Fermilab

Particle Physics Division

12-01-01

Mechanical Department Engineering Note

Number: MD-Eng-72

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Project: Linear Collider Detector

Project Internal Reference:

Title: Tail-catcher and Muon Tracker Detector Lifting Fixture and Support Frame, and Positioning Frame

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Reviewer(s): Zhijing Tang 12-7-05

Key Words:

Linear Collider Detector (LCD), Tail-catcher and Muon Tracker Detector (TCMD), Cassette, Lifting Fixture and Support Frame, and Positioning Frame.

Abstract/Summary:

The Tail-catcher and Muon Tracker Detector (TCMD) is composed of eight 4" steel plates and eight ¾" steel plates. Each plate is 43" in height and 56" in width. Between these plates, there is a 1.25" gap to permit the installation of the Cassettes. These plates are supported by the support frame which also doubles as a lifting fixture. Thus, the TCMD can be lifted by the building crane at four pick up points at the top of the Support Frame. Two of the larger pick up points are used to tilt the whole assembly 90 degrees.

On the bottom of this Lifting Fixture and Support Frame, four Hilman rollers are attached to permit horizontal movement of the assembly. This Lifting Fixture and Support Frame is supported by a Positioning Frame which also has four Hilman rollers to permit horizontal movement along the beam.

The Positioning Frame rolls on steel plates which are supported by the concrete shield blocks. These blocks are necessary to elevate the TCMD to the beam height of 65" above the existing floor in Meson Building.

The design analysis was performed according to both AISC, "Allowable Stress Design "Code and ANSI B30_20,"Below the Hook Lifting Devices". The results show that all members will safely support TCMD assembly within the allowable stress and deflection limits.

Applicable Codes:

1. AISC Manual of Steel Construction, Allowable Stress Design, Ninth Edition

2. ANSI/ASME B30.20-1993, Below the Hook Lifting Devices

3. Hilti North America Product Technical Guide

4. ACI 318-02/318R-02, Building Code and Commentary, 2002

Reference Books:

1. Mechanical Engineering Design, Fifth Edition, by Shigley and Mischke

2. Mark's Standard Handbook for Mechanical Engineers, Tenth Edition, by Avallone Baumeister

3. Design of Welded Structures, by Blodgett

4. Steel Design Manual, by R.L. Brochenbrough

5. Structural Steel Design, by Lynn S. Beedle

6. Fundamentals of Mechanical Design, Third Edition, by Phelan

Tailcatcher and Muon Tracker Detector

INGRID FAN
2/8/05

1. Design load for Lifting fixture and support frame

4" plate weight $= 43 \times 56 \times 4 \times 490/12^3 = 2731$ lbs

3/4" Plate weight $= 2731 \times \frac{0.75}{4} = 512$ lbs

Module weight $= 8 \times 2731 + 8 \times 512 = 25945$ lbs

Cassets weight $= 16\,(ea) \times 32 = 512$ lbs

Thus, Module and Cassets weight $= 25945 + 512$
$= 26457$ lbs

use 30,000 lbs as design load

2. Design analysis from Ansys

(See attached Lifting fixture and support frame sketches)

Models are built to study the following load cases

load case 1 : Two point lift from 6x6 end

load case 2 : Four point lift from horizontal (unequal legs)

load case 3 : Sitting on the Thin End after being rotated 90°.

load case 4 : Hilman Roller supports

The maximum deflection on the thinner plate is 0.01"

3. Detail design based on the FEA results

a. Column 6×6×5/16. → $A = 6.86 in^2$ $W = 23.34$ lb/ft

$I = 36.3 in^4$

$S = 12.1 in^3$

$r = 2.30$ in

$J = 58.9 in^4$

$F_x = 1678$ (S)

$F_y = 13549$ (T)

$F_z = 6255$ (S)

$M_x = 40621$ (B)

$M_y = 39657$ (Twisting)

$M_z = 32812$ (B)

Tension/compression: $f_t (f_a) = \frac{F_y}{A} = \frac{13549}{6.86} = 1.975$ ksi

$F_t = \frac{1}{3} F_y = \frac{1}{3} \times 46 = 15.3$ ksi

F_a: $\frac{kl}{r} = \frac{1 \times 43}{2.30} = 18.6 < C_c$

$C_c = 111.6$ $\frac{kl}{r} / C_c = \frac{18.6}{111.6} = 0.167$

$F_a = C_a F_y$ $C_a \to 0.570$

$= 0.570 \times 46 = 26.22$ ksi

$F_a = \frac{1}{3} F_y$

$= \frac{1}{3} \times 46 = 15.3$ ksi (governs)

Shear.

$f_{vz} = \frac{F_z}{A} = \frac{6255}{6.86} = 911.8$ psi

$f_{vx} = \frac{F_x}{A} = \frac{1678}{6.86} = 244.6$ psi

$f_s = \frac{T}{2bht} = \frac{39657}{2 \times 6 \times 6 \times 5/16} = 1762$ psi

$$f_v = \sqrt{(f_{vz} + f_s)^2 + (f_{vx} + f_s)^2}$$

$$= \sqrt{(911.8 + 1762)^2 + (244.6 + 1762)^2}$$

$$= 3.34 \text{ ksi} < F_v \quad \underline{\underline{OK}}$$

$$F_v = \frac{1}{3} F_y = \frac{1}{3} \times 46 = 15.3 \text{ ksi}$$

Bending: $f_{bx} = \dfrac{M_x}{S_x} = \dfrac{40621}{12.1} = 3.357 \text{ ksi}$

$$f_{bz} = \frac{M_z}{S_z} = \frac{32812}{12.1} = 2.7 \text{ ksi}$$

$$F_{bx} = F_{bz} = \frac{1}{3} F_y = \frac{1}{3} \times 15.3 = 15.3 \text{ ksi}$$

Combined stress:

$$\frac{f_a}{F_a} = \frac{1.975}{15.3} = 0.13 < 0.15$$

thus, $\dfrac{f_a}{F_a} + \dfrac{f_{bx}}{F_{bx}} + \dfrac{f_{bz}}{F_{bz}} \leq 1$

$$0.13 + \frac{3.357}{15.3} + \frac{2.7}{15.3} = 0.5 < 1$$

$\underline{\underline{OK}}$

$$\frac{f_t}{F_t} + \frac{f_{bx}}{F_{bx}} + \frac{f_{bz}}{F_{bz}} \leq 1$$

$$0.13 + \frac{3.357}{15.3} + \frac{2.7}{15.3} = 0.5 < 1 \quad \underline{\underline{OK}}$$

b. Column $6 \times 2 \times 5/16 \rightarrow$ $A = 4.36\ in^2$ $w = 14.83\ lb/ft$

$S_x = 5.34\ in^3$

$S_y = 2.62\ in^3$

$F_x = -4471\ (S)$

$F_y = 5963\ (T)$

$F_z = 4693\ (S)$

$M_x = 12519$ (weak B)

$M_y = 4026$ (Twisting)

$M_z = 15976$ (strong B)

Tension / Compression: $f_t (f_a) = \frac{F_y}{A} = \frac{5963}{4.36} = 1.367\ ksi$

Bending: $f_{bx} = \frac{M_z}{S_x} = \frac{15976}{5.34} = 2.991\ ksi$

$f_{by} = \frac{M_x}{S_y} = \frac{12519}{2.62} = 4.778\ ksi$

Combined stress:

$$\frac{f_a}{F_a} = \frac{1.367}{15.3} = 0.09 < 0.15$$

$$\frac{f_a}{F_a} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} \leq 1.0$$

$$0.09 + \frac{2.991}{15.3} + \frac{4.778}{15.3} = 0.6 < 1.0$$

OK

$$\frac{f_t}{F_t} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} \leq 1.0$$

$$0.09 + \frac{2.991}{15.3} + \frac{4.778}{15.3} = 0.6 < 1.0$$

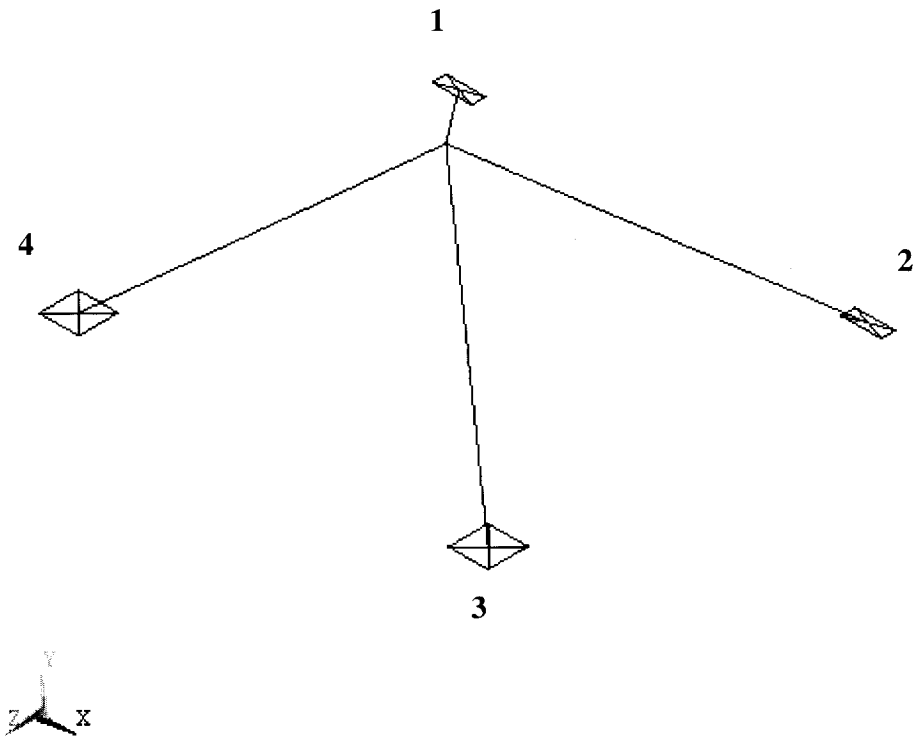
OK

B. Wands
1/28/05

The following table gives the forces acting on each pick point for the case of a four point pick with unequal sling lengths (as specified by I. Fang on January 27, 2005)

Sling Reactions at Pick Points

Point	FX	FY	FZ
1	7056	6278	7368
2	-7029	6247	7342
3	-9772	8878	-7378
4	9746	8855	-7331



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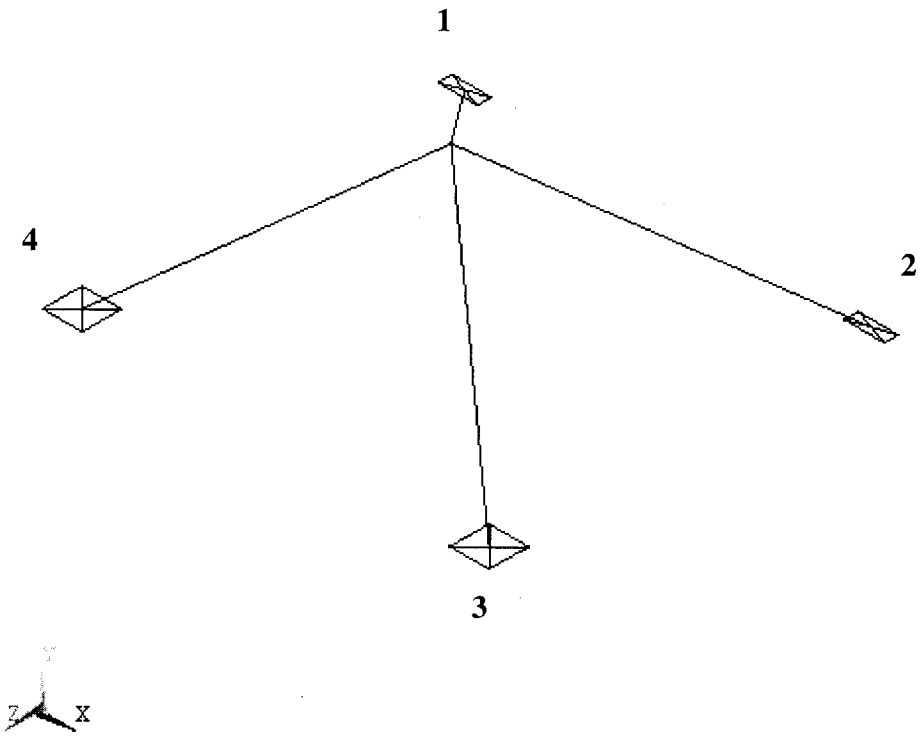
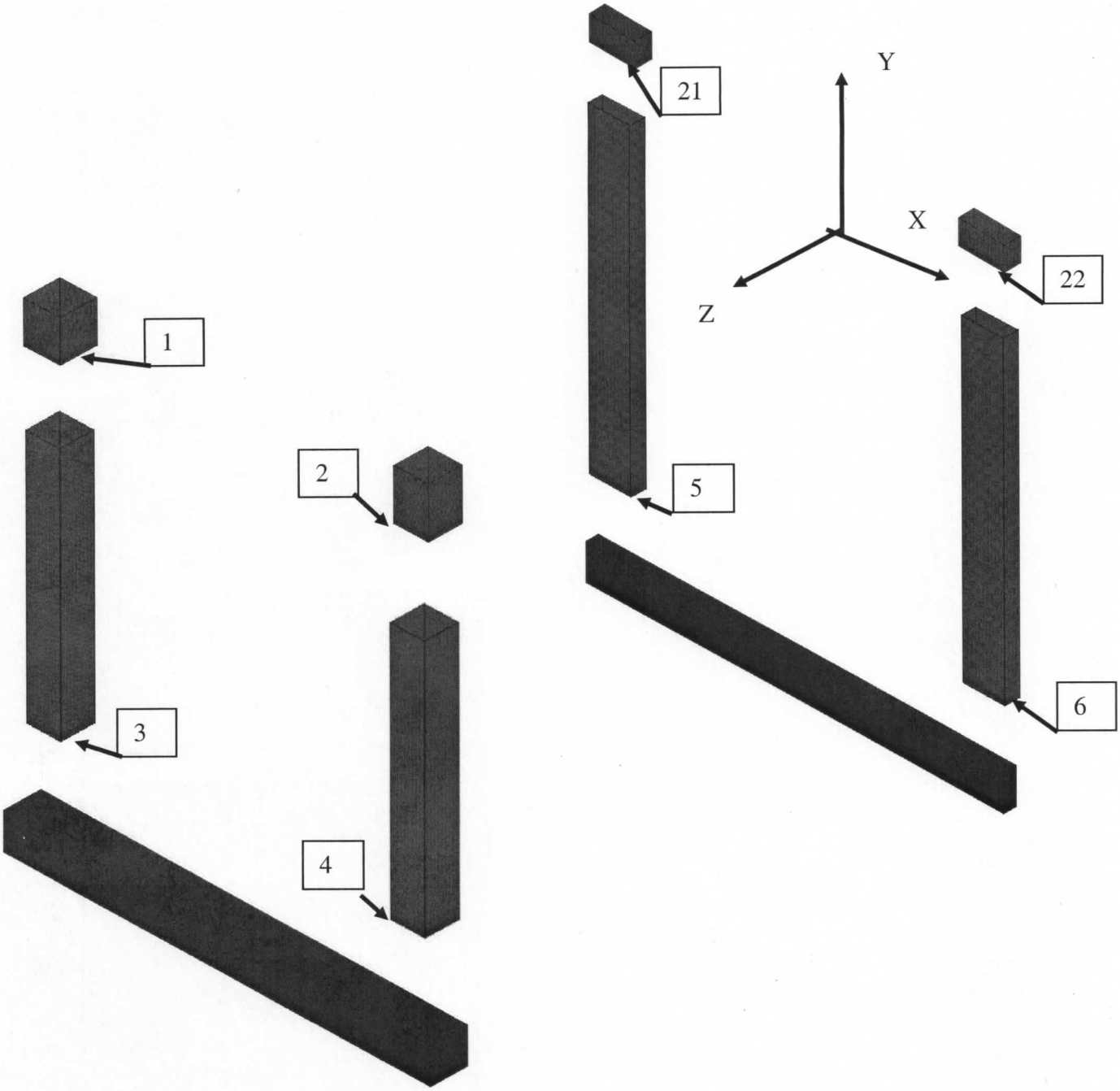




