AN ANALYSIS OF THE ROOF FRAMING OF BERKEY HALL

Thesis for the Degree of B. S.

MICHIGAN STATE COLLEGE

George W. Fox

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An Analysis of the Roof Framing

of

Berkey Hall

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AGRICULTURE AND APPLIED SCIENCE

by

George W. Fox

Candidate for the Degree of

Bachelor of Science

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PREFACE

Wishing to continue my study of steel construction, I decided to analyze the roof structure of Berkey hall to determine if it were adequate enough to carry the roof loads.

I wish at this time to express my gratitude to the Building and Utilities Department of Michigan State College for the use of their plans and specifications, and to Mr. Shermer for his advice and suggestions on this work.

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INTRODUCTION

The roof of Berkey hall is supported by a system of steel rafters and trusses. The roof is composed of re-inforced gypsum slabs covered with roofing felt and slate. The gypsum slabs weigh 20 pounds per square foot, the roofing felt, according to specifications, weighs 30 pounds per square, and the slate weighs 15 pounds per square foot. The total weight per square foot of the roofing materials is 35.3 pounds. There are four types of purlins used in the roof framing. They are a 7"9.8 channel, a 8"11.5 channel, a 9"13.4 channel, each with a $1\frac{1}{2}x1\frac{1}{2}x\frac{1}{4}$ angle welded to the back, and a 8B10 section. As the purlins are fastened to the rafters with standard connections, there was no need to determine the type of connections to be used. The average weight per square foot of roof surface of these purlins is 2.9 pounds, so I used an average roof load of 38 pounds per square foot.

Using the value of 38 pounds per square of roof as the dead weight Ifigured the reactions for each length of purlin. For purlins framing into valley rafters, I figured the loaded roof area as a trapezoid with the purlin length as the average of the sum of the bases.

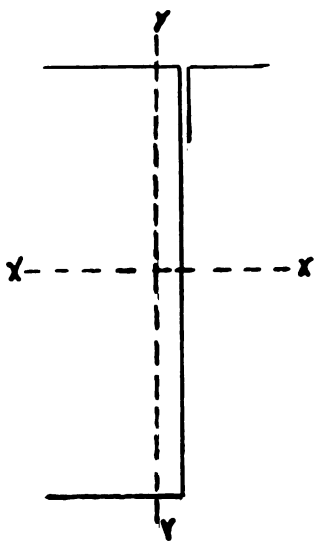
In analyzing the members of the roof structure, I used the Lansing Building and Safety code, which was used in designing the structure. It specifies a maximum fiber stress in bending of 18,000 pounds per square inch, a maximum shearing for steel beam webs of 12,000 pounds per square inch, amaximum tensile stress of 18,000 pounds per square inch, a shearing stressof 13,500 pounds per square inch in rivets and bolts, a bearing stress on rivets in single shear of 24,000 pounds per square inch, and a bearing stress on rivets in double shear of 30,000 pounds per square inch. It provides that when wind load is used in figuring stressed that all stresses shall be increased 331 /3%. The Lansing Building and Safety code also specifies that the live load on a roof is to be figured as 30 pounds per square foot, and that the horizontal wind load is to be 20 pounds per square foot. The live load and the wind load on the roof were figured in the same manner as the dead load.

SCHEDULE OF PURLIN REACTIONS

Length	Area	Dead Load	LIve Load	Wind Load
2	8.56	330	257	171
2.8	14.7	560	441	294
3.16	16.6	630	498	332
3.5	18.4	700	554	368
3.8	20	760	600	400
5.5	28.8	1090	864	576
7	25.7	956	756	504
7	36.8	1400	1100	736
8	41.5	1580	1245	830
9	47	1790	1420	844
9.25	48.5	1840	1460	970
10.5	55	2090	1650	1100
11	39.6	1500	1190	792
11	57.7	2190	1730	1154
11.5	60.4	2300	1810	1208
11.58	60.7	2300	1821	1214
12	63	2400	1880	1260
12.5	65.5	2490	1965	1310
13	68.3	2600	2050	1366
15	78.8	3000	2360	1575
16	84	3200	2520	1680
17	89.3	3400	2680	1786
19.5	102	3880	3060	2040
22	115	4360	3450	2300

As there are no section moduli for channels with $1\frac{1}{2}x1\frac{1}{2}x\frac{1}{4}$ angle welded on the back given the A.I.S.C. handbook, I had to figure the section moduli of these purlins before I could determine the stresses.

They are as follows:



Area of channel = 2.85 in.^{2} Area of angle = .69 In.^{2}

I_{x-x} " " = 21.1 in.^{4} I " " = .14 in.^{2}

I_{y-y} " " = .98 in.^{4} x or y = .47 in.

x " " = .55 in.

Y " " 3.5 in.

y_1 = 2.85x3.5+.69x.47/3.54=2.82 in.

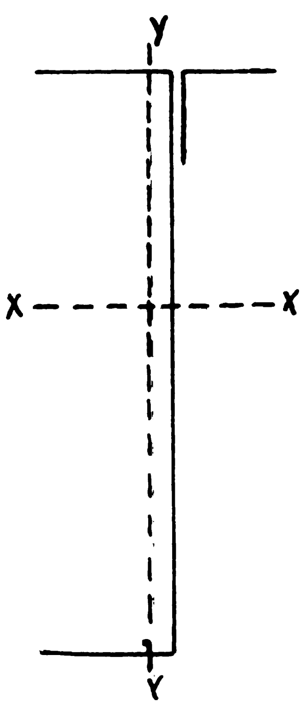
x_1 = 2.85x.55+.69x.47 ÷ 3.54 = .55 in.

I_{x-x} = 21.1+2.85x(.68)^2+ .69x(2.35)^2 = 26.3 in.^{4}

I_{y-y} = .98+ 2.85x.04+.14 .69x1 = 1.92 in.^{4}

Z_{x-x} = 26.3/2.82 = 9.4in.^{3}

Z_{y-y} = 1.92/2.05 = .94 in.^{3}



Area of 8" channel = 3.36 in.^{2}

I_{x-x} " " = 32.3 in.^{4}

I_{y-y} " " = 1.3 in.^{4}

x " " = .58 in.

y " " = 4 in.

y_1 = 3.36x4 + 69x.47/4.05 = 3.4 in.

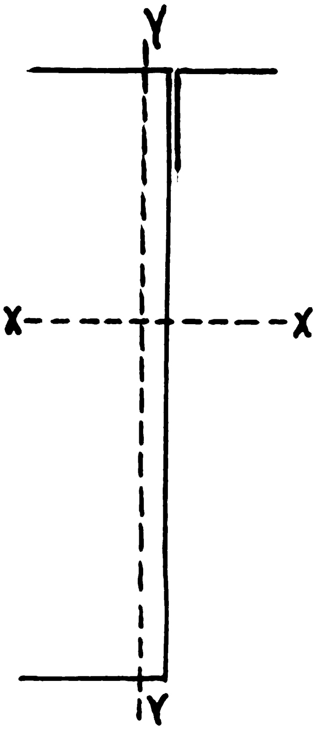
x_1 = 3.36x.58+.69x.47/4.05 = .56 in.

