtained from ASCE 7-16, Tables 13.5-1 and 13.6-1

a_p =	2.50	R_p =	2.50	I_p =	1.00	S_{DS} =	0.606
z =	15.00 ft	h =	16.00 ft	Ωo =	2.0		

$F_p = (0.4[(a_p)(S_{DS})(I_p)(1+2(z/h)]/R_p) \times W_p =$ 0.697 Wp

$F_{p\text{-}MIN} = 0.3(S_{DS})(I_p) \times W_p =$ 0.182 Wp

$F_{p\text{-}MAX} = 1.6(S_{DS})(I_p) \times (W_p) =$ 0.970 Wp

$F_v = 0.2(S_{DS})W_p =$ 0.121 Wp

STRENGTH	
$F_p = V_h$ =	**314 lbs**
$F_v = V_v$ =	**55 lbs**

SERVICE	
$0.7F_p = V_h$=	**220 lbs**
$0.7F_v = V_v$ =	**38 lbs**

Unit Anchorage:

Fp in "B" Direction

connections shear 4

connections tension 2

STRENGTH @ 1.0E		STRENGTH @ W		SERVICE	
$M_{OT} = V_hH_{CG}$ =	1281 lb-ft	$M_{OT} = \Omega V_hH_{CG}$ =	2561 lb-ft	$M_{OT} = V_hH_{CG}$ =	896 lb-ft
M_R= $(0.9\text{-}0.2S_{ds})W_p(B/2)$ =	423 lb-ft	$M_R = (0.9\text{-}0.2S_{ds})W_p(B/2)$ =	423 lb-ft	$M_R = (0.6\text{-}0.14S_{ds})W_p(B/2)$ =	280 lb-ft
$T = (M_{OT} - M_R) / d_B$ =	381 lbs	$T = (M_{OT} - M_R) / d_B$ =	950 lbs	$T = (M_{OT} - M_R) / d_B$ =	274 lbs
T_{conn} = T / # conn tension =	190 lbs	T_{conn} = T / # conn tension =	475 lbs	T_{conn} = T / # conn tension =	137 lbs
$V_{conn} = V_h$ / # conn shear =	78 lbs	$V_{conn} = \Omega V_h$ / # conn shear =	157 lbs	$V_{conn} = V_h$ / # conn shear =	55 lbs

Fp in "L" Direction

connections shear 4

connections tension 2

STRENGTH @ 1.0E		STRENGTH @ W		SERVICE	
$M_{OT} = V_hH_{CG}$ =	1281 lb-ft	$M_{OT} = \Omega V_hH_{CG}$ =	2561 lb-ft	$M_{OT} = V_hH_{CG}$ =	896 lb-ft
M_R= $(0.9\text{-}0.2S_{ds})W_p(L/2)$ =	540 lb-ft	$M_R = (0.9\text{-}0.2S_{ds})W_p(L/2)$ =	540 lb-ft	$M_R = (0.6\text{-}0.14S_{ds})W_p(L/2)$ =	357 lb-ft
$T = (M_{OT} - M_R) / d_L$ =	254 lbs	$T = (M_{OT} - M_R) / d_L$ =	693 lbs	$T = (M_{OT} - M_R) / d_L$ =	185 lbs
T_{conn} = T / # conn tension =	127 lbs	T_{conn} = T / # conn tension =	346 lbs	T_{conn} = T / # conn tension =	92 lbs
$V_{conn} = V_h$ / # conn shear =	78 lbs	$V_{conn} = \Omega V_h$ / # conn shear =	157 lbs	$V_{conn} = V_h$ / # conn shear =	55 lbs

Max Connection Forces:

STRENGTH @ 1.0E	
Tmax =	190 lbs
Vmax =	78 lbs

STRENGTH @ W	
Tmax =	475 lbs
Vmax =	157 lbs

SERVICE	
Tmax =	137 lbs
Vmax =	55 lbs

Connection/Anchorage Notes:

Seismic governs Tmax = 475 lbs Vmax = 157 lbs
In Connetion use galv 3/8" A307 bolt at mounting bracket.
Tu= 3712 lbs D/C = 0.128

Check wind loading of Roof mounted Equipment

- ASCE 7-16 - 29.4 Risk category B $V = 95$

$A_F < (0.1\ B \times h) \Rightarrow G_{Cr} = 1.9$ and $G_{Cr} = 1.5$ for verticle

$F_h = q_h (G_{Cr}) A_F$

$q_h = 0.00256 \cdot 0.57 \cdot 1 \cdot 0.85 \cdot 1 \cdot 95^2 = 11.85\ psf$

$G_{Cr} = 1.9$

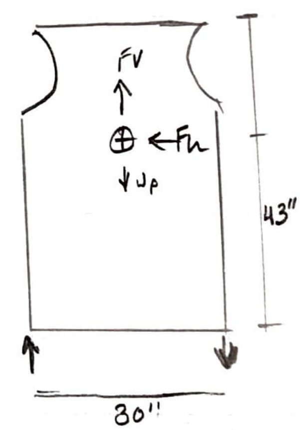
$F_V = q_h (G_{Cr}) A_r = 11.85 \times 1.5 \times A_r$

Wind check of VRF-1 (

$F_h = q_h (G_{Cr}) \times (36'' \times 65'') = 11.85\,psf \times 1.9 \times \frac{(36 \times 65)}{144} = \boxed{365.8\ lbs}$

$F_V = q_h (G_{Cr}) \times \frac{36 \times 30}{144} = 11.85\,psf \times 1.5 \times \left(\frac{36 \times 30}{144}\right) = 133.3$

$W = 450$



$M_{ot} = F_h \times 43'' + F_V \times (30/2)'' - W_p \times 30/2 =$

$= 365.8 \times 43 + 133.3 \times 15'' - 450 \times 15'' =$

$M_{ot} = 10478.9\ lb\text{-}in$ $M_{ot}/d = \frac{10478.9}{30''} = 365.9\ lbs$

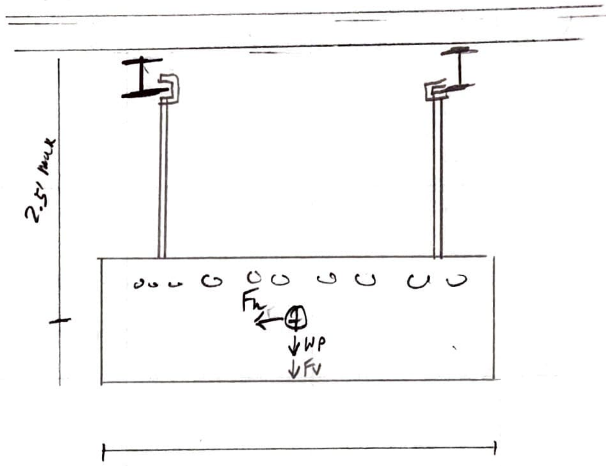
- 2 connections ea side

ea bolt $= 365.9/2 = 182.9\ lbs$ tension

$365.8/4 = 91.45\ lbs$ shear

BC-1 Suspended Anchorage

$W_p = 150\,lbs$ $a_p = 2.5$ $R_p = 2.5$ $I_p = 1.0$ $S_{DS} = 0.606$ $z = 14$ $h = 14$
$d = 11.5''$



$$F_p = \frac{0.4 \times a_p \times S_{DS} \times W_p}{R_p/I_p}\left(1+2\left(\frac{z}{h}\right)\right)$$

$$F_p = \frac{0.4 \times 2.5 \times 0.606 \times 150}{2.5/1.0}\left(1+2\left(\frac{14}{14}\right)\right)$$

$F_p = 109.08\,lbs$

$F_{p\,min} = 0.3 \times S_{DS} \times I_p \times W_p = 27.27\,lbs$

$F_{p\,max} = 1.6 \times S_{DS} \times I_p \times W_p = 145.44\,lbs$

$F_v = 0.2 \times 0.606 \times W_p = 18.18$

- Check Connection to Ceiling Support framing (LRFD)