Now, use the reactions from the table to calculate the forces and moments on sections 21 and 22 (from the sling reactions on Points 1 and 2 in the table). Note: Sling reactions are not quite symmetric. Since Point 1 has slightly higher loads, it will be used to evaluate the section.

For the calculation of moments, a lever arm of 3 inches is assumed.

At section 21, the forces are the same as the sling reactions:

Fx = 7056 lbs
Fy = 6278 lbs
Fz = 7368 lbs

The bending moments are:

Mx = 3Fz = 22104 in-lbs
My = 0 (no basis for calculating the torsion)
Mz = 3Fx = 21168 in-lbs

Section properties for a 6x2x0.3125 wall tube:

A = 5.08 in^2
Sxx = 2.62 in^3
Szz = 5.34 in^3

Stress from normal force Fy = 6278/A = 1236 psi
Stress from bending moment Mx = 22104/Sxx = 8437 psi
Stress from bending moment Mz = 21168/Szz = 3964 psi

The total normal stress on Section 21 is the sum of these three stresses:

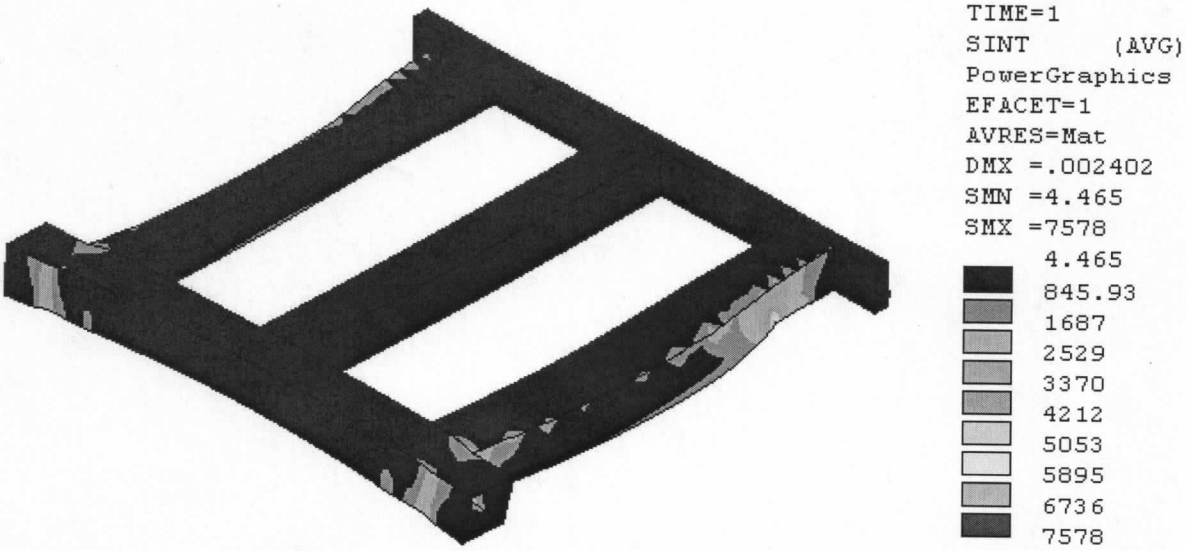
1236 + 8437 + 3964 = 13637 psi

The shear stress in the xz plane is caused by Fx and Fz. The vector sum of these forces is 10201 lbs. The shear area is A = 5.08 in^2. Therefore

The shear stress on Section 21 is:

10201/5.08 = 2008 psi

1	-100	7	21	104	188	376
2	102	-257	-23	-1357	-320	154
3	589	-3915	324	44	-2511	-8274
4	-432	-3698	450	944	2206	7700
5	172	-326	-60	206	383	-1693
6	-148	-343	-93	29	-342	1596
7	25	-68	-123	1806	-14	295
8	-9	-68	-142	1946	27	-235
9	146	-185	63	2196	216	-39
10	-140	-202	48	2880	-213	-398
11	205	-1363	355	-2099	-502	-3630
12	-220	-1261	341	-2128	400	3298
13	305	-1781	-250	-1388	-140	-4271
14	-171	-1962	-28	-1865	369	5088
15	156	427	75	371	-479	2083
16	-13	-70	-4	86	-270	22
17	-166	419	82	583	-107	-1195
18	209	-2962	-71	-750	1875	8347
19	13	-55	4	307	-427	-22
20	-160	-3140	-170	-1996	-1575	-8575
21	0	-11	0	0	0	0
22	0	-5	0	0	0	0



Without Extra Plate

Load Case 1 - Two point lift from 6x6 end

Joint	FFX	FFY	FFZ	MMX	MMY	MMZ
1	-810	13547	6254	40615	39669	-32803
2	1680	6855	-550	17293	-17186	15151
3	-622	441	-25	1454	1862	-651
4	602	345	106	1130	-923	177
5	-182	193	-16	-70	87	553
6	126	228	22	12	-24	-750
7	-132	246	150	-1704	131	-2612
8	144	281	164	-1847	-74	2750
9	-50	-1	-114	-1570	147	-2018
10	-18	-1	-89	-2025	-83	2090
11	-1981	-7497	-10334	-30453	-8514	-11611
12	3748	-5535	-10767	-24730	-2021	-5392
13	275	-2816	510	-3161	-5782	-4167
14	-1578	-2518	647	-4027	4858	3665
15	-73	-144	66	-19	-273	146
16	-3	-38	-16	-140	-92	-116
17	132	-163	19	-164	231	-575
18	98	-431	539	1111	-244	1238
19	3	-50	-73	-186	-50	116
20	-565	-150	481	1129	553	1201
21	0	-8	-8	-28	1	1
22	0	-4	-4	-9	0	1

Load Case 2 - Four point lift from horizontal

1	1268	9318	-3709	-4873	-7200	-12220
2	840	6371	-134	-2841	-6175	6703
3	-623	1185	292	2368	-326	2489
4	444	966	328	2114	376	-2188
5	-984	2244	-1071	-2357	2241	3572
6	752	2366	-740	-2965	-552	-4386
7	-6608	954	-6114	-27272	4990	25423
8	6925	524	-6947	-19239	-8501	-17785

9	166	-4447	-383	8704	4116	-13025
10	-91	-4365	-20	10715	-2963	10962
11	-2106	-2596	7521	9044	-3120	-1003
12	1964	-1980	6528	4847	1803	-3401
13	5	-3877	-776	-4057	-1237	-11280
14	-614	-3133	-85	-2330	2186	10924
15	233	-2194	1122	3526	2887	5514
16	-10	-71	-84	-734	-188	-581
17	9	-2238	773	3234	-3077	-7521
18	248	-1631	-719	-2383	247	4142
19	10	-55	84	1164	-378	581
20	-132	-1503	-900	-2475	-272	-2960
21	5458	5798	5426	24064	-100	-33243
22	-6082	6276	6228	12264	3735	17098

Load Case 3 - Sitting on Thin End

1	-172	-39	-38	-261	636	277
2	169	-48	27	-220	-460	-38
3	-62	-111	-97	-26	1123	-295
4	74	-90	-50	112	-876	223
5	-110	-4	30	-41	-379	65
6	56	30	39	-13	309	-31
7	-198	67	-7600	-482	1921	-15
8	208	116	-6733	-332	-1301	89
9	-193	-76	-4735	-825	1547	-138
10	170	-48	-5335	-613	-1728	166
11	1149	-214	855	-353	-2262	-316
12	-1121	-385	764	-689	2105	332
13	952	188	442	199	-1278	362
14	-975	29	508	-10	1388	-83
15	-1	-32	-2410	-773	109	154
16	-4	13	-514	-822	-105	-26
17	18	-41	-2687	-806	-53	-295
18	-47	201	217	829	539	-519
19	4	-13	389	103	-120	26
20	38	195	228	953	-765	591
21	98	-427	606	522	98	-189
22	-125	-395	484	496	-192	28

With removable beam moved upward to be flush with plates

4107	4726	-1783	-9437	-14201	-13559
1166	6728	-83	-3076	-5551	8158
-519	1028	271	2144	2	2200
443	999	302	2140	391	-2379
-842	2025	-997	-2103	1941	3216
703	2109	-663	-2658	-658	-4200
-5151	-78	-5168	-18097	8128	12923
5259	214	-5340	-17166	-7722	-13355
-162	-3790	-522	7700	3468	-9718
-92	-3976	-3	9585	-2386	10734
-2506	-2652	5490	3061	-2243	1093
2530	-2370	5490	3033	960	-2449
172	-3317	-140	-2516	-1941	-10298
-294	-3565	-90	-3147	1889	10965
203	-1953	1049	3392	2730	5369
-6	-71	-80	-696	-151	-51
-58	-2004	691	2815	-2602	-6044
211	-1450	-680	-2023	483	3222
6	-55	80	1116	-156	51
-112	-1545	-776	-2393	91	-3717
4471	5963	4693	12517	-4077	-14794
-4464	5939	4680	13679	4024	15976

With Extra Plate

FFX	FFY	FFZ	MMX	MMY	MMZ
-806	13549	6255	40621	39657	-32812
1678	6852	-552	17268	-17170	15143
-622	440	-26	1453	1862	-651
601	345	106	1129	-924	177
-179	177	-9	-57	61	509
124	212	25	26	-14	-726
-78	181	149	-1753	137	-3812
96	230	164	-1891	-72	3688
-81	42	-106	-1568	116	-1795
8	40	-86	-2019	-54	1908
-1980	-7499	-10333	-30487	-8519	-11528
3747	-5537	-10765	-24758	-2015	-5456
275	-2813	509	-3158	-5784	-4170
-1578	-2517	645	-4025	4859	3664
-69	-128	59	-39	-292	90
-3	-38	-16	-137	-91	-116
127	-148	16	-173	245	-516
98	-431	539	1112	-243	1235
3	-50	-73	-187	-49	116
-565	-149	481	1130	554	1203
-178	-143	-11	121	-175	329
142	-66	-6	62	132	-137

1278	9352	-3607	-5471	-6832	-12428
843	6610	-85	-4154	-6349	6944
-620	1187	301	2416	-351	2488
445	991	328	2126	325	-2185
-855	1911	-1003	-2087	2183	3431
622	1992	-687	-2634	-736	-4234
-221	10	-6364	-30576	5631	6934
634	-496	-6997	-21056	-9320	-5854

-185	-3788	-251	9602	2810	-10054
-42	-3760	60	10959	-1867	9777
-1899	-2550	7593	8873	-3921	358
2050	-1949	6813	5022	2510	-5012
-6	-3837	-805	-3934	-1270	-11503
-636	-3221	-175	-2662	2303	10796
366	-1854	1046	3412	2515	4232
-6	-72	-75	-683	-100	-408
-128	-1871	719	3074	-2495	-5786
229	-1655	-761	-2426	340	4131
6	-54	75	1177	-224	408
-162	-1517	-879	-2522	-67	-3131
1095	9883	4772	20803	-1461	-228
-2303	7787	5484	10590	4921	5352