I_{x-x} = 32.3 +.14 3.36x.36+.69x(2.93)^2 = 39.57 in.^{2}

I_{y-y} = 1.3+3.36x.04+.69x(1.03)^2 = 2.3 in.^{4}

Zx-x = 39.57/3.4 = 11.6 in.^{3}

Z_{y-y} = 2.3/ 2.06 = 1.12 in.^{3}



Area of 9" channel $= 3.89$ in.^{2}

I_{x-x} " " $= 47.3$ in.^{4}

I_{y-y} " " $= 1.8$ in.^{4}

x " " $= .61$ in.

y " " $= 4.5$ in.

$y_1 = 3.89x4.5 + .69x.47/4.56 = 3.9$in.

$x_1 = 3.89x.61 + 69x.47/4.58 = .59$ in.

$I_{x-x} = 47.3 + 14 + 3.89x.36 + .69(3.43)^2 = 56.9$ in.^{4}

$I_{y-y} = 1.8 + 3.88x.04 + .69(1.06)^2 = 2.87$ in.^{4}

$Z_{x-x} = 56.9/3.9 = 14.5$ in.^{3}

$Z_{y-y} = 2.87/2.09 = 1.37$ in.^{3}

As the wind load has no component parallel to the roof, it is onl y necessary to determinethe stresses in the purlins due to the dead load plus live load. This stress is determined by finding the moments due to the loading in a normal direction and a parallel d irection to the roof. In the plane of the roof the sag rods act as supports for the purlins and the purlins act as beams between sag rods. The stress in the p rlin is thus the sum of the stress due to the moment due to the loa d component normal to the roof a nd the greates t mome nt due to the load component parallel to the roof.

Stresses in the 8" channel

D.L.+L.L. $68x5.25x.707 = 252$ #/ft.

$M_1 = 252x6(16-6)x12/2 = 91{,}000$ in.lb.

$M_2 = 252x16x12/8 = 6050$ in.lb.

$S = 91{,}000/11.6 + 6050/1.12 = 12{,}900$#/in.^{2}

Stresses in the 8B10 section.

D.L.+L.L.= 68x5.25x.707x12=3040#

M_1=3040x12x12/8=54,900 in. lb.

M_2=1010x4x12/8=6060 in.lb.

S=54,900/7.79+6060/1.01=13,000#/in.^{2}

Stresses in the 9" channel.

D.L.+L.L.=252#/ft.

M_1=252x10x(22-10)x12/2=181,000 in.lb.

M_2=252x16x12/8=6100 in.lb.

S=181,000/14.5+6100/1.37=16,950#/in.^{2}

Stresses in the 7" channel.

D.L.+L.L.=252#/ft.

M_1=252x5.4x(11.5-5.4)x12/2=45,600 in.lb.

M_2= 252x16x12/8=6100in.lb.

25

S=45,600/9.4+6100/.94=11,350#/in.^{2}

The sections are adequate.

The sag rods must take the component of the dead load and live load which is parallel to the roof. They must be des igned to carry the largest possible load to come on any one of them, consequently I used the sag rod at the peakin the wides t bay to determine the size of sag rod necessary in this build ing.

D.L.+L.L.=40x4x68x.707=7700lbs.

P=18,000A=7700

A=.428 in.^{2}

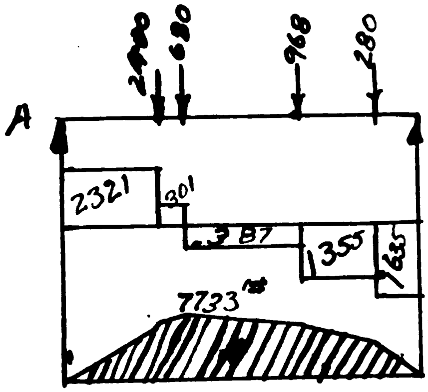
$A=3.14d^2/4$

$d=(4A/3.14)^{\frac{1}{2}}$

$d=(1.712/3.14)^{\frac{1}{2}}$

d=.74 in.

To get an adequate cross-section at the root of the threads, a one inch diameter rod should be used.



The d ormer not being an integral part of the roof I did not attempt to determine the stress in its' members, but did determine the effect of the dormer weight on the roof members. For this purpose I assumed the weight of the dormer roof to be 38 pounds per square foot and the masonry walls to weigh 20 pound s per square foot. The wind load acts normalto the roof surface, but due to the fact that the design load is given as a horizontal force it has to be transposed into a force normal to the roof. Grinter in Design of Modern Steel Structures page281 suggests the use of the formala P_n PΘ/45 where P is the load in pounds per s quare foot on a verticalsurface P_n is the load normal to the roof surfacein pounds per square foot, and Θ is the angle of inclination in degrees. As the angle of inclination of this roof is 45degrees, P_n P 20 pound s per square foot of roof surface. The dormer walls are made of masonry covered with slate so the dead load is 40 pounds per square foot.

Dead load on dormer 6.5x6.5/2x40 6.5x3.52x3.8 1716

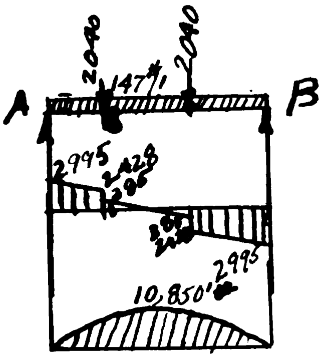
L.L. 6.5x3.52x30 690 D.L. L.L. 2400 D.L. L.L. 288 at valley ends.

Wind load on the dormer front is 340 pounds, as this load is horizontal it has a component normal to the beam, this componentis 240#

W.L. on roof is 455# As the dead load of the dormer acting through its ' center of mass is roughly equivalent to the actual load on the beam, I used this load in determining the bending moment in the beam, which is an 8"11.5 channel.

D .L.+L.L.

11Rb=2400x3+680x3.75+968x7.35+288x9.75=19,650
Rb=1790 Ra= 4336-1790=2545
M=2546x3+146x.65=7733 ft.lb.
~~3*8770x12~~/Z=7733x12/18,000=5.15 In.^{3}



Dormer beam continued:

D.L.+$\frac{1}{2}$L.L.+W.L.

11 Rb = 2686x3+820x3.65+1110x7.35+300x9.75=22.282

Rb=2020 Ra=4900-2020=2880

M=2880x3+200x.65=8770 ft.lb.

Z=8770x12/24,000=4.37 in.^{3}

The be ams that support the wall and roof of the dormer fra mes into purlins that act as beams carrying the end reactionsof the dormer beams.

D.L.+L.L. upper beam

Roof load=54.6x68a.707/13=202#/ft.

M_1=(1250x3.8+202x169/8)12=108,000 in.lb.

M_2= 202x16x12/8=4848 in.lb.

S=108,000/11.6+4848/1.12=13,600#/in.^{2}

R

Ra=Rb=1250+202x13/2=2563 13 ft.span. Ra=Rb=1250+202x11/2=2363 11 ft.

D.L.+$\frac{1}{2}$L.L.+W.L. upper beam

D.L.+$\frac{1}{2}$L.L.=156#/ft. D.L.+$\frac{1}{2}$L.L.+W.L.=240#/ft.

M_1=1425x3.8+240x169/8=10475 ft.lb.

M_2=156x16 x12/8=3740 in.lb S=10,745x12/11.6+3750/1.12=14,090#/in.^{2}

Ra=Rb=1425+240x13/2=2995# 13ft. span.Ra=Rb=1425+240x11/2=2750# 11ft.s pan.

D.L+ L.L. lower beam.

Roof load=33.35x68x.707/13=123#/ft.