Max forces in Vertical Support Connection

Dfactor = 1.2 # of Connections = 4

$$T_{max} = \frac{1.2 \times W_p + F_v}{4} = \frac{1.2 \times 150 + 18.18}{4} = 49.5\,lbs$$

$T_{max} = 49.5\,lbs$

- Use Beam Clamp to W member above
- Unistrut P2897

$T_u = 550\,lbs$ $\boxed{D/C = 0.09}$

- Check Braces

Braces Resisting EA Direction = 1 Brace Angle = (45°-60°)

Lateral Load to each brace = F_p = 109.08

Max axial load to each brace = 109.08/cos(60) = 218.16 lbs

Use 1/8" diameter Aircraft Cable

As = 0.0123 Fy = 42ksi Tu = As*Fy = 520 lbs $\boxed{D/C = 0.419}$

EF-2 Suspended Anchorage

- unit weight = 100lbs $a_p = 2.5$ $R_p = 2.5$ $I_p = 1.0$ $S_{ds} = 0.606$ $Z = 14$ $h = 14$

$$F_p = \frac{0.4 \cdot a_p \cdot S_{DS} \cdot W_p}{R_p / I_p}\left(1 + 2\times\left(\frac{z}{h}\right)\right)$$

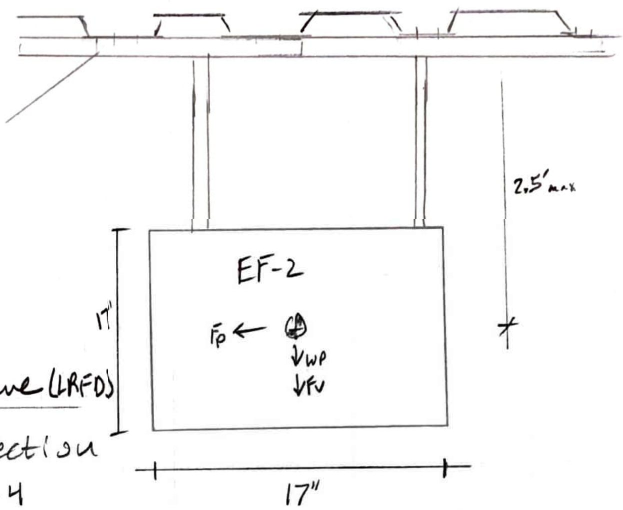
$$F_p = \frac{0.4 \cdot 2.5 \cdot 0.606 \cdot 100}{2.5/1.0}\left(1 + 2\times\left(\frac{14}{14}\right)\right)$$

$\underline{F_p = 72.72}$

$F_{p\,min} = 0.3 \times S_{ds} \times I_p \times W_p = 18\text{ lbs}$

$F_{p\,max} = 1.6 \times S_{ds} \times I_p = 96.96$

$F_v = 0.2 \times S_{ds} \times W_p = 12.12\text{ lbs}$



- Check Connection to Ceiling Support frame (LRFD)

Max forces in vertical support connection

D factor = 1.2 # of connections = 4

$$T_{max} = \frac{1.2 \times W_p + F_v}{4} = \frac{1.2 \times 100\text{ lbs} + 12.12}{4} = 33.03$$

$T_{max} = 33.03\text{ lbs}$

- use Detail 10/S5.3

$T_u = 50\text{ lbs}$

$\boxed{D/C = 0.66}$

- Check Braces

Braces Resisting EA direction = 1 Brace angle (45°-60°)

- lateral load to EA Brace = F_p = 72.72 lbs
- max axial load to EA Brace = $72.72/\cos(60°) = 145.5\text{ lbs}$