-172	-39	-38	-261	636	277
169	-48	27	-221	-460	-38
-62	-111	-97	-26	1124	-296
74	-90	-50	112	-876	224
-110	0	27	-47	-374	68
57	34	37	-19	308	-38
-275	85	-7601	-619	1827	17
276	129	-6732	-432	-1212	173
-194	-86	-4731	-794	1542	-162
175	-56	-5332	-588	-1720	183
1149	-214	855	-353	-2262	-316
-1121	-385	764	-691	2105	329
952	188	442	200	-1278	359
-975	29	508	-8	1388	-81
0	-38	-2414	-739	109	171
-4	13	-514	-813	-104	-26
22	-47	-2691	-776	-46	-324
-47	200	217	829	540	-516
4	-13	389	105	-119	26
38	195	228	953	-766	588
130	-394	527	427	368	-122
-124	-371	428	418	-340	5

Shear: $f_{vx} = \frac{f_x}{A} = \frac{4471}{4.36} = 1.03$ ksi

$f_{vz} = \frac{f_z}{A} = \frac{4693}{4.36} = 1.1$ ksi

$f_s = \frac{T}{2bht} = \frac{4024}{2 \times 2 \times 6 \times 5/16} = 0.5$ ksi

$f_v = \sqrt{(f_{vx} + f_s)^2 + (f_{vz} + f_s)^2}$

$= \sqrt{(1.03 + 0.5)^2 + (1.1 + 0.5)^2}$

$= 2.2$ ksi $< F_v = 15.3$ ksi

OK

C Bottom Frame members

C1: Column 6x6x5/16 tie member 6x6x5/16

$F_x = 622$ (C)

$F_y = 1187$ (S)

$F_z = 328$ (S)

$M_x = 2416$ (T)

$M_y = 1862$ (B)

$M_z = 2488$ (B)

Compression: $f_a = \frac{F_x}{A} = \frac{622}{6.86} = 0.09$ ksi

Bending: $f_{by} = \frac{M_y}{S_y} = \frac{1862}{12.1} = 0.15$ ksi

$f_{bz} = \frac{M_z}{S_z} = \frac{2488}{12.1} = 0.2$ ksi

Combined stress:

$$\frac{f_a}{F_a} \quad \frac{0.09}{15.3} = 0.006 < 0.15$$

$$\frac{f_a}{F_a} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} \leq 1.0$$

$$0.006 + \frac{0.15}{15.3} + \frac{0.2}{15.3} = 0.03 < 1.0 \quad \underline{\underline{OK}}$$

shear:

$$f_{vz} = \frac{f_z}{A} = \frac{328}{6.86} = 0.05 \text{ ksi}$$

$$f_{vy} = \frac{f_y}{A} = \frac{1187}{6.86} = 0.2 \text{ ksi}$$

$$f_s = \frac{T}{2bht} = \frac{m_x}{2bht} = \frac{2416}{2 \times 6 \times 6 \times 5/16} = 0.1 \text{ ksi}$$

$$f_v = \sqrt{(f_{vz} + f_s)^2 + (f_{vy} + f_s)^2}$$

$$= \sqrt{(0.05 + 0.1)^2 + (0.2 + 0.1)^2}$$

$$= 0.33 \text{ ksi} < F_v = 15.3 \text{ ksi}$$

$\underline{\underline{OK}}$

C2. Column 2x6x5/16 tie member 2x6x5/16

$F_x = -855$ (C)

$F_y = 1992$ (S)

$F_z = -1003$ (S)

$m_x = 2634$ (T)

$m_y = 2183$ (B)

$m_z = -4234$ (B)

Compression: $f_a = \frac{F_X}{A} = \frac{855}{4.36} = 0.2\ ksi$

Bending: $f_{by} = \frac{M_Y}{S_Y} = \frac{2183}{262} = 0.8\ ksi$

$f_{bz} = \frac{M_Z}{S_Z} = \frac{4234}{5.34} = 0.8\ ksi$

Combined stress:

$$\frac{f_a}{F_a} = \frac{0.2}{15.3} = 0.01 < 0.15$$

$$\frac{f_a}{F_a} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} \le 1.0$$

$$0.01 + \frac{0.8}{15.3} + \frac{0.8}{15.3} = 0.12 < 1.0$$

OK

Shear: $f_{vy} = \frac{F_Y}{A} = \frac{1992}{4.36} = 0.5\ ksi$

$f_{vz} = \frac{F_Z}{A} = \frac{1003}{4.36} = 0.2\ ksi$

$f_s = \frac{T}{2bht} = \frac{2634}{2 \times 2 \times 6 \times 5/16} = 0.5\ ksi$

$$f_v = \sqrt{(f_{vy} + f_s)^2 + (f_{vz} + f_s)^2}$$

$$= \sqrt{(0.5 + 0.5)^2 + (0.2 + 0.5)^2}$$

$$= 1.22\ ksi < F_v = 15.3\ ksi$$

OK

C3. Three bottom members 6x6x5/16

$F_x = 2303$ (S)

$F_y = 9883$ (S)

$F_z = 5484$ (T)

$M_x = 20803$ (B)

$M_y = 4921$ (B)

$M_z = 5352$ (Twisting)

Tension: $f_t = \frac{F_z}{A} = \frac{5484}{6.86} = 0.8$ ksi

Bending: $f_{bx} = \frac{M_x}{S_x} = \frac{20803}{12.1} = 1.7$ ksi

$f_{by} = \frac{M_y}{S_y} = \frac{4921}{12.1} = 0.4$ ksi

Combined stress: $\frac{f_t}{F_A} + \frac{f_{bx}}{F_{bY}} + \frac{f_{by}}{F_{bY}} \leq 1.0$

$\frac{0.8}{15.3} + \frac{1.7}{15.3} + \frac{0.4}{15.3} = 0.2 < 1.0$

OK

Shear: $f_{vx} = \frac{F_x}{A} = \frac{2303}{6.86} = 0.3$ ksi

$f_{vy} = \frac{F_y}{A} = \frac{9883}{6.86} = 1.4$ ksi

$f_s = \frac{5352}{2 \times 6 \times 6 \times 5/16} = 0.23$ ksi

$f_v = \sqrt{(f_{vx} + f_s)^2 + (f_{vy} + f_s)^2}$

$= \sqrt{(0.3 + 0.23)^2 + (1.4 + 0.23)^2} = 1.7$ ksi $< F_v = 15.3$ ksi

OK

D. Side members 6x6x 5/16

$F_x = 3747\ (S)$

$F_y = 7499\ (S)$

$F_z = -10765\ (T)$

$m_x = 30576\ (B)$

$m_y = 9320\ (B)$

$m_z = 11528$ (Twisting)

Tension: $f_t = \frac{F_z}{A} = \frac{10765}{6.86} = 1.09$ ksi

Bending: $f_{bx} = \frac{m_x}{S_x} = \frac{30576}{12.1} = 2.5$ ksi

$f_{by} = \frac{m_y}{S_y} = \frac{9320}{12.1} = 0.7$ ksi

Combined stress:

$$\frac{f_t}{F_t} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} \leq 1.0$$

$$\frac{1.09}{15.3} + \frac{2.5}{15.3} + \frac{0.7}{15.3} = 0.28 < 1.0$$

OK

Shear: $f_{vx} = \frac{F_x}{A} = \frac{3747}{6.86} = 0.5$ ksi

$f_{vy} = \frac{F_y}{A} = \frac{7499}{6.86} = 1.09$ ksi

$f_s = \frac{T}{2bht} = \frac{11528}{2 \times 6 \times 6 \times 5/16} = 0.5$ ksi

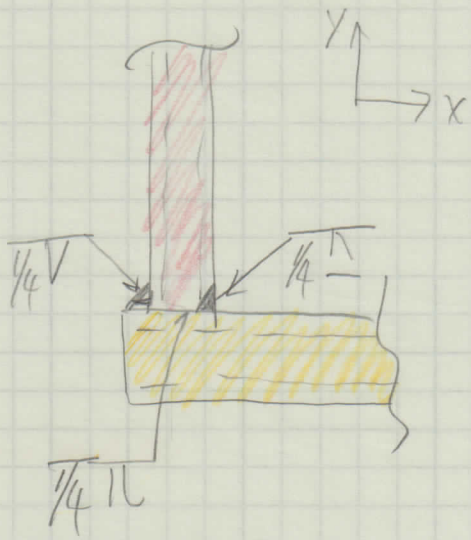
$$f_v = \sqrt{(f_{vx}+f_s)^2 + (f_{vy}+f_s)^2}$$

$$= \sqrt{(0.5+0.5)^2 + (1.09+0.5)^2}$$

$$= 1.87 \text{ ksi} < F_v = 15.3 \text{ ksi} \quad \underline{\underline{OK}}$$

E. Connection Design:

E1. Connection at ③ & ④



$F_x = 622$ (S)

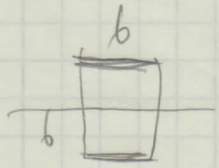
$F_y = 1187$ (T)

$F_z = 328$ (S)

$M_x = 2466$ (B)

$M_y = 1862$ (Twisting)

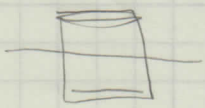
$M_z = 2488$ (B)



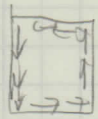
$$f_{vx} = \frac{622}{b+b} = 52 \text{ lb/in}$$

$$f_t = \frac{1187}{4 \times b} = 50 \text{ lb/in}$$

$$f_{vz} = \frac{328}{b+b} = 27 \text{ lb/in}$$



$$f_t' = \frac{2466}{48} = 50 \text{ lb/in} \quad \text{where } S = bd + \frac{d^3}{3} = b \times b + \frac{b^3}{3} = 48 \text{ in}^2$$



$$f_s = \frac{1862}{Jw} = \frac{1862}{288} = 6 \text{ lb/in} \quad \text{where } J = \frac{(b+d)^3}{6} = \frac{(6+6)^3}{6} = 288 \text{ in}^2$$

$$f_t'' = \frac{2488}{Sw} = \frac{2488}{48} = 52 \text{ lb/in}$$

resultant force:

$$f_r = \sqrt{(f_t + f_t' + f_t'')^2 + (f_{vx} + f_s)^2 + (f_{vz} + f_s)^2}$$

$$= \sqrt{(50+50+52)^2 + (52+6)^2 + (27+6)^2}$$

$$= 166 \text{ lb/in} < 2.7 \text{ kip/in} \quad \underline{\underline{\text{OK}}}$$

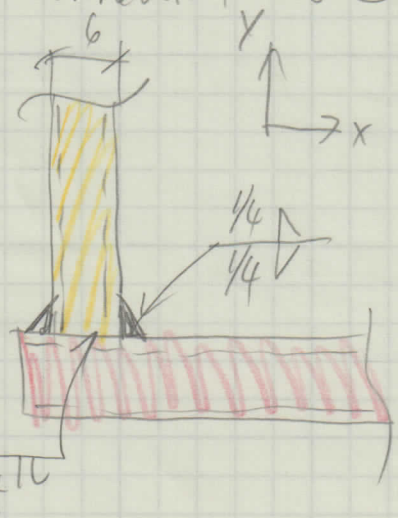
Allowable on the welds:

$$e = \frac{5}{8} \times T_1 = \frac{5}{8} \times 5/16 = 0.19 \text{ in}$$

$$e = 0.707 \times 1/4 = 0.17 \text{ in (governs)}$$

$$15.3 \times 0.17 = 2.7 \text{ kip/in}$$

E2. Connection at 5 & 6



$F_x = 855$ (S)

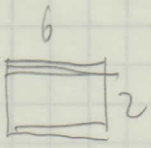
$F_y = 1922$ (T)

$F_z = 1003$ (S)

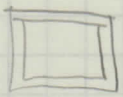
$M_x = 2634$ (B)

$M_y = 2183$ (Twisting)

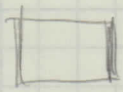
$M_z = 4234$ (B)



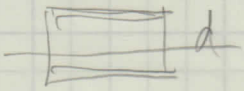
$$f_{vx} = \frac{855}{6+6} = 71 \text{ lb/in}$$



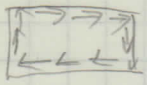
$$f_t = \frac{1922}{(6+2)\times 2} = 120 \text{ lb/in}$$



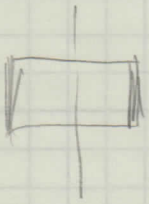
$$f_{vz} = \frac{1003}{(2+2)} = 250 \text{ lb/in}$$



$$f_t' = \frac{2634}{S_w} = \frac{2634}{146} = 179 \text{ lb/in} \quad \text{where } S_w = bd + \frac{d^3}{3} = 2\times 6 + \frac{2^3}{3} = 146 \text{ in}^2$$



$$f_s = \frac{2183}{J_w} = \frac{2183}{85} = 26 \text{ lb/in} \quad \text{where } J_w = \frac{(b+d)^3}{6} = \frac{(2+6)^3}{6} = 85 \text{ in}^2$$



$$f_t'' = \frac{4234}{S_w} = \frac{4234}{84} = 50 \text{ lb/in} \quad \text{where } S_w = bd + \frac{d^3}{3} = 2\times 6 + \frac{6^3}{3} = 84 \text{ in}^2$$