M1=(1800x3.8 123x.69/8) x12=113,000 in.lb. M_2=(123x27.6/8)x12=5100 in.lb.

$$S=\frac{113,000}{11.6}+\frac{5100}{1.12}=13,350 \#/in.^2$$

Ra=R b=1800+123x13/2=2600# 13 ft. span. Ra=Rb=1800+123x11/2=2475 11ft.

D.L.+½L.L.+W.L. lower beam

D.L.+½L.L.=96#/ft. D.L.+½L.L.+W.L.=147#/ft.

M_1=(2040x3.8+147x169/8)x12=130,500 in.lb.

M_2=96x16x12/8=2[illegible]00 in.lb. S=130,500/11.6=2[illegible]00/1.12=13,300#/in.^{2}

Ra=Rb=2040+147x13/2=2995# 13ft. span

Ra=Rb=2040+147x11/2 11 ft. span

In this roof structure there are several rafters which act as beams spanning the distance from the eave to the ridge, supporting the roofing material. In the north end on the north side there are two rafters supporting a section of the foof and half of one dormer load, and two supporting one dormer load, these are designated by me as N-1 and N-2 respectively. In the northend there are a ls o two rafters supporting the south side of the roof, these are designated by me as N-3. The valley rafters where the main north and south roof frames into the east and west roof on the north end are des igna te d by me as N-4.

Where the main east and west roof frames into the main north and south roof there are two rafters at the north side each carrying a section of roof and one half of one dormer load, which a re designated by me as M-1. On the south side of this intersection there arefour rafters, two of which carry a section of roof and one half of a dormer load, and two of which carry a section of roof plus the rea ction from the load carried by the valley rafter on the south side, these are designated by me asM-2 and M-3 respectively. The valley rafter on the north is designated as ~~M-4~~ M-4, and the valley rafter on the south is designated as M-5. This structure is supported nea r the peak by two trusses, truss T-1 supports the valley on the north and truss T-2 supports the valley on the north.

The valley rafter at the east end where the ma in eas t and west roof frames into the north and south roof on the east end I have des ignatedas E-1. On the east side of this roof are four rafters, two of which carry a section of roof and one half a dormer load and two of which carry one dormer load . Ihave designated these rafters E-2 and E-3 respectively.

The rafters N-1, N-2, M-1, M-2, and M-3 are all the same length and cross-section. N-1 is the most heavily loaded of these rafters and has the greatest moment, so I analyzed N-1 and figured the reactions for N-2, M-1, M-2, and M-3 and noted these reactions with the analysis of N-4, as Ineed these reactions to analyze the ridges and trusses supporting the several rafte rs .

The rafters E-2 and E-3 are of the same length and cross-section but E-2 is more heavily loaded, s o I analyzed E-2 and figured the reaction of E-3 and noted it with analysis of E-2, as I need this reaction to analyze the ridge.

Rafter N-3

D.L. 18.75Rb=1310x5.25+1560x10.5+1300x15.75=57,700

Rb=2760# Vert.=3900#

M=2760 x3+960x5.25=13,350 ft.lb.

L.L. 18.75Rb=1030x5.25+1225x10.5+1170x15.75=36,000

Rb=1920# Vert.=2710#

M=1920x3+790x5.25=9910 ft.lb.

W.L. 18.75Rb=977x5.25+1146x10.5+1378x15.75= 38,340

Rb=2040# Vert.=1440#

M=2040x3+702x5.25=9820 ft.lb.

D.L.+L.L. M=23,260 ft.lb. Z =23,260x12/18,000=15.5 $in.^3$

D.L.+½L.L.+W.L . M=28,120 ft.lb. Z=28,120x12/24,000=14 $in.^3$

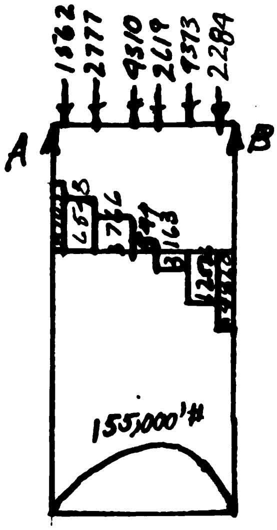
Rafter N-1

D.L.+L.L. 31.25 Rb=4493x2+2653x7.25+2653x12.5+4398x17.75=320,560

Rb=10,250# Vert.=14,500#

M=10,250x3+6710x5.25/3170x5.25=82,700 ft.lb.

Z=82,700x12/18,000=55$in.^3$



D.L.+½L.L.+W.L. 31.25Rb=4200x2+3118x7.25+4015x17.75

+4231x23+4231x28.25=359,700

Rb=11500# Ve rt.=16,250#

M=11,500x3+7269x5.25+3038x5.25=88,500 ft.lb.

Z=88,500x12/24,000=44 in.^{3}

Rafter N-4

D.L. Rb=762x2.8+1350x9.1+5,000x15.4+1280x21.7+5,000x28

+1110x34.3=295,250

Rb=7880# Vert.=9850#

L.L. 38Rb=600x2.8+1070 x9.1+3680x15.4+1000x21.7+3980x28+880x34.3 =231,430

Rb=6100# Vert.=7650#

W.L. 38Rb=500x2.8+892x9.1+2470x15.4+839x21.7+2378x2.8+754x34.3=147,400

Rb=3890# Vert.=3100#

D.L.+L.L. M=13,980x3.6+12,000x6.3+3020x6.3=149,250 ft.lb.

Z=149,250x12/18,000=99in.^{3}

D.L.+½L.L.+W.L. M=14820x3.6+12540x6.3+3170x6.3+650x6.3=155,000 ft.lb.

Z=155,000x12/24,000=77.7in.^{3}

D.L.+L.L. S_s =13,980/2.98=4,700#/in.^{2}

D.L.+½L.L.+W.L. S_s =14,820/2.98=4,960#/in.^{2}

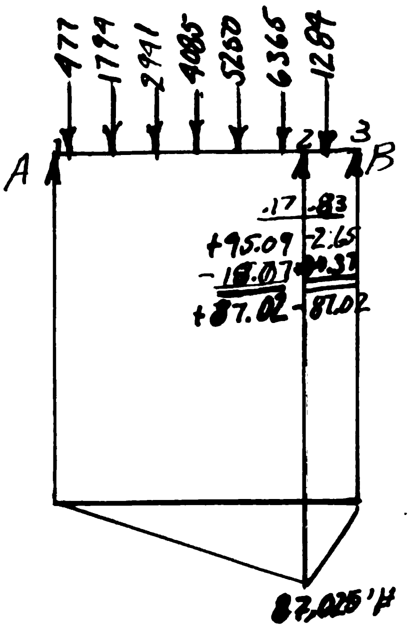
Reactions of N-2,M-1,M-3,M-2 at point where the frame into ridge or truss.

N-2: D.L.+L.L.=12,300# D.L.+½L.L.+W.L.=15,000#

M-1: D.L.+L.L.=13,000# D.L.+½L. L.+W.L.=15,500

M-3: ~~D.L.+L.L.=4750#~~ D.L.=4750 L.L.=3750 W.L.=2160

M-2 D.L.+L.L.=9900 D.L.+½L.L.+W.L.=13,300#



D.L. 38.9 Rb=264x2.8+873x9.1+1430x15.4+1990x21.7

+ 2560x28+3100x34.3=251,444

. Rb=6450#

M=6450x4.5+3350x6.3+790x6.3=55,000ft.lb.