Use 1/8" diameter Aircraft Cable

As = 0.0123 Fy = 42ksi Tu = As*Fy = 520 lbs $\boxed{D/C = 0.23}$

ERV Suspended Anchorage

- Unit Weight $W_p = 300$ lbs $a_p = 2.5$ $R_p = 2.5$ $I_p = 1.0$ $S_{ds} = 0.606$ $z = 14'$ $h = 14'$

$$F_p = \frac{0.4 \times a_p \times S_{ds} \times W_p}{R_p/I_p}\left(1 + 2\left(\frac{z}{h}\right)\right)$$

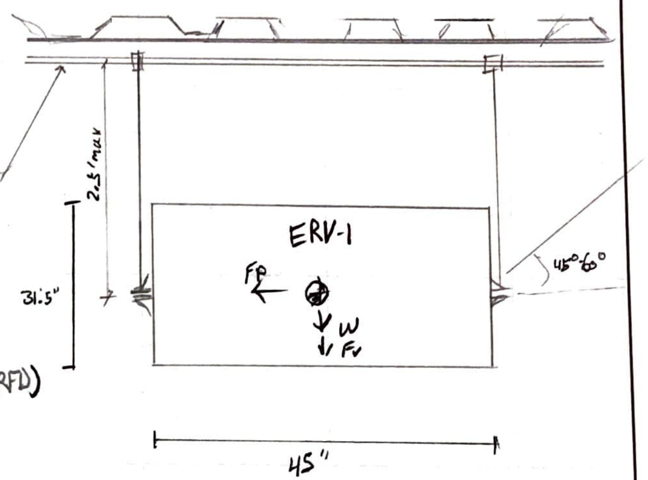
$$= \frac{0.4 \times 2.5 \times 0.606 \times 300}{2.5/1.0}\left(1 + 2 \times \frac{14}{14}\right) = 218.16$$

$F_p = 218.16$ lbs

$F_{p\,min} = 0.3 \times S_{ds} \times I_p \cdot W_p = 54.54$ lbs

$F_{p\,max} = 1.6 \cdot S_{ds} \times I_p \times W_p = 290.88$ lbs

$F_V = 0.2 \times S_{ds} \times W_p = 36.36$ lbs



- Check Anchorage to Ceiling Support frame (LRFD)

Max forces in vertical support connection to framing above
Dfactor = 1.2 # of connections = 4

$$T_{max} = \frac{1.2 \times W_p + F_V}{4} = \frac{1.2 \times 300 + 36.36}{4} = 99.09$$

$T_{max} = 99.09$ lbs

- Use Beam Clamp unistrut P2897
$T_u = 550$ lbs

$\boxed{D/C = 0.18}$

- Check Braces

- Num Braces Resisting EA direction = 1 ; Brace Angle (45°–60°)
- Lateral load to EA Brace $= F_p = 218.16$
- Max axial load to EA Brace $= 218/\cos(60°) = 436$ lbs
  T_{max} @ connection $= 218 \times \tan(60°) = 377$

Use 1/8" diameter Aircraft Cable
As = 0.0123 Fy = 42ksi Tu = As*Fy = 520 lbs

$\boxed{D/C = 0.72}$

BUEHLER

PROJECT Delano DMV

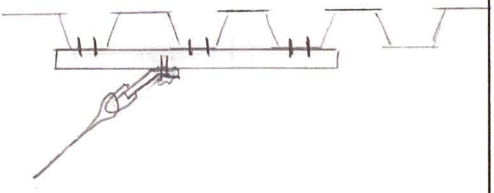
PROJECT NO. 2018-0187 DATE 4/1/2020

CLIENT NLA BY ICG SHEET NO.

E12

Wire brace connection design

- Braceing worst case ERV-1 Deck: 1 1/2" x 18 GA B Deck



max lateral load $F_p = 218$

T_{max} @ connection = tan(60) x FP = 377.86 lbs

V_{max} = 218 lbs

W/ 6 #10 SMS in 18 GA deck

#10 screw capacity in 18 GA deck

T_u = 109 x 6 = 654 lbs

V_u = 263 x 6 = 1578 lbs

$\boxed{\text{Tension D/C} = 0.58}$