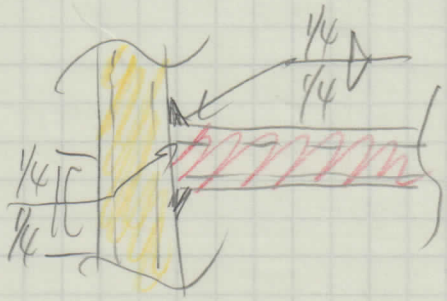
$$f_r = \sqrt{(f_t + f_t' + f_t'')^2 + (f_{vx} + f_s)^2 + (f_{vz} + f_s)^2}$$

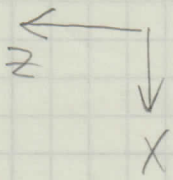
$$= \sqrt{(120+179+50)^2 + (71+26)^2 + (250+26)^2}$$

$$= 455 \text{ lb/in} < \text{Allowable } 217 \text{ kip/in}$$

OK

E3 Connection at 8, 10, 12 & 14 (fix side)





$F_x = 3747\ (S)$

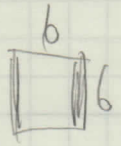
$F_y = 7499\ (S)$

$F_z = 10765\ (T)$

$M_x = 30576\ (B)$

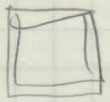
$M_y = 9320\ (B)$

$M_z = 11528$ (Twisting)



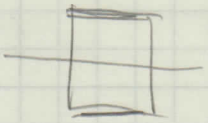
$$f_{vx} = \frac{3747}{6t_b} = 312\ \text{lb/in}$$

$$f_{vy} = \frac{7499}{6t_b} = 625\ \text{lb/in}$$

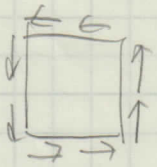


$$f_t = \frac{10765}{6 \times 4} = 448\ \text{lb/in}$$

$$f_t' = \frac{30576}{48} = 637\ \text{lb/in}$$



$$f_t'' = \frac{9320}{48} = 194\ \text{lb/in}$$



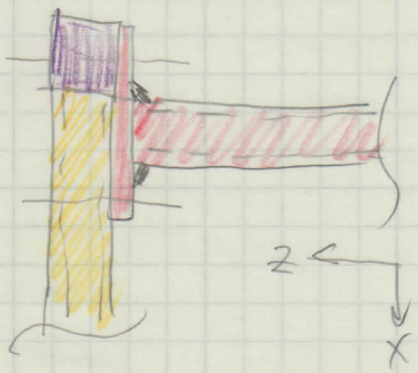
$$f_{vs} = \frac{11528}{288} = 40\ \text{lb/in}$$

$$f_r = \sqrt{(f_t + f_t' + f_t'')^2 + (f_{vx} + f_{vs})^2 + (f_{vy} + f_{vs})^2}$$

$$= \sqrt{(448 + 637 + 194)^2 + (312 + 40)^2 + (625 + 40)^2}$$

$$= 1.48 \text{ kip/in} < \text{Allowable} = 2.7 \text{ kip/in}$$ OK

E4. Connection at ⑦ ⑨ ⑪ & ⑬ (removable side)



$F_x = 3747$ (S)

$F_y = 7499$ (S)

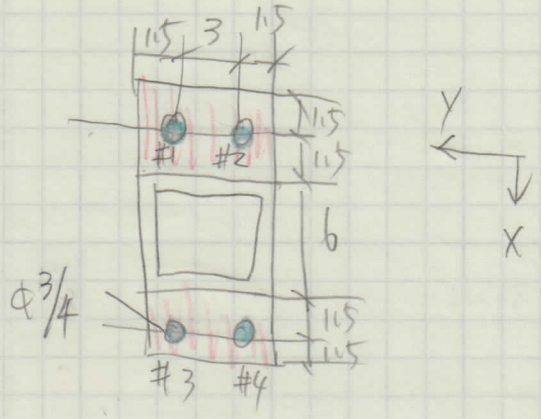
$F_z = 10765$ (T)

$m_x = 30576$ (B)

$m_y = 9320$ (B)

$m_z = 11528$ (Twisting)

— Bolted Joints:



$F_x = 3747(S)/4 = 936$ lb

$F_y = 7499(S)/4 = 1875$ lbs

$F_z = 10765(T)/4 = 2691$ lbs

$m_x = 30576$ (B)

#1 & #3 (T) $\frac{30576}{3} = 10192$ lbs
(z direction)

$m_y = 9320$ (B)

#3 & #4 (T) $\frac{9320}{3+6} = 1035$ lbs
(z direction)

$M_z = 11528$ (Twisting)

#1, #2, #3 & #4 (shear) $= \frac{11528}{6+3} = 1281$ lbs
(y-direction)

* f_v total shear $= \sqrt{f_{vx}^2 + f_{vy}^2 + f_{vz}^2}$

$= \sqrt{936^2 + 1875^2 + 1281^2}$

$= 2.456$ kips > 9.3 kips ok

Allowable shear for ϕ 3/4 $= 9.3$ kips

* f_t total Tension $= 2691 + 10192 + 1035$

$= 13918$ lbs

Allowable tension for ϕ 3/4 $= \sqrt{(44)^2 - 4.39 f_v^2}$

$= \sqrt{44^2 - 4.39 \times \left(\frac{2.456}{0.4418}\right)^2}$

$= \sqrt{44^2 - 4.39 \times 309}$

$= 42.4$ ksi

Actual tension on Bolt $= \frac{13918}{0.4418} = 31.5$ ksi

< 42.4 ksi

ok

* Bearing:

$f_p = \frac{f_v}{A} = \frac{2.456}{2.06}$ where $A = \frac{2\pi D}{2} \times t$

$= 1.19$ ksi $= \frac{2\pi \times 0.75}{2} \times 0.875$

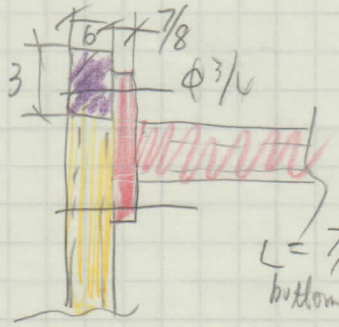
$< F_p = 12$ ksi ok $= 2.06$ in^2

Allowable $F_p = \frac{1}{3} F_y$ for A36

$= \frac{1}{3} \times 36 = 12 ksi$

* Bolt length:

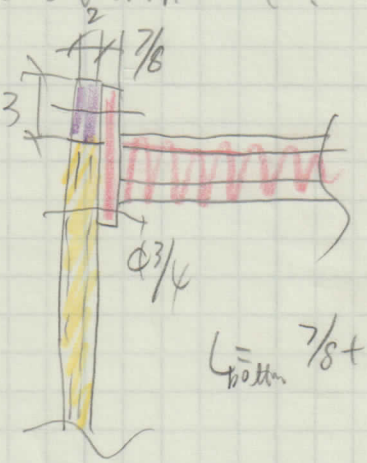
for 6x6x5/16 column side:



$L_{top} = 7/8 + 2.375$
$= 3.25''$ ~ use 3" Length

$L_{bottom} = 7/8 + 6 + 1 = 7\frac{7}{8}''$
use 8"

for 6x2x5/16 column side:



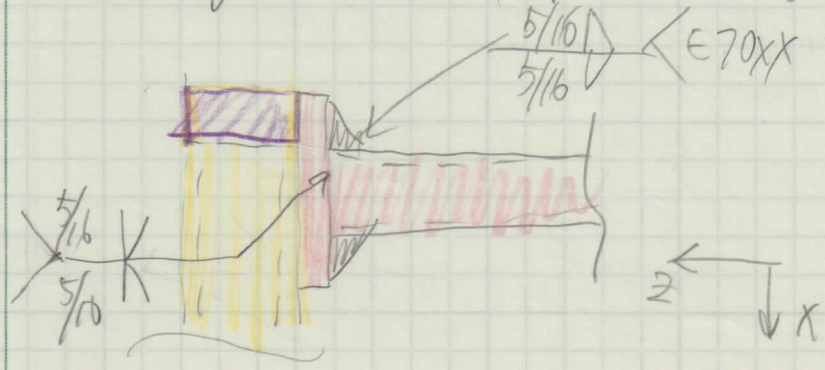
$L_{top} = 7/8 + 2$
$= 2.875''$

use 3" length

$L_{bottom} = 7/8 + 2 + 1 = 3\frac{7}{8}''$
use 4"

lock washer will be needed

= Design the tube & plate connections:



$F_x = 3747\ (S)$

$F_y = 7499\ (S)$

$F_z = 10765\ (C)$

$m_x = 30576\ (B)$

$m_y = 9320\ (B)$

$m_z = 11528\ (\text{Twisting})$

* check Bending on the plate : (T-1) Alloy material

$$f_{bx} = \frac{M_x}{S_x} = \frac{30576}{S_x} \quad \text{where } S_x = \frac{ab^2}{6} = \frac{12 \times 0.875^2}{6} = 1.53\ in^3$$

$$= \frac{30576}{1.53}$$

$$= 19.968\ ksi$$

$$f_{by} = \frac{M_y}{S_y} = \frac{9320}{S_y} \quad \text{where } S_y = \frac{ab^2}{6} = \frac{6 \times 0.875^2}{6} = 0.76\ in^3$$

$$= \frac{9320}{0.76}$$

$$= 12.173\ ksi$$

$$F_{bx} = F_{by} = \frac{1}{3} F_y = \frac{1}{3} \times 100 = 33.3\ ksi$$

$$\frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} \leq 1.0 .$$

$$\frac{19.968}{33.3} + \frac{12.173}{33.3} = 0.965 < 1.0$$ ok

* check Bearing on the plate:

$P = 10765$ lbs.

$$t_p = 2n\sqrt{\frac{f_p}{F_y}}$$

$$= 2 \times 3 \times \sqrt{\frac{f_p}{F_y}}$$ where $f_p = \frac{10765}{14 \times 6}$

$= 128$ psi

$= 0.214 < 7/8''$

$F_y = 100$ ksi

ok

* check welding

From P14 know: $f_r = 1.48$ kip/in < 3.36 kip/in

ok

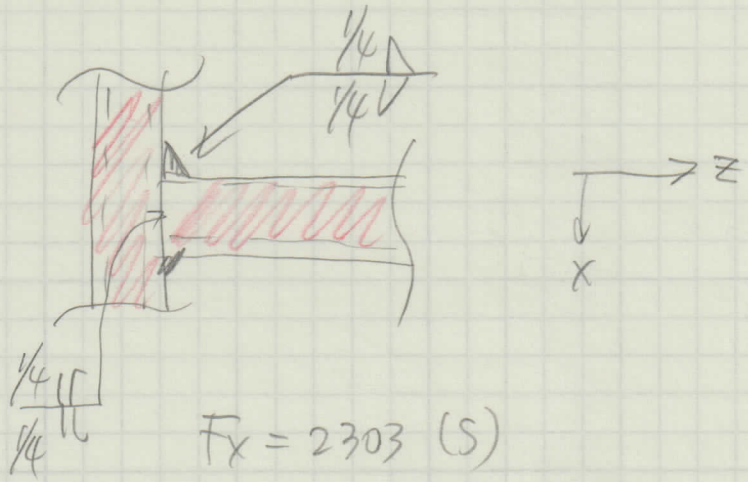
Allowable on weld:

$e = 5/16 \times 0.707 = 0.22$ in (governs)

$e = 5/16 = 0.3125$ in

$15.3 \times 0.22 = 3.366$ kip/in

F. CONNECTION at (15) (16) (17) (18) (19) (20)



$F_x = 2303$ (S)

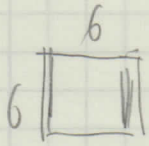
$F_y = 9883$ (S)

$F_z = 5484$ (T)

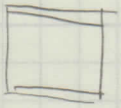
$M_x = 20803$ (B)

$M_y = 4921$ (B)

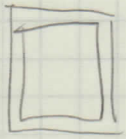
$M_z = 5352$ (Twisting)



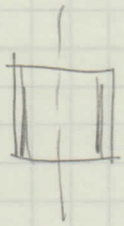
$$f_{vx} = \frac{2303}{6+6} = 192 \text{ lb/in}$$



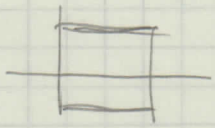
$$f_{vy} = \frac{9883}{6+6} = 823 \text{ lb/in}$$



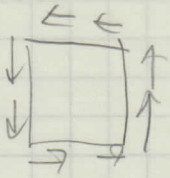
$$f_t = \frac{5484}{6 \times 4} = 229 \text{ lb/in}$$



$$f_t' = \frac{20803}{S_w} = \frac{20803}{48} = 433 \text{ lb/in}$$



$$f_t'' = \frac{4921}{8W} = \frac{4921}{48} = 103 \text{ lb/in}$$



$$f_{vs} = \frac{5352}{3W} = \frac{5352}{288} = 18.5 \text{ lb/in}$$

$$f_r = \sqrt{(f_t + f_t' + f_t'')^2 + (f_{vx} + f_{vs})^2 + (f_{vy} + f_{vs})^2}$$

$$= \sqrt{(229 + 433 + 103)^2 + (192 + 18.5)^2 + (823 + 18.5)^2}$$

$$= 1.156 \text{ kip/in} < 2.7 \text{ kip/in}$$ OK

G. Hoist Ring at ① & ② points (6x6x5/16 end)

Forged steel 1¼"-7 Capacity = 15000 lbs

L = 2.63"
D = 9.22"
E = 1.25"

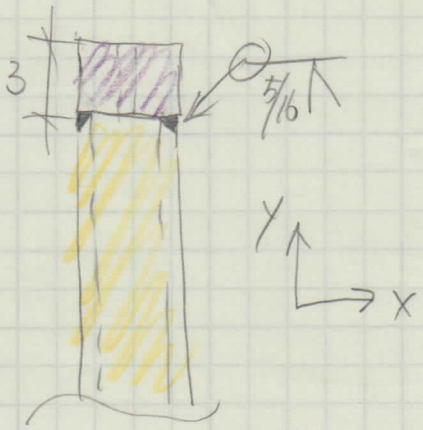
$ 207.90

This end will be used to lift the module at two pick up points in order to rotate the entire load.