L.L. 38.9Rb=205x2.8+690x9.1+1135x15.4+1510x21.7

+2020x28+2450x34.3=197,655

Rb=5200#

M=5200x4.5+2650x6.3+630x6.3=43,000 ft.lb.

W.L. 38.9Rb=171x2.8+576x9.1+944x15.4+1310x21.7+1680x28+2040x34.3=165,620

Rb=4250#

M=4250x4.5+2210x6.3+530x6.3=36,380 ft.lb.

D.L+L.L.: M=82,440 Z=82,440x12/18,000=56 in^3.

D.L.+½L.L.+W. L. M=87,025 ft.lb. Z=87,025x12/24,000=43.5 in^3.

The reactions at 2 being necessary to determine the ~~re~~ load on trus s T-2 I found them as follows:

D.L+ L.L. ΣM_1=38.9 R_2-469x2.8 - 1563x9.1 - 2665x15.4 - 3500x21.7-

4580x28 - 5550x34.3 - 82,450=0

38.9 R_2=531,544

R_2=13,650#

ΣM_3=8.1R_2-1116x4-82,450=0

8.1 R_2=86,914

R_2=10,750 Total=24,400# Vert.=30,560#

D.L.+½L.L.+W.L. ΣM_1=38.9R_2-737x2.8- 1794x9.1- 2941x15.4- 4055x21.7-
5250x28-6365x34.3-94,445=0
38.9 R_2=610,064
R_2=15,650#
ΣM_3=8.1R_2-1264x4-94,445=0
8.1R_2=99,550
R_2=12,300# Total=27,950# Vert.=30,900#

Rafter M-5 is situated the same as rafter M-4,?it is of a different section so I had to analyze it. I us ed the same method of analysis as in M-4, and found the reactions on trussT-1 and rafterM-3 in the same manner.

D.L. 23.4Rb=715x6.3+1270x12.6+1830x18.9=55,000

Rb=2350#

M=2350x4.5+520x6.3=13,880 ft.lb.

L.L. 23.4Rb=48.6x6.3+1000x12.6+1325x18.9=41,400

Rb=1765#

M=1765x4.5+400x6.3=10,470 ft.lb.

W.L. 23.4Rb=472x6.3+840x12.6+1200x18.9=36,300

Rb=1550#

M=1550x4.5+350x6.3=9200 ft.lb.

D.L.+L.L. M=18,650 ft.lb. Z=18,650x12/18,000=12.4in.^{3}

D.L.+$\frac{1}{2}$L.L.+W.L. M=21,635 ft.lb. Z=21,635x12/24,000=10.8 in.^{3}

Reaction of M-5 on M-3

D.L.+L.L.=1750# Vert.=1250#

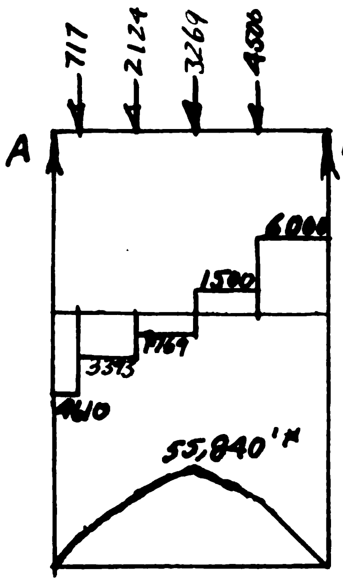
D.L.+$\frac{1}{2}$L.L.+W.L.=2045# Vert.=2260#

Reaction of M-5 on truss T-1

D.L.+L.L.=7760# Vert.=9700#

D.L.+$\frac{1}{2}$L.L.+W.L.=9000# Vert.=10,240#

Rafter E-1



D.L. 28Rb=350x2.8+1035x9.1+1590x15.4+2245x21.7

=83,620

Rb=2990# Vert.=3750#

L.L. 28Rb=276x2.8+815x9.1+1260x15.4+1695x21.7

=64,374

Rb=2300# Vert.=2880#

W.L. 28Rb=229x2.8+682x9.1+1049x15.4+1415x21.7

=53,690

Rb=1950# Vert.=1560#

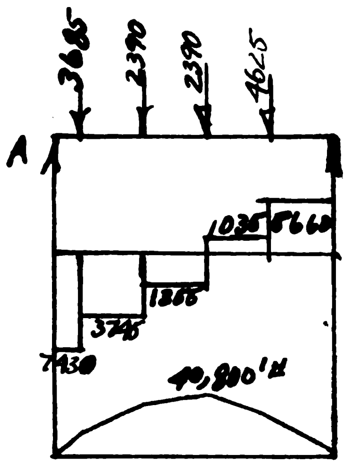
D.L.+L.L. M=5290x6.3+1745x6.3=44,350 ft.lb.

Z=44,350x12/18,000=29.5 in.^{3}

D.L.+½L.L.+W.L. M65 M=6580x6.3+2265x6.3=55,840 ft.lb.

Z=55,840x12/24,000=27.9 in.^{3}

Rafter E-2



D.L.+L.L. 24

24Rb=3180x2+1930x7.25+1930x12.5+4083x17.75=116,860

Rb=4875# Vert.=6900#

M=4875x6.25+792x5.25 34,450 ft.lb.

Z=34,450x12/18,000=23 in.^{3}

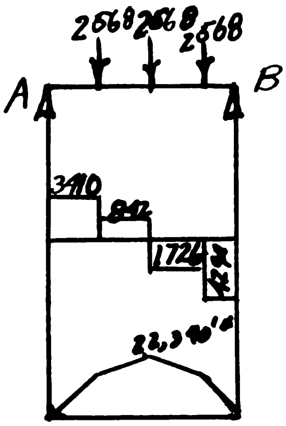
D.L.+½L.L.+W.L. 24Rb=3685x2+2390x7.25+2390x12.5+4625x17.75=136,170

Rb=5660# Vert.=8,000#

M=5660x6.25+1035x5.25=40,850 ft.lb.

Z=40,850x12/24,000=20.4 in.^{3}

The vertical reaction of rafter E-3 at the ridge is 6700# D.L.+L.L. and 7900# D.L.+½L.L.+W.L.



The main east and west roof ends in a gable on the eas t end. The roof at this end is supported by an 8WF17 section. This section is used because there is an air duct at this end, and the section gives clearanceto the duct and adequate support to the roof.

D.L.+L.L. 19.2Ra=2150x3.3+2150x8.55+2150x13.8=55,200

Ra=2860#

M=2860x5.25+710x5.25=18,630 ft.lb.

Z=18,630x12/18,000=12.4 $in.^3$

D.L.+ $\frac{1}{2}$L.L.+W.L. 19.2 Ra=2568x3.3+2568x8.55+2568x13.8=65,890

Ra=2830#

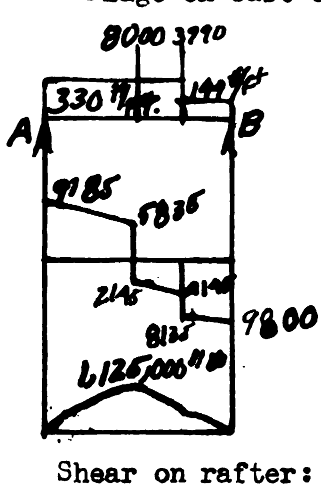
M=2830x5.25+855x5.25=22,340 ft.lb.