30000/2 = 15000 lbs = Hoist Ring Capacity

OK

H. Design connections at ① & ② point.



$F_x = 1678$ (S)

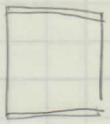
$F_y = 13549$ (T)

$F_z = 6255$ (S)

$M_x = 40621$ (B)

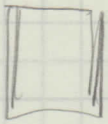
$M_y = 39657$ (Twisting)

$M_z = 32812$ (B)

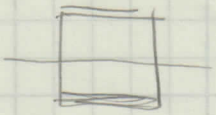


$$f_{vx} = \frac{1678}{6+6} = 140 \text{ lb/in}$$

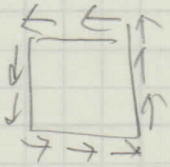
$$f_t = \frac{13549}{6 \times 4} = 564 \text{ lb/in}$$



$$f_{vz} = \frac{6255}{6+6} = 521 \text{ lb/in}$$



$$f_t' = \frac{40621}{S_w} = \frac{40621}{48} = 846 \text{ lb/in}$$



$$f_{vs} = \frac{39657}{J_w} = \frac{39657}{288} = 138 \text{ lb/in}$$

$$f_t'' = \frac{32812}{S_w} = \frac{32812}{48} = 684 \text{ lb/in}$$

$$f_v = \sqrt{(f_t + f_t' + f_t'')^2 + (f_{vx} + f_{vs})^2 + (f_{vz} + f_{vs})^2}$$

$$= \sqrt{(564 + 846 + 684)^2 + (140 + 138)^2 + (521 + 138)^2}$$

$$= 2.2 \text{ kip/in} < 3.36 \text{ kip/in} \quad \underline{\underline{\text{ok}}}$$

I. check hoist ring mounting plate: (Large end)

Tension: $f_t = \frac{15000}{6 \times 6} = 0.41 \text{ ksi} < F_t = \frac{1}{3} F_y = 12 \text{ ksi}$

Bending: $f_{bx} = \frac{40621}{S_x}$ $\quad S_x = \frac{ab^2}{6} = \frac{6 \times 3^2}{6} = 9 \text{ in}^3$

$$= \frac{40621}{9}$$

$$= 4.5 \text{ ksi}$$

$$f_{bz} = \frac{32819}{S_y} = \frac{32819}{S_x} = \frac{32819}{6} = 3.6 \text{ ksi}$$

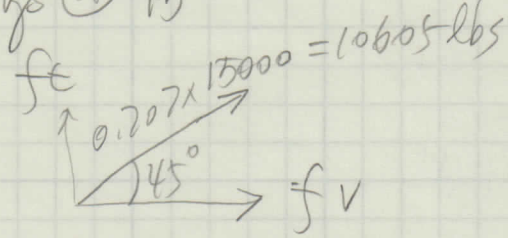
Combined stresses,

$$\frac{f_t}{F_t} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} \leq 1.0$$

$$\frac{0.41}{12} + \frac{4.5}{12} + \frac{3.6}{12} = 0.73 < 1.0 \quad \underline{\underline{\text{ok}}}$$

J. check Hoist Ring Bolt $\phi 1\frac{1}{4}''$

* Slings @ 45°



Actual $f_v = 10605 \times \cos 45° = 7.498$ kips < 25.8 kips

OK

Allowable shear for $\phi 1\frac{1}{4}'' = 25.8$ kips

Allowable Tension for $\phi 1\frac{1}{4}''$:

$$F_t = \sqrt{(44)^2 - 4.39 f_v^2}$$

where $f_v = \frac{7.498}{1.227} = 6.1$ ksi

$$= \sqrt{(44)^2 - 4.39 \times 6.1^2}$$

$$= 42 \text{ ksi}$$

Actual Tension:

$$f_t = \frac{10605 \times \sin 45°}{1.227} = 6.1 \text{ ksi} < 42 \text{ ksi}$$

OK

* Slings @ 90°

Actual tension = 15000 lbs = 15 kips < 54 kips

Allowable tension = 54 kips

OK

* Slings @ 0°

Actual shear = 15000 lbs = 15 kips < 25.8 kips

Allowable shear = 25.8 kips

OK

* Bolt Bearing:

$$f_p = \frac{15000}{\pi D \times t} = \frac{15000}{\pi \times 1.25 \times 3} = 1.27 \text{ ksi} < 12 \text{ ksi}$$

$$F_p = \frac{1}{3} F_y = \frac{1}{3} \times 36 = 12 \text{ ksi}$$

OK

* Bolt tear out:

$$f_t = \frac{15000 \times 2}{3 \times 3.8} = 2.6 \text{ ksi} < 12 \text{ ksi}$$

OK

$$F_t = \frac{1}{3} F_y = \frac{1}{3} \times 36 = 12 \text{ ksi}$$

K. Hoist Ring at 21 & 22 points. (2x6x5/16 end)

Forged steel 1"-8, capacity = 10,000 lbs

L = 2.45"

E = 4" $131.53

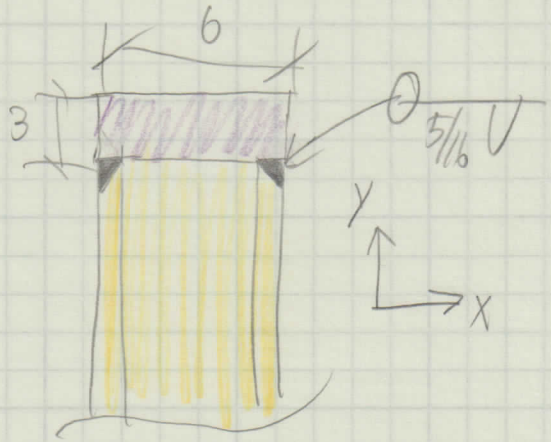
D = 7"

This end will be used to lift the module at four pickup point only

30000/4 = 7500 lbs < 10,000 lbs capacity

OK

L. Design connections at (21) & (22) points.



$F_x = 4471$ (S)

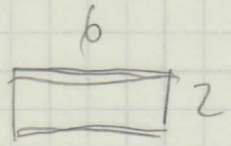
$F_y = 5963$ (T)

$F_z = 4693$ (S)

$M_x = 13679$ (B)

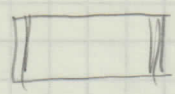
$M_y = 4024$ (Twisting)

$M_z = 15976$ (B)



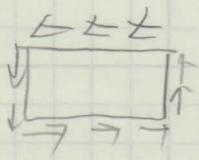
$$f_{vx} = \frac{4471}{6+6} = 372 \text{ lb/in}$$

$$f_t = \frac{5963}{(6+2)\times 2} = 372 \text{ lb/in}$$

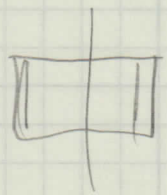


$$F_{vz} = \frac{4693}{2+2} = 1173 \text{ lb/in}$$

$$f_t' = \frac{13679}{S_w} = \frac{13679}{14.6} = 937 \text{ lb/in}$$



$$f_{vs} = \frac{4024}{J_w} = \frac{4024}{84} = 48 \text{ lb/in}$$



$$f_t'' = \frac{15976}{S_w} = \frac{15976}{85} = 188 \text{ lb/in}$$

$$f_r = \sqrt{(f_t + f_t' + f_t'')^2 + (f_{vx} + f_{vs})^2 + (f_{vz} + f_{vs})^2}$$

$$= \sqrt{(372 + 937 + 188)^2 + (372 + 48)^2 + (1173 + 48)^2}$$

$$= 1.9 \text{ kip/in} < \text{allowable } 3.36 \text{ kip/in}$$ OK

M. check hoist Ring mounting plate (small end)

Tension: $f_t = \frac{30000/3}{2 \times 6} = 833 \text{ psi}$

Bending: $f_{bx} = \frac{13679}{S_x}$ $\quad S_x = \frac{ab^2}{6} = \frac{6 \times 3^2}{6} = 9 \text{ in}^3$

$$= \frac{13679}{9}$$

$$= 1.5 \text{ ksi}$$

$f_{bz} = \frac{15976}{S_z}$ $\quad S_z = \frac{ab^2}{6} = \frac{2 \times 3^2}{6} = 3 \text{ in}^3$

$$= \frac{15976}{3}$$

$$= 5.2 \text{ ksi}$$

Combined stress:

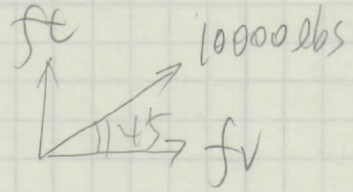
$$\frac{f_t}{F_t} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} \leq 1.0$$

$$\frac{0.833}{12} + \frac{1.5}{12} + \frac{5.2}{12} = 0.6 < 1.0$$

OK

N. Check Hoist Ring Bolt 4"

* Slings @ 45°



Actual $f_v = 0.707 \times 10000 = 7070$ lbs $= 7.07$ kips < 16.5 kips

Allowable shear $= 16.5$ kips

OK

Actual tension:

$$f_t = \frac{7070}{0.7854} = 9 \text{ ksi} < 39 \text{ ksi}$$

OK

Allowable tension:

$$F_t = \sqrt{44^2 - 4.39 f_v^2}$$

$$= \sqrt{44^2 - 4.39 \times 9^2}$$

$$= 39 \text{ ksi}$$

* Sling @ 90°

Actual tension $= 10000$ lbs $= 10$ kips < 34.6 kips

Allowable tension $= 34.6$ kips

OK

* Sling @ 0°

Actual shear $= 10000$ lb $= 10$ kips < 16.5 kips

Allowable shear $= 16.5$ kips

OK

* Bolt Bearing:

$$f_p = \frac{10000}{3 \times 2D} = \frac{10000}{3 \times 2 \times 1} = 1.667 \text{ ksi} < 12 \text{ ksi}$$

$$F_p = \frac{1}{3} F_y = \frac{1}{3} \times 36 = 12 \text{ ksi}$$

OK

* Bolt tear out:

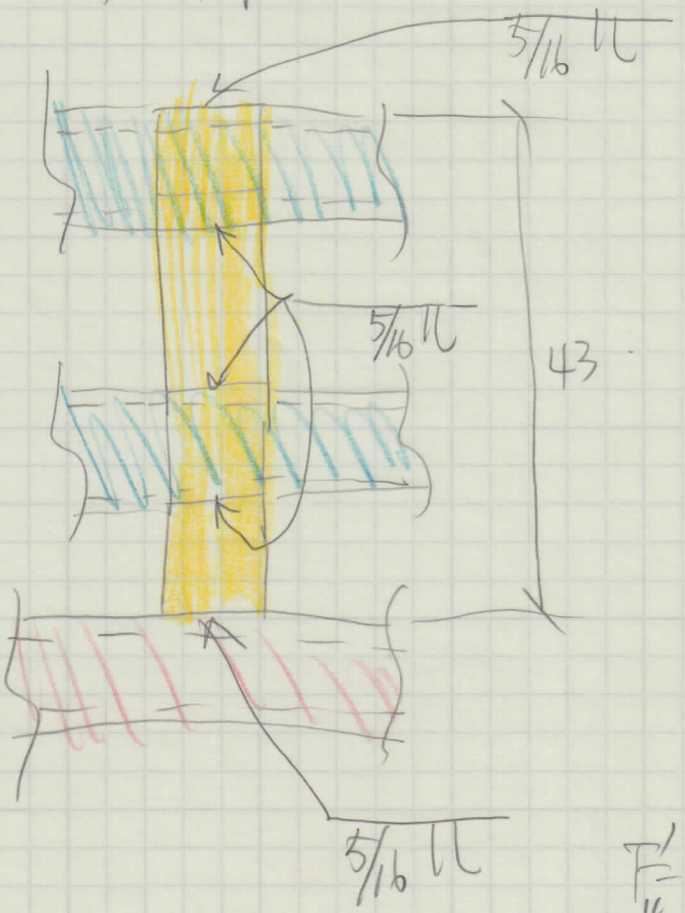
$$f_t = \frac{10000 \times 2}{L \times t} = \frac{2000}{0.914 \times 3} = 7.2 \text{ ksi} < 12 \text{ ksi}$$

OK

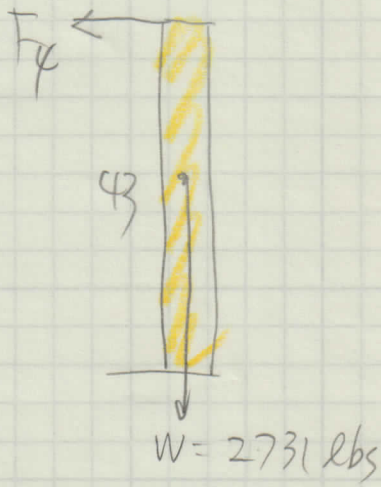
$$F_p = \frac{1}{3} \times F_y = \frac{1}{3} \times 36 = 12 \text{ ksi}$$