Z=22.340x12/24,000=11.1 $in.^3$

Shear inrafter: D.L.+ L.L. S =3590/1.69=2130#/$in.^2$

D.L.+$\frac{1}{2}$L.L.+W.L. S=4294/169=2540#/$in.^2$

Ridge on east end.



D.L.+L.L.=357#/ft. D .L.+$\frac{1}{2}$L.L.+W.L.=330#/ft.
Mas onry load=144#/ft.

D.L.+L.L. 25Rb=6900x12+4000x18.1+357x18.1x 9

+144x6.9x21.55=234,000

Rb=9400 Ra=18555-9400=8950

Z=(9400+5100x144/2)/18,000=58 $in.^3$

D.L.+$\frac{1}{2}$L.L.+W.L. 25Rb=8,000x12+3990x18.1+330x18.1x9

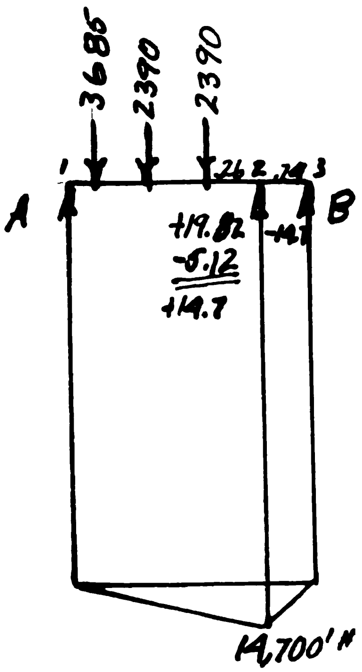
+995x2155= 245,150

Rb=9800

Z=(9800+5830x144/2)/24,000=47 $in.^3$

Shear on rafter:
D.L.+L.L.
S_s = 9400/2.7=3500#/$in.^2$
D.L.+$\frac{1}{2}$L .L.+W.L.
S_s =9800/2.7=3640#/$in.^2$

Section E-E is shown in plate 10 . I analyzed it as follows:



D.L.+L.L.

$17.75 R_b = 3180 \times 2 + 1930 \times 7.25 + 1930 \times 12.5 = 44,300$

$R_b = 2500\#$

$M = 2500 \times 5.25 + 570 \times 5.25 = 16,000$ ft.lb.

D.L.+$\frac{1}{2}$L.L.+W.L.

$17.75\ R_b = 3685 \times 2 + 2390 \times 7.25 + 2390 \times 12.5 = 54,670$

$R_b = 3080\#$

$M = 3080 \times 5.25 + 690 \times 5.25 = 19,820$ ft.lb.

D.L.+L.L.

$\Sigma M_3 = 6.25 R_2 - 11,800 = 0 \quad \Sigma M_2 = 6.25 R_3 - 11800 = 0$

$R_2 = 1900\# \qquad R_3 = -1900\#$

$\Sigma M_1 = 17.75 R_2 - 3180 \times 2 - 1930 \times 7.25 - 1930 \times 12.5 - 11,800 = 0$

$17.75 R_2 = 56,100$

$R_2 = 3160 \qquad$ Total $= 9145\#$ H $= 13,000\#$

Value of 2 angl es $3 \times 2\frac{1}{2} \times \frac{1}{4} = 39,500\#$ in compression.

D.L.+$\frac{1}{2}$L.L.+W.L. $\Sigma M_2 = 6.25 R_3 - 14,700 = 0 \qquad \Sigma M_3 = 6.25 R_2 - 14,700 = 0$

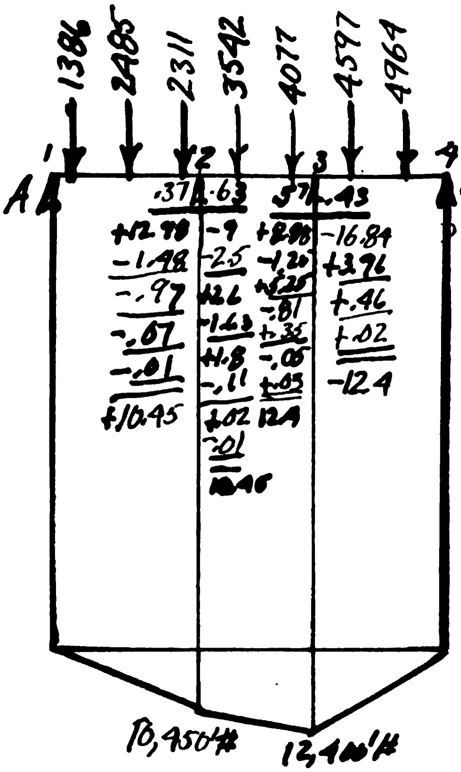
$R_3 = -2350\# \qquad R_2 = 2350\#$

$\Sigma M_3 = 17.75 R_2 - 3685 \times 2 - 2390 \times 7.25 - 2390 \times 12.5 - 14,700 = 0$

$R_2 = 3900\# \qquad$ Total $= 10875\#$ H $= 15,400\#$

Value of 2 angles $4 \times 4 \times \frac{1}{4} = 52,750\#$ in compression.

The load on the angle in this section is 7,225#. The value of one bolt in shear is 5950# and in bearing, single shear, it is 6750#. As there are nine bolts and the load may be assumed to be applied to each equally it is obvious that no bolt is overstressed. The size a nd number of bolts was determined by the bond betw een the bolts and the masonry.



The main roof truss I analyzed as a 3 span continuous beam supported at the panel points. I found the reactions at the panel points in the same manner as in rafter M-5 and used these reactions as loads on the roof.

D.L.+L.L. 15 ft. span

M_2=1160x26x17/450=1140 ft.lb.

M_2=2080x7.25x7.75x22.25/450=5880 ft.lb.

M_2=2525x12.5x25x27.5/450=4830 ft.lb.

D.L.+½L.L.+W.L. 15 ft. span

M_2=1386x26x17/450=1365 ft.lb.

M_2=2485x7.25x7.75x22.25/450=6850 ft.lb.

M_2=3012x12.5x2.5x27.5/450=5765 ft.lb.

D.L.+L.L. center span

M_2=2970x2.75x81/139=4700 ft.lb.

M_2=3410x8x14.4/139=2825 ft.lb.
Total = 7525 ft.lb.

M_3=2970x7.55x9/139=1450 ft.lb.

M_3=3410x64x3.8/139=5960 ft.lb.
Total =7410

D.L.+½L.L.+W.L. centerspan

M_2=3547x2.75x81/139=5625 ft.lb.

M_2=4077x8x14.4/139=3380 ft.lb.
Total=9000 ft.lb.

M_3=3547x7.55x9/139=1730 ft.lb.

M_3=4077x64x3.8/139=7140 ft.lb.
Total = 8880 ft.lb.

D.L.+L.L. end span

M_2=3850x15.2x22.3/278=4700 ft.lb.

M_3=4155x35.2x17.05/278=8960 ft.lb.
Total=13,660 ft.lb.

D.L.+½L.L.+W.L. end span

M_3=4597x15.2x22.3/278=6115 ft.lb.

M_3=4965x35.2x17.05/278=10,720 ft.lb.
Total=16,835 ft.lb.