O. Fixed side member welds.

* for 4" plate



tip over Force F



$$F_\psi \times 43 = 2731 \times 2$$

$$F_\psi = 127 \text{ lbs}$$

$$F'_\psi = 10\% \, W = 273 \text{ lb (governs)}$$

Welds: $e = 0.707 \times 5/16 = 0.22$ in

$e = 5/8 \times 5/16 = 0.19$ in (governs)

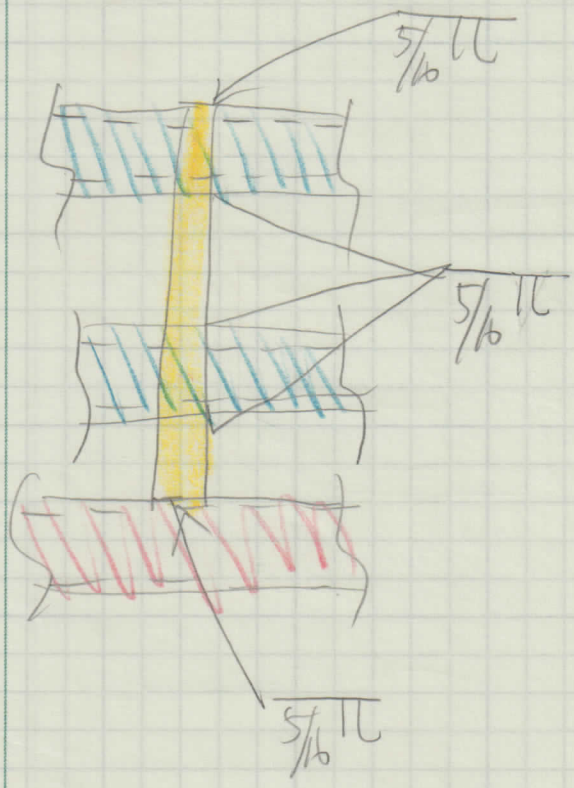
allowable on weld:

$12 \times 0.19 = 2.34$ kips/in

Actual weld: $4 \times 5 \times 2.34 = 46$ kips $\gg 273$ lbs

OK

* for 3/4" plate



tip over Force F

$W = 512$ lbs

$F_{3/4} \times 43 = 512 \times \frac{0.25}{2}$

$F_{3/4} = 4.46$ (lbs)

$F_{3/4}' = 10\% \, W$

$= 512 \times 10\%$

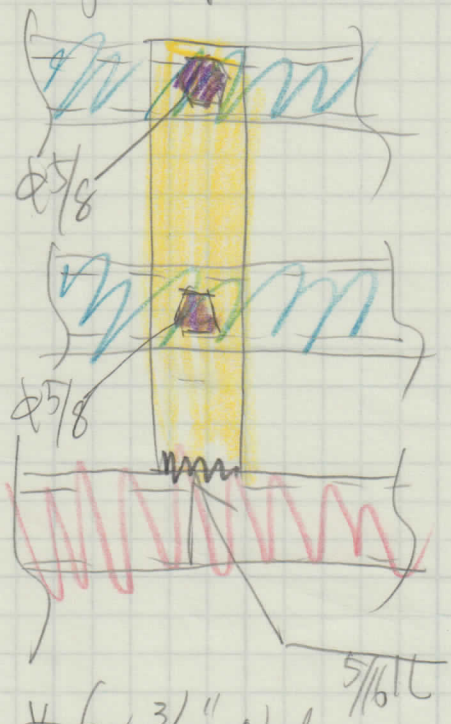
$= 51$ (lbs) (governs)

Actual weld : $3/4 \times 5 \times 2.34 = 8.8$ kips $\gg 512$ lbs

OK

P. Removable side connections.

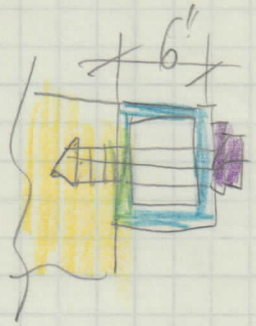
* for 1/4" plate



Bolt load $= W/2 = 273/2 = 1366$ lbs

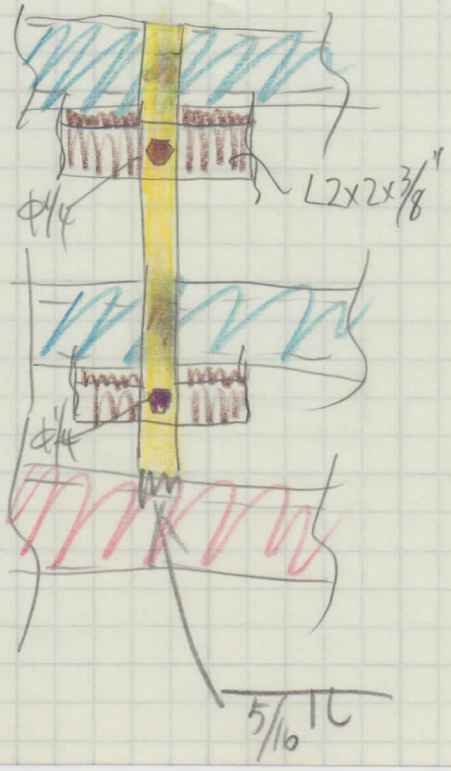
For Φ5/8" Allowable shear $= 3100$ lbs

$SF = \frac{3100}{1366} = 2.27$ OK



$L = 39/64 + 6 + 5/32$
$= 6.76"$
use 7"

* for 3/4" plate



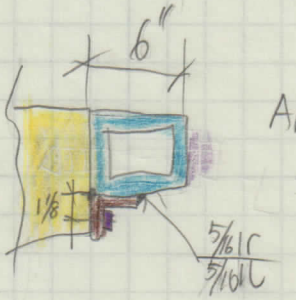
Bolt load $= \frac{W}{2} = \frac{512}{2} = 256$ lbs

For Φ1/4" Allowable shear

$= 10 \times \frac{1}{4}\pi \times (0.25)^2$

$= 490$ lbs < 256 lbs

OK



Angle Bending:

$M = F \times e$

$= 256 \times 1.125 = 288$ lb-in

$S = \frac{ab^2}{6} = \frac{(0.75 + \frac{1.25}{2}) \times 0.375^2}{6} = 0.032 \text{ in}^3$

$f_b = \frac{M}{S} = \frac{288}{0.032} = 8.9$ ksi

< 12 ksi OK

Q Support frame weight Calculation:

— 6x6x 5/16 (W = 23.34 lb/ft)

$(54 \times 3) + (4 \times 54) + (68.5) + (2 \times 43) = 427.5''$

bottom side long column

$W_1 = 23.34 \times \frac{427.5}{12} = 831.5$ lbs

— 6x2x 5/16 (W = 14.83 lb/ft)

$(68.5) + (2 \times 43) = 154.5'' = 12.8'$

long column

$W_2 = 14.83 \times 12.8 = 189.8$ lbs

— total support frame weight

$W_1 + W_2 + 8 \times (2731) + 8 \times (512) + 4$ Hillmans

$= 831.5 + 189.8 + 21848 + 4096 + 24 \times 4$

$= 27061.3$ lbs

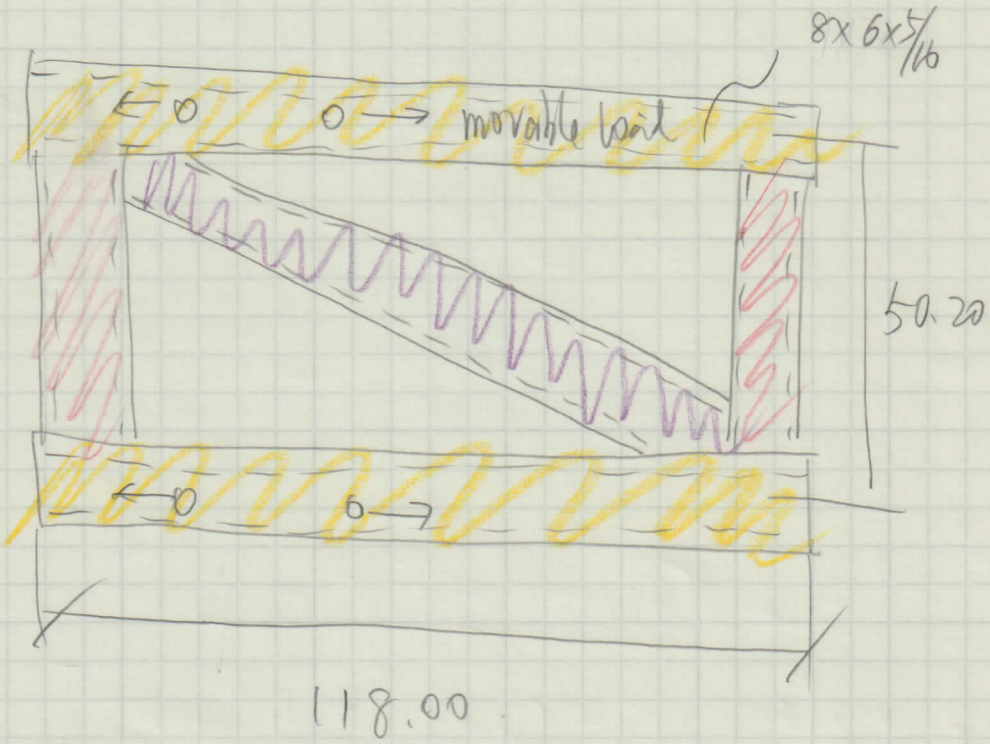
↳ use 30000 lbs for positioning Frame Design

3. Design load for positioning frame

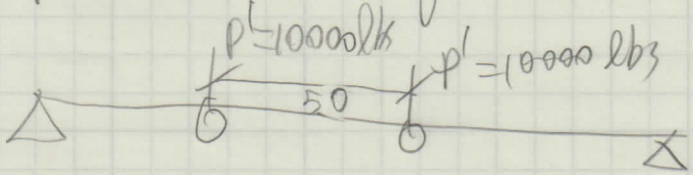
$P = 30000$ lbs

Each Hi'man $= \frac{30000}{3} = 10000$ lbs

4. Design Details for the positioning frame.



A. Simple Beam with moving loads



$L = 118$ in

$$R_{1\,max} = P\left(2 - \frac{a}{l}\right) = 10000 \times \left(2 - \frac{50}{118}\right)$$
$$= 15763 \text{ lbs}$$

when, $a < 0.586l = 0.586 \times 118 = 69$ in. OK

$$M_{max} = \frac{P}{2l}\left(l - \frac{a}{2}\right)^2$$

$$= \frac{10000}{2 \times 118}\left(118 - \frac{50}{2}\right)^2$$

$$= 366483 \text{ lb-in} \quad \text{(governs)}$$

$$M_{max} = \frac{Pl}{4}$$

$$= \frac{10000}{4} \times 118 = 295000 \text{ lb-in}$$

$$f_b = \frac{M}{S} \quad \text{where } S_x = 18.1 \text{ in}^3$$

$$= \frac{366483}{18.1}$$

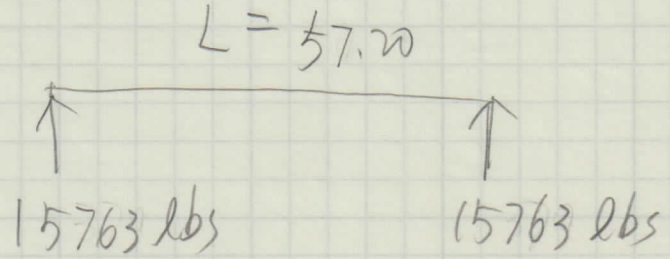
$$= 20.2 \text{ ksi} < F_b \quad \text{OK}$$

$$F_b = 0.6 F_y$$

$$= 0.6 \times 46$$

$$= 27.6 \text{ ksi}$$

B. Cross member



$$R = 15763\,lb = \frac{Wl}{2} = \frac{W \times 57.20}{2}$$

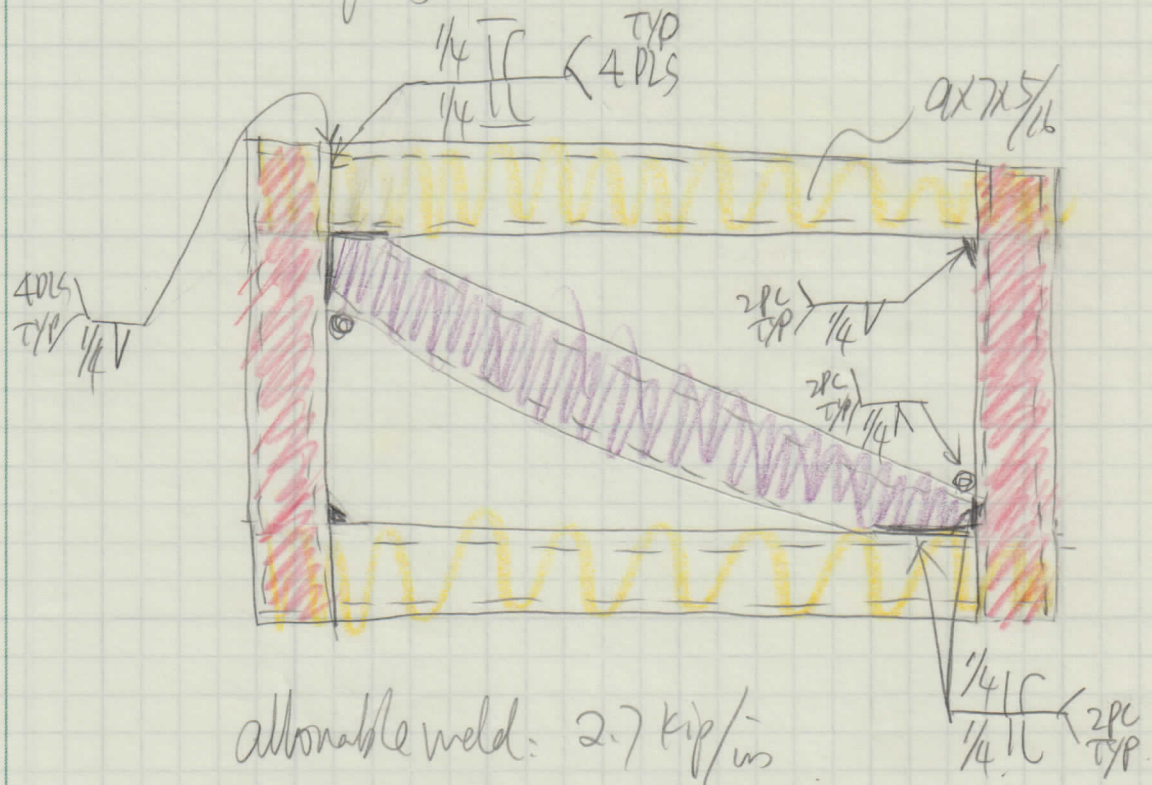
$$W = 551.2\ lb/in$$

$$M_{max} = \frac{Wl^2}{8} = \frac{551 \times 57.2^2}{8} = 225410.9\ lb\text{-}in$$