D.L.+L.L. Z=10,110x12/18,000=6.75 in^3

D.L.+½L.L.+W.L Z=12,090x12/24,000=6.05 in^3

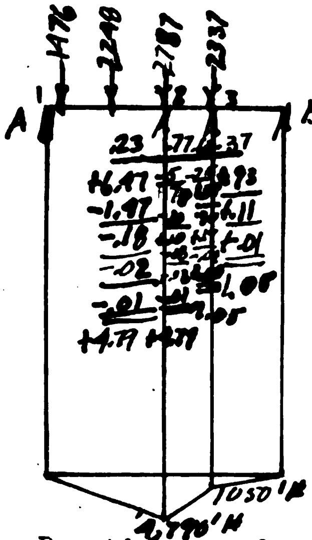
Vertical reactions used as loads on the truss.

D.L.
R_1=1480#
R_2=5500#
R_3=7150#
R_4=1485#

L.L.
R_1=1165#
R_2=4550#
R_3=5800#
R_4=1250#

W.L. normal to the roof
R_1=1035#
R_2=3065#
R_3=3740#
R_4=[illegible]

Frame "D" is made similarly to the main roof truss and I analyzed it in the same way.



D.L.+L.L.

M_2=1235x21x14.5/312=1200ft.lb.

M_2=1880x2.25x5.25x19.75/312=4525 ft.lb.

D.L.$\frac{1}{2}$L.L. W.L.

M_2=1750x21x14.5/312=1700 ft.lb.

M_2= 1979x2.25x5.25x19.75=4767 ft. lb.

Z=4160x12/18,000=2.77in.^{3} D.L.+L.L.

Z=4685x 12/24,000=2.34 in.^{3} D. L.+$\frac{1}{2}$L .L.+W.L.

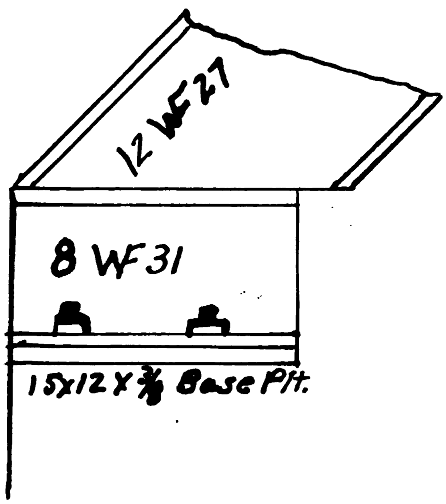
Reactions used as loads

D.L.	L.L.	W.L.
R_1=1190#	R_1=920#	R_1= 635#
R_2=2000#	R_2= 1700#	R_2= 1008#
R_3= -900#	R_3= -740#	R_3= -406#

Shearing stresses in the beam

D.L.+L.L. S=11,500/2.12=5400#/in.^{2} D.L. +$\frac{1}{2}$L.L.+W.L. S=12884/2.12=6000#/in.^{2}

Detail "A-A" is used as the base for the rafters and the main truss.



The loads on this section that I used in analyzzingit are the greatest that I figured as coming on the section.

D.L.+L.L.=24,403#

D.L.+$\frac{1}{2}$L.L.+W.L.=25,476#

According to the Lansing Building and Safety code, if the web of a girder or beamis 60 times its' thickness or greater, the allowa ble Shearing stress is to be determinedby the formula; $\dfrac{18,000}{1+\dfrac{h^2}{7200t^2}}$ in which h is the distance between the flangesand t is the thickness of the web In this section h is 7.134" , t is .288" 60t is17.28" . The allowable shearing ~~for~~ stress in beams where the height does not exceed 60t is 12,000#/in^2.

The allowable load on this section is 12,000x7.134x.288 or 24,600 pounds. The section is not overstressed.

The rafter is welded to the 8WF31 section and the section is welded to the base plate. The necessary length of weld is24,400x.707/5000x.25 or 13.8 inches. The welds are longer than this, so they a re not ofer-stressed.

Check base plate:

Bearing value of base plate=24,400/180=136#/In^2 D.L.+L.L.

M=136x2x1=272 in.lb./in.

$t=(6M/bf)^{\frac{1}{2}} = (6x272/18,000)^{\frac{1}{2}} = .301"$

Bearing value of base plate= 25,476/180=142#/in^2. D.L+ $\frac{1}{2}$L. L.+W.L.

M=142x2x1=284 in.lb./in.

$t=(6M/bf)^{\frac{1}{2}}=(6x284/24,000)^{\frac{1}{2}}=.308"$ A3/8" ba se plate is necessary for this plate.

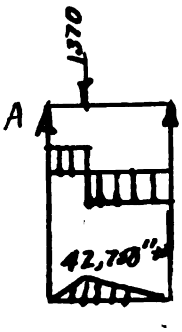
Check bolts: D.L.+L.L. on each bolt=24,400/4=6100#

Shear=6100/.44=13,850#/in^2. Be aring=6100/.28=21,800#/in^2.

D.L.+$\frac{1}{2}$L.L.+W.L. on each bolt=6400#

Shear=6400/.44=14,600#/in^2. Bearing 6400/.28=23,000#/in^2.

The bolts are not overstressed.



Atthe intersection of the two main roofs there are four ridges carrying the roof between the nearest truss and the intersection of the valley rafters. These ridges are the same length and cross-section and ca rry the same load.

D.L.

5.8 Rb=750x1.9

Rb=250# Ra=500#

L.L.

5.8 Rb=645x1.9

Rb=215# Ra=430#

W.L.

5.8 Rb=300x1.9

Rb=100# Ra=200#

D.L.+L.L.

M=43,400 in.lb.

Z=43,400/18,000=2.4 $in.^3$

D.L.+$\frac{1}{2}$L.L.+W.L.

M=42,750 in.lb.

Z=42,750/24,000=1.79 $in.^3$

Shear in beams:

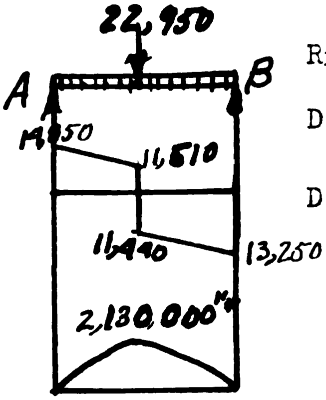
D.L.+L.L.

S=930/.655=1420#/$in.^2$

D.L.+$\frac{1}{2}$L.L.+W.L .

S=915/.655=1400#/$In.^2$

A 6Jr 4.4 section should be used for this ridge.



Ridge at the north end

D.L.+ L.L.=204#/ft/

D.L.+ $\frac{1}{2}$L.L.+W.L.=180#/FT.

D.L.+L.L. Ra=21,100x15.25/28.25+204x28.25/2=14,200#

Rb=26800-14,200 =12,600#

M=(14,200+11,550/2)x13x12=2,010000 in.lb.

Z=2,010,000/18,000=112 $in.^3$

D.L.+$\frac{1}{2}$L.L.+W.L.

Ra=22,950x15.25/28.25+130x28.25/2 =14,850#

Rb=28,050-14,850=13,250#

M=(14,850 +11,510/2) x13x12=2,130,000 in.lb.

Z=2,130,000/24,000=89 $in.^3$

Shear

D.L.+L.L.

S=14,200/7=2028#/$in.^2$

D.L.+$\frac{1}{2}$L.L.+W.L.

S= 14,850/7=2121#/$in.^2$

The center ridge on the north end is the same as the center ridge on the east end. It is more heavily loaded with vertical loa ds but the sag rod# loads are the same. I analyzedthis ridge the same as I did the purlins, using dead load plus live load and combining the stresses in the horizontaland vertical direction.

D.L.= 114#/ft.   L.L.= 90#/ft.

M=114$(11.58)^2$x12/8   M=90$(11.58)^2$x12/8

M=23,000in.lb.   M=18,600 in.lb.

D.L.+ L.L. S = 41,000/34.1=1200#/$in.^2$

Sag rod loads D.L.+L.L.

P=2x.707x30x20+2x.707x38x20+202x3.89+123x3.89/2=1595#

M=1595x3.97x12=73,000 in.lb.

S=73,000/5.1=14,300#/$in.^2$ Total stress=15,500#/$in.^2$

Loads on north end column: D.L.+ L.L.=46,170#
D. L.+$\frac{1}{2}$L.L.+W.L.=47,658

Column va lue=136,500#

Loads on east end column: D.L.+ L.L.+W.L.= 9700#
D.L.+ $\frac{1}{2}$L.L.+W.L.=11,700# column value=104,500#

The main roof truss is shown in Plate 1. The analysis of its' members is based on the loads tabulated there.

Member 1-2, a 12 WF27 section, which acts as a column supporting the truss, is the most highly stressed. As it is the longest member of this cross-section in this truss, if it is not overstressed the other, similar, members will not be overstressed. The allowable load on this section is determined by the formula $P = \frac{18,000A}{1 + \frac{L^2}{18,000r^2}}$

in which P is the allowable load in pounds, L is the unsupported length in inches, A is the cross-sectional area of the section in square inches, and r is the least radius of gyration of the member. In this member $P = 18,000 x 7.97/1+(180)^2/18,000 x (1.44)^2 =$ 76,700# . The member is not overstressed.

Member U_3L_4 is made up of 2 angles $3\frac{1}{2}x2\frac{1}{2}x\frac{1}{4}$. Its ' value is 25,000# compression. It is fastenedto the other members by a gusset plate with $\frac{3}{4}$ inch bolts. The value of one bolt in shear is 13,500x.44= 5950# . The number of bolts needed is 9000/5950= 1.5. As the minimum number of bolts used is three, the bolts will not shear. The value of one bolt in bearing in single shear is $24,000x\frac{3}{4}x3/8=6750$#, therefore the joint will not fail. As this member is not in tension, the net area of the gusset plate need not be figured.

Member U_4L_4 is made up of 2 angles$2\frac{1}{2}x2\frac{1}{2}x\frac{1}{4}$and is in tension its' value in tension is 43,000# . It has loads of 13,000 (D.L.+L.L.) and 15,000# (D.L.+L.L.+W.L.). It is not overstressed. As neither of these loads is more than three times the value of one bolt, the bolts will hold the loads. The net width required is 13,000x8/18,000x3 = 1.92". As there has to be an edge distance of $1\frac{1}{4}$" on each side of the bolts, the net width has to be 3-3/8 inches.

The lower chord is in tension, it is a 6B15.5 sectionand its' value is 83,000# in tension, it is not overstressed.

Frame "D" is similar to the main truss (see Plate2) except that its' ## upper chord is al..F21 section whose value is 64,000#, its ' load is 23,000#C so it is not overstressed. The lower chord is a 6B15.5 section witha value of 83,000# in tension.

MemberU_3L_4 is made up of 2 angles $2\frac{1}{2}x2\frac{1}{2}x\frac{1}{4}$ it has compressive o loads of 4000# (D.L.+L.L.) and 4078# (D.L.+$\frac{1}{2}$L.L.+W.L.). As both of thes e loads are less than the value of one bolt, the three bolts will hold the member.

Member U_4L_4 is made up of 2 angles$2\frac{1}{2}x2\frac{1}{2}x\frac{1}{4}$, it has tensile loads of 5800# (D.L.+L.L.)and 5900# (D.L.+$\frac{1}{2}$L.L.+W.L.). It has a value of 42,900# tension, therefore it is not overstressedand the three bolt minimumwillhold the loads. The loads are so small tha t the minimum width of 3-3/8 inches for the gusset plate controls in selecting a gusset plate.

Frames "B" and "B1" are similar. The members are made up of 2 angles $2\frac{1}{2}x2\frac{1}{2}x\frac{1}{4}$. The value of the longest member in bothe frames in compression is 14,200#, the value of a member in tension is 33,800#, as none of the members have loads of this magnitude they will not fail. The three bolt minimum and the minimum width ##### of gusset plate will take theloads that come upon the structure.

Frame "C" is made up of members which are 2 angles $2\frac{1}{2}x2\frac{1}{2}x\frac{1}{4}$. The longest membe r is 7.75 feet long, its' value is 11,600# in compression,,it is not overstressedwith a 1 oad of 9000#(D.L.+L.L.) The value of the tension members is 33,800#, they are not overs tres-se sed. The three bolt minimum and the minimum width of 3-3/8 inches for the gusset plate take care of the loads.

Trusses T-1 and T-2 are the same except that members U_2L_3 a nd U_3L_3 are heavier in truss T-2. All members in T-2 are more heavily loaded than in T-1 so I will use the loads on the members of T-2 in analyzing the truss. The gusset plates used throughout this truss are $\frac{3}{8}$" thick and the members are bolted into it with $\frac{3}{4}$" bolts . The value of one bolt in bearing in single shear is $24,000x\frac{3}{4}x\frac{1}{2}= 9000\#$.

Member U_2L_2 is in compression, its' value is 49,500#, it has loads of 48,953# (D.L.+L.L.) and 50,106# (D.L.+$\frac{1}{2}$L.L.+W.L.).
No. of bolts neede d to resist shear: 48,953/5950 =8.25
No of bolts needed for bearing: 48,953/9000= 5.45 Nine bolts should be used in this joint.

Member U_3L_3; Value: 42,500# Loads: 33,290# (D.L.+L.L.), 44,250# (D.L.+$\frac{1}{2}$L.L.+W.L.). No. of bolts to resist shear: 33,290/5950= 5.6
No. of bolts for bearing: 33,290/9000= 3.7 Six bolts should be used in this joint.

Member U_2U_3, value149,000# is not overstressed with 147,500# (D.L.+$\frac{1}{2}$L.L.+W.L.). As it is the longest unsupported section of the upper chord and also the heaviest loaded, the other members of the truss with cross-section are not overstressed either. It is a continuous member so there are no joints to analyze. Member L_2L_3 which is of the same cross-section is in tension. Its' value in tension is 207,000#. As it has the same loading as U_2U_3 it is not overstressed, and as it is continuous there are no joints to analyze.

Member U_1L_2; value: 103,000# tension Load: 80,000#(D.L.+L.L.) 82,500# (D.L.+$\frac{1}{2}$L.L.+W.L.). No. of bolts neededto resist s hear= 80,000/5950=13.4. No. of bolts needed in bearing=80,000/9000=8.9 14 bolts should be used. Net width= 80,000=18,000x$\frac{1}{2}$xb, b=8.9"
Gross width should be 9.775".

Member U_2L_3; value 96,000 tension; loads:91,000# (D.L.+L.L.) 93,000# (D.L.+½L.L.+W.L.). No.ofbolts to resist shear:91,000/5950=15.3 No. of bolts need ed in bearing: 91,000/9000= 10.1 sSixteen bolts should be used. Net width= 91,000x2/18,000=10.1" Gross width should be 11".

In trussT-1, members U_2L_3 and U_3L_3 were made lighter than the same members in truss T-2 due to their lighter loads.

Member U_3L_3; value: 32,800# compression; loads: 20,695#(D.L.+L.L.) and 20,900# (D.L.+½L.L.+W.L.).

No. of bolts required to resist shear: 20,695/5950=3.5

No. of bolts needed in bearing: 20,695/9000= 2.3 Four bolts should be used.

Member U_2L_3; value:61,000# tension; loads: 46,000#(D.L.+L.L.) and 60,000# (D.L.+L.L.+W.L.).

NO. of bolts required to resist shear: 60,000/5950=10.1

No. of bolts needed in bearing: 60,000/9000=6.7 Eleven bolts should be used. Net width:60,000x2/1 8,000=6.7" Gross width of the gusset plate should be 7.5"

Truss T-3 (Plate 7) carries the ends of the purlins on the main north and south roof.

Member U_2U_3; value: 24,700# Compression; loads 8600#(D.L.+LL.) and 9300#(D.L.+½L.L.+W.L.). As neither of these loads are greater than three times the value of one bolt in shear or bearing, the threebolt minimum will hold the member.

Member U_1L_1; value 17,500# compression; loads: 1400#(D.L.+L.L.) and 1700# (D.L.+½L.L.+W.L.). The three bolt minimum will hold this member as well as member U_2L_2 , which has a load of 2400#(D.L. +L.L.) and 2900# (D.L.+½L.L.+W.L.).

Member L_0L_1; value: 38,200# tension; loads: 8600#(D.L.+L.L.)

and 10,400# (D.L.+$\frac{1}{2}$L.L.+W.L.). The three bolt minimum will hold this member in rivet shear and be aring. Net width: 8600x8/18,000x3= 1.275". Due to the necessity of having a1$\frac{1}{4}$ edge distance at each side of the bolts the gusset plate has to be at least 3-3/8 inches wide.

Member U_3L_1; value: 61,000# tension, it is loaded the same as member L_0L_1. The three bolt minimum will hold the member, and the gusset plate has to be at least 3-3/8 inches wide.

The wind bracing diagonals take only tension, the value of the members is 19,100#, as there are no diagonals loaded this high, the diagonals are adequate. The struts in the wind bracing have a value of 18,730# compression, as no struts are loaded this high, the struts are adequate. The three bolt minimum in the gusset plate is sufficient to hold the members.

CONCLUSION

The roof structure of this building is adequate for supporting the load that is upon it. It will be noticed that most of the members are understressed. This is due to the fact that the framing was designed to carry a metal lath and plaster ceiling. As I was analyzing the roof structure with reference only to the roof loads, most of the members are not carrying as much load as they are capable of carrying.

According to the Lansing Building and Safety code, this building is classified as class D, and the metal lath and plaster ceiling on the trusses and purlins is not necessary for fireproofing in this type of building. Therefore I did not consider the ceiling load in analyzing the roof structure. However, I believe the members would be able to carry the ceiling if it were applied.

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A.I.S.C. Steel Construction Manual

Ginter L.E. Design of Modern Steel Structures

Malcolm C. W. Graphics Statics

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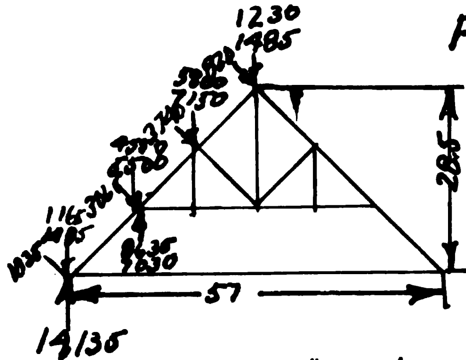
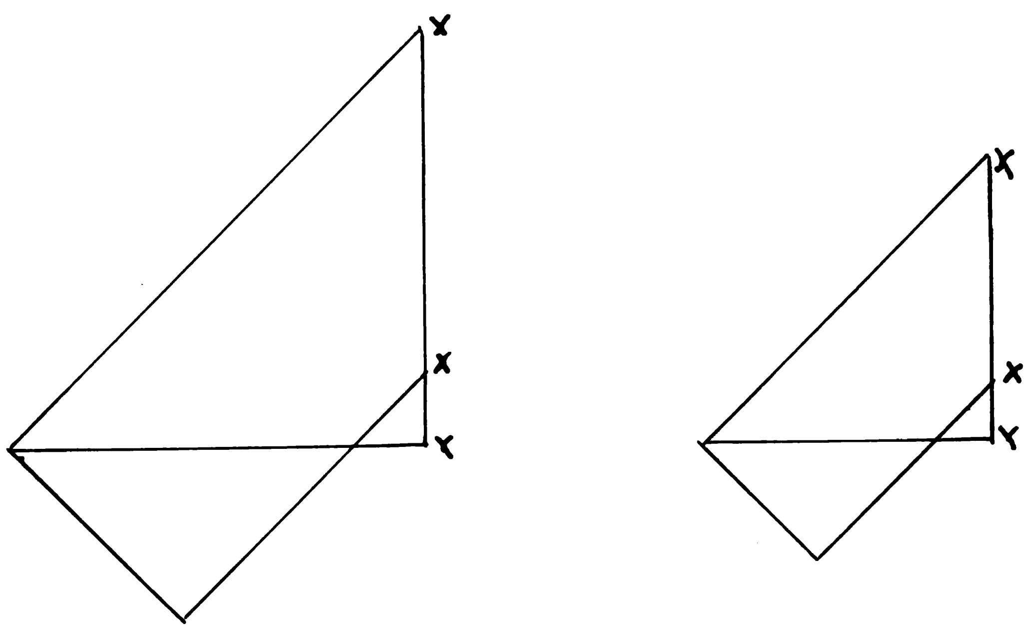
PLATE I



SCALE: 1" = 30'

MEM	D.L.	L.L	W.L.	DL+L.L	DL+½LL.W.
U_2U_3	-12,000	-9800	-4500	-21,800	-20,900
U_3U_4	-7200	-5700	-4600	-12,900	-14,560
U_3L_4	-5000	-4000	-4760	-9000	-11,760
U_4L_4	+7000	+6000	+5000	+13,000	+15,000
L_2L_4	+8500	+7000	+6500	+15,500	+18,500
1-2	-20,000	-18,000	-4250	-38000	-33,250



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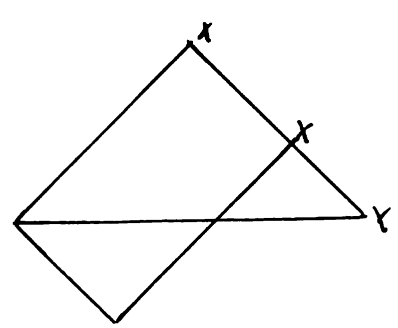

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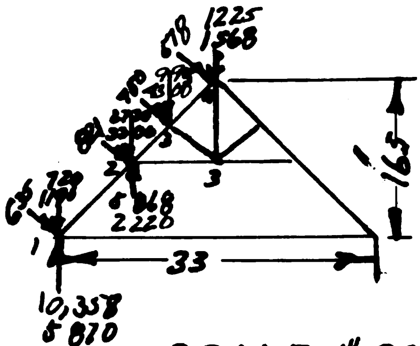