$$f_b = \frac{M}{S} = \frac{225410.9}{18.1} = 12.4\ ksi < F_b = 22.6\ ksi$$

OK

C. Weld Design for positioning frame



allowable weld: 2.7 kip/in

Actual weld: $7 \times 2.7 = 18.9$ kips >> 15.370 kips

OK

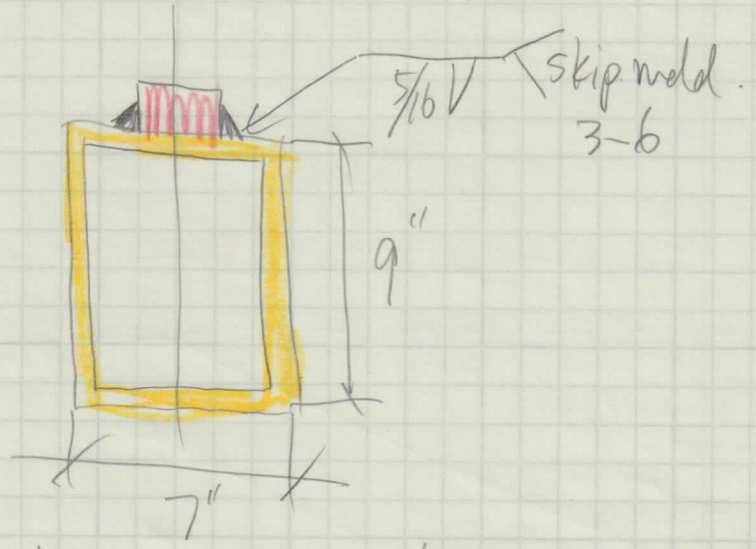
D. Track plate 1"H x 3/8"W x 107"L

Cam Guide inner distance = 3/4"

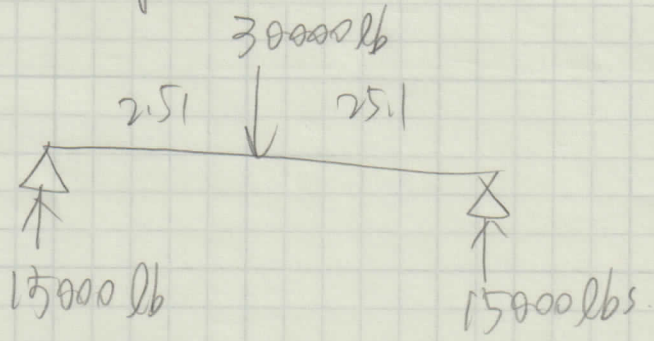
Track plate width = 3/8"

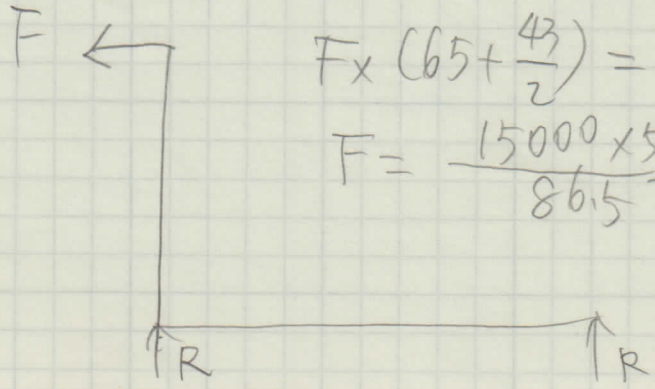
Δ = 1/8" clearance

E. Track plate mold Design.



F. Module Tip over calculation



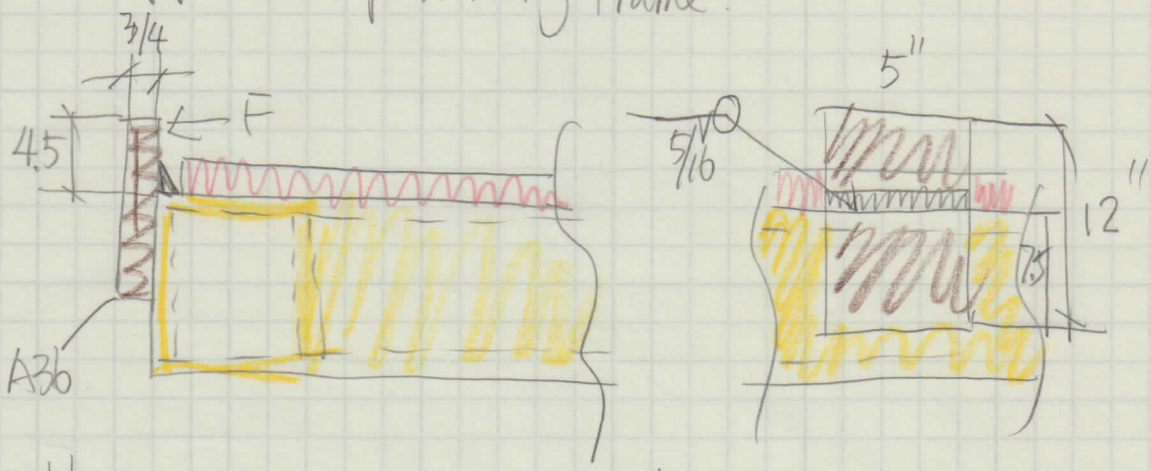


$$F \times \left(65 + \frac{43}{2}\right) = R \times 50.2$$

$$F = \frac{15000 \times 50.2}{86.5} = 8705 \text{ lbs}$$

(not likely)

G. Stop plate on positioning Frame.



H Design load $= 10\% (30000)/2$
$= 1500$ lbs.

$F = \text{impact} = 2 \times H/2 = 2 \times 1500/2 = 1500$ lbs

* Plate Bending: $M = 1500 \times 4.5 = 6750$ lb-in

$$S = \frac{ab^2}{6} = \frac{5 \times 0.75^2}{6} = 0.468 \text{ in}^3$$

$$f_b = \frac{M}{S} = \frac{6750}{0.468} = 14.4 \text{ ksi} < F_b$$

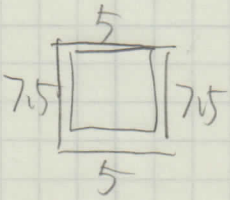
OK

$$F_b = 0.75 F_y = 0.75 \times 36 = 27 \text{ ksi}$$

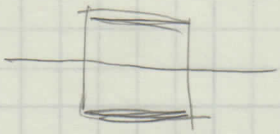
* Plate weld:

Tension $= 1500$ lbs

Bending $= 6750$ lb-in.



$$f_t = \frac{1500}{(5+7.5) \times 2} = 60 \text{ lb/in}$$



$$f_t' = \frac{6750}{S_w} \quad \text{where } S_w = bd + \frac{d^2}{3}$$

$$= \frac{6750}{56.25} \qquad = 5 \times 7.5 + \frac{7.5^2}{3}$$

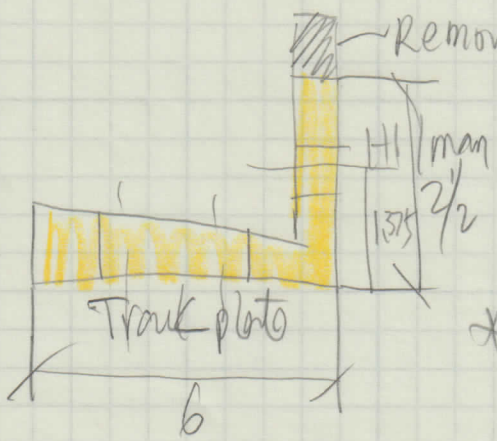
$$= 120 \text{ lb/in} \qquad = 56.25 \text{ in}^2$$

$$f_r = f_t + f_t'$$

$$= 60 + 120 = 180 \text{ lb/in} < 2.6 \text{ kip/in} \quad \underline{\underline{\text{ok}}}$$

Allowable = 2.6 kip/in

H. Hilman Roller Holder Design (Not Used)
(Use C-Clamp instead)



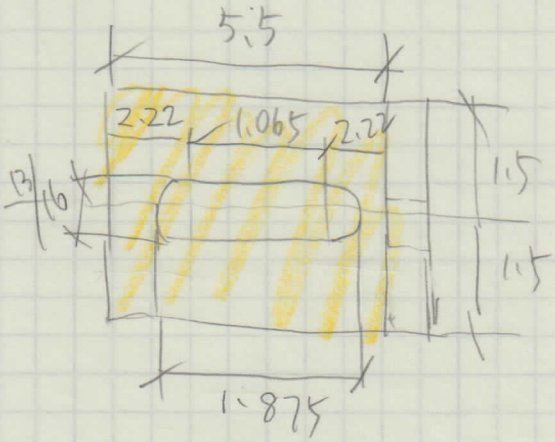
L 6 x 3½ x ½ x 3" L

* Angle Bending:

$$M = 1.375 \times 1500$$

$$= 2062.5 \text{ lb-in}$$

$$S = \frac{ab^2}{6} = \frac{3 \times 0.5^2}{6} = 0.125 \text{ in}^3$$



$$f_b = \frac{M}{S} = \frac{2062.5}{0.125} = 16.5 \text{ ksi}$$

$$< F_b$$

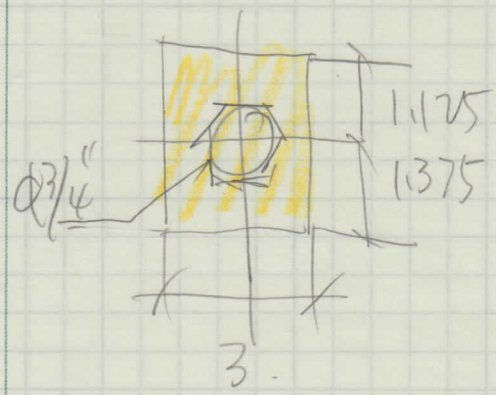
$$F_b = 0.6 \times F_y$$

$$= 0.6 \times 36$$

$$= 21.6 \text{ ksi}$$

$\underline{\underline{\text{OK}}}$

* Bolt connected to Hilman Roller side.



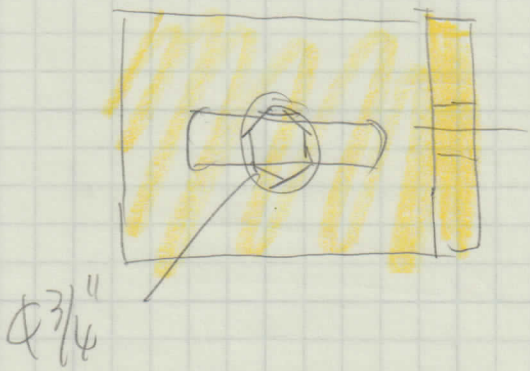
Tension = 1500 lbs < 19.4 ksi

Allowable tension for Φ3/4" = 19.4 ksi ok

Drill & Tap the Hilman Roller plate

Bolt + Nut + Washers

* Bolt connected to the track plate side.



Shear = 1500 lbs < 5.3 kips ok

Allowable shear Φ3/4 = 5.3 kips

Drill & Tap the track plate.

oversize washers will be used

Bolt + Nuts.

* Check edge distance for the long slot

Required parallel to line

$$C_1 = 1\tfrac{1}{2}d - \tfrac{1}{16} = 1\tfrac{1}{2} \times 0.75 - \tfrac{1}{16} = 1.0625$$

$$C + C_1 = 1 + 1.0625 = 2.0625'' < 2.22''$$

OK

Required ⊥ to line

$$C_2 = \tfrac{3}{4}d = \tfrac{4}{3} \times \tfrac{4}{3} = 0.5625''$$

$C + C_2 = 1 + 0.5625 = 1.56'' \approx 1.5''$ OK

5. Concrete Shielding Block

A. Bearing on concrete (Allowable)

ACI 318, 10.17.1. Bearing strength $\phi(0.85 f_c' A_1)$

Where $\phi = 0.65$ [9.3.2.2.(b)]

$f_c' = 5000$ psi

$A_1 = 8.25 \times 3\frac{1}{8}$

$= 25.78 \text{ in}^2$

$\phi(0.85 f_c A_1)$

$= 0.65 \times 0.85 \times 5000 \times 25.78$

$= 71220$ lbs

B. Actual Bearing stress:

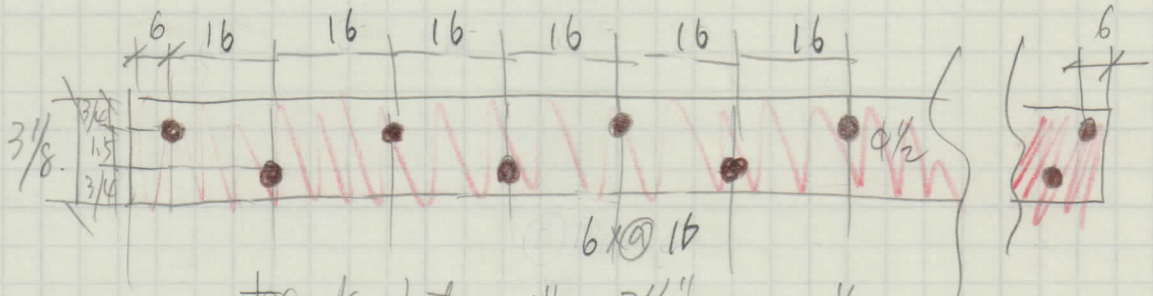
$U = 1.7 \times (D + L)$

$= 1.7 \times (30{,}000)$

$= 51000$ lbs $<$ 71220 lbs

OK

C. Track plate connection of the Block



track plate 1"H x 3 1/8"W x 108"L x (2) ea

socket head cap screw Φ1/2

Head height = 1/2"

Head width = 3/4"

Locker washer for Φ1/2 min. TK = 0.125"

* Level the track plate by shimming prior to 3/4" Grout

West side Industrial Supply

South Elgin, IL

1-847-931-7200

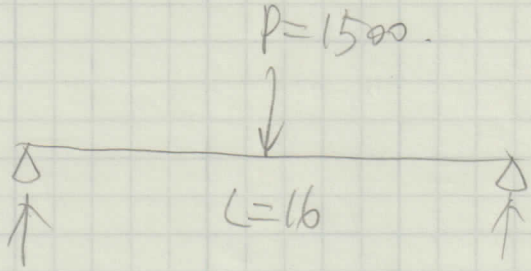
UNISORB V100 EPOXY GROUT (STANDARV100)

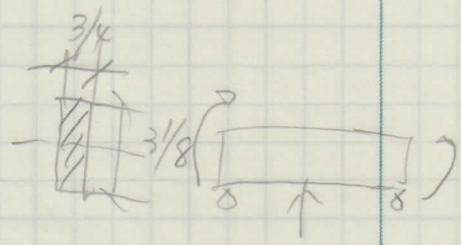
55# kit = $194.75

covers 0.522 ft³

11# kit = $50

* check track plate Bending due to shear load





When the roller reaches the Anchor, Anchor takes the full load. When the roller in the middle span, the Anchor only takes the half of the load. the other half of load goes to the track plate.

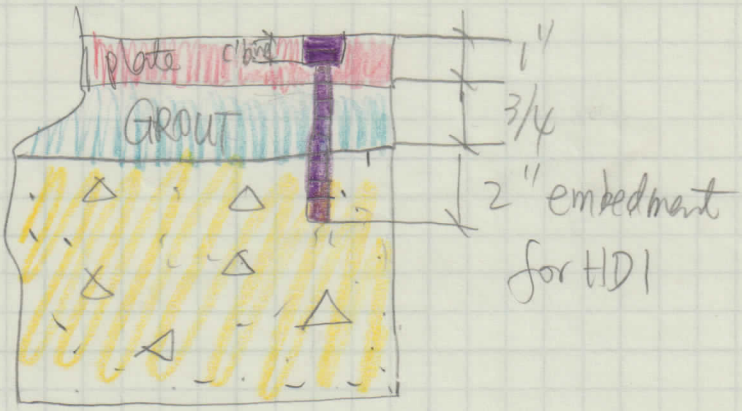
$$M_{max} = \frac{1}{4}PL = \frac{1}{4} \times 1500 \times 16 = 6000 \text{ lb-in}$$

$$S = \frac{ab^2}{6} = \frac{0.75 \times 3.125^2}{6} = 1.22 \text{ in}^3$$

$$f_b = \frac{M_{max}}{S} = \frac{6000}{1.22} = 4.9 \text{ ksi} < F_b = 0.6 \times 36$$

$$= 21.6 \text{ ksi}$$

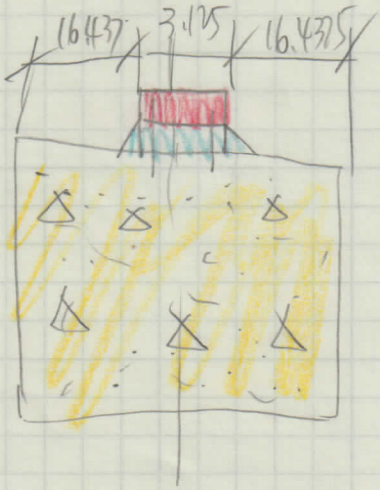
OK



$C'bore = \frac{1}{2} + 1/8 + 1/8$
$= 0.75''$

$L_{screw\ length} = (1'' - 0.75'') + 3/4 + 2$
$= 3''$

* HD1 Φ1/2



S = 7" @ 100% (Shear)
C = 6" @ 100% (Shear)

full strength for shear = 2125 lbs

Actual Shear = 1500 lbs < 2125 lbs

OK

Torque HD1 Φ1/2 to 22 Lbs-Fts

D. Stop plate on Concrete Block.

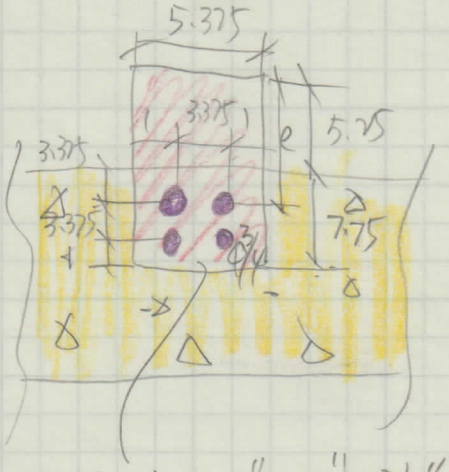
* Hilti HVA 3/4": Min. embendment 6 5/8"
Rod HAS ASTM A36 STD

S = 3 3/8 @ 70% T/S
C = 3 3/8 @ 70% T

Allowable tension concrete = 8615 lbs

Allowable HAS = 8455 lb

Actual = 8615 x 0.7 x 0.7 = 4221.35 lbs (governs)



$e = 10.3125''$

$M = F \times e$

$= 1500 \times 10.3125$

$= 15468.75$ lb-in

$T/c = \frac{M}{e} = \frac{15468.75}{3.375} = 4584$ lbs

A36 $5.375'' \times 13'' \times 3/4''$

Top row tension per Hilti:

$4584/2 = 2292$ lbs < 4221.35 lbs

ok

Bottom row Compression per Hilti:

$2292 + \frac{1500}{4} = 2667$ lbs

* plate Bending:

$M = 1500 \times 5.25 = 7875$ lb-in

$S = \frac{ah^2}{6} = \frac{5.375 \times 0.75^2}{6} = 0.5039$ in³

$f_b = \frac{M}{S} = \frac{7875}{0.5039} = 15.6$ ksi < 21.6 ksi

ok

* Concrete Bearing Stress:

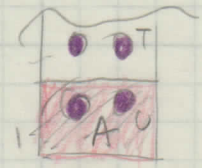
ACI 318, 10.17.1, Bearing Strength $\phi\,(0.85 f_c' A_1)$

where $\phi = 0.65$ [9.3.2.2(b)]

$f_c' = 5000$ psi

$A_1 = 5.375 \times \left(1 + \frac{3.375}{2}\right)$

$= 14.4 \text{ in}^2$



$\phi\,(0.85 f_c' \times A)$

$= 0.65 \times 0.85 \times 5000 \times 14.4$

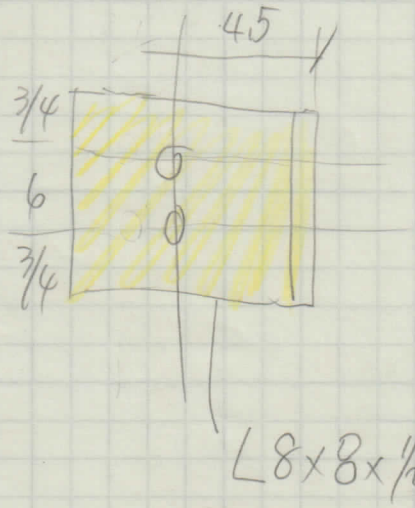
$= 39780$ psi

$$\text{Actual } f_p = \frac{1.7 \times (2) \times (2667)}{144} = 629 \text{ psi}$$

< 39780 psi

OK

E. E BLOCK HOLDING BRACKET.



for shear on 1/2" HDI x (2).

$H/DI = \frac{3000}{4} = 750$ lbs

$S = 6$ @ $f_A = 0.83\%$

$C = 4.5$ @ $f_{RV} = 0.7K\%$

Allowable $= 2125$ lbs

Actuall $= 0.83 \times 0.7K \times 2125$

$= 1305$ lbs > 750 lbs

OK

CHECK HILMAN ROLLER SUPPORT FRAME:

DESIGN LOADS FROM INGRID FANG'S CALCULATIONS:

4" PLATE = 2731#
3/4" PLATE = 512#

MODULE WT = 8 X 2731 + 8 X 512 = 25944#
CASSETTES - 16 X 32 = 512#
26456#

LIFTING FIXTURE:

2X6 X 5/16 X 59 X 1 = 73#
6X6 X 5/16 X 59 X 1 = 115#
6X6 X 5/16 X 54 X 5 = 525#
2X6 X 5/16 X 43 X 1 = 53#
6X6 X 5/16 X 43 X 1 = 84#
2X6 X 5/16 X 43 X 1 = 53#
6X6 X 5/16 X 43 X 1 = 84#
987#

439736 X 1 = 140#
439732 X 1 = 140#
4 HILMAN ROLLERS @ 22# EA = 88#
MISC. PLATES, BOLTS = 500#
1855# → 1855
+ 26456
28,311#

POSITIONING FRAME:

FROM DWG 439626 1305#

TOTAL = 29616#

USE 30,000# TOTAL LOAD

LOAD PER HILMAN ROLLER = 30,000/4
= 7500#
=

BASED ON ORIGINAL 92" C TO C SPACING OF POSITIONING FRAME:

MAXIMUM REACTION FOR 2 EQUAL LOADS ON SUPPORT BEAM =

$R = P(2 - a/\ell)$ WHERE a = DISTANCE BETWEEN LOADS

ℓ = BEAM SPAN
P = LOAD

$= 7500(2 - 50/92)$

$= 10\,924^{\#}$

HILMAN CAPACITY = 5 METRIC TONS

$= 5(2205^{\#})$

$= 11025^{\#} > 10924$

OK

BASED ON NEW 118" - 6" SPACING

$R = 7500(2 - 50/112)$

$= 11652^{\#} > 11025^{\#}$ NG

USE 3 RAILS:

$R = 7500(2 - 50/56)$

$= 8304^{\#} < 11025^{\#}$ OK

LOCATION OF EYE BOLTS FOR REMOVING 6x6x5/16 TUBES FROM TCMT

TUBE WT = 23.34/12 (52.5) = 101#

ANGLE WT = 4.7/12 (15.75) = 6#

END PLATE = 490/1728 (6 x 13.25 x .75) = 17.0# EA

TOTAL LOAD:

101 + 6 + 17 + 17 = 141#

LOCATION OF EYE BOLT FROM LEFT END:

MEMBER	WEIGHT	DISTANCE	WEIGHT x DISTANCE
PLATE	17.0	0.375	6.375
ANGLE	6.0	8.625	51.75
TUBE	101.0	27.0	2727.0
PLATE	17.0	53.625	911.625
	141		3696.75

$\bar{y}$ = 3696.75/141

= 26.218" FROM LEFT END

CHECK 2-SPAN BEAM:

$\ell = (106.5/2) - 3 = 50.25''$

$R_{MAX} = \frac{Pa}{2\ell^3}(2\ell^2 + b(\ell + a))$

$a = \ell - 50/2 = 50.25 - 25 = 25.25''$

$= \frac{7500(25.25)}{2(50.25)^3}(2(50.25)^2 + 25(50.25 + 25.25))$

$b = \ell - a = 50.25 - 25.25 = 25.0''$

$= 5177^{\#}$ FROM EA. SIDE @ CENTER SUPPORT

TOTAL $= 5177^{\#}(2) = 10354^{\#} < 11025^{\#}$ OK

CHECK MOVING LOAD MOMENT ON BEAM:

USE 2 SINGLE SPANS → CONSERVATIVE

a = DISTANCE BETWEEN LOADS $\quad 0.586\ell = 0.586(50.25) = 29.45''$

$a > 0.586\ell \therefore$

$M_{MAX} = P\ell/4 = 7500(50.25)/4 = 94219^{\#}\text{-IN}$

$L_c = (1950 + 1200\, M_1/M_2)\, b/F_y$ (F3-2 P 5-48 AISC)

$= (1950)(6/46) = 246'' > 50.25''$

$\therefore F_b = 0.66 F_y = 0.66(46000) = 30360$ psi

$S_{x\,REQ'D} = 94219/30360 = 3.10\ IN^3 < 18.1\ IN^3$ USE T3 8x6x5/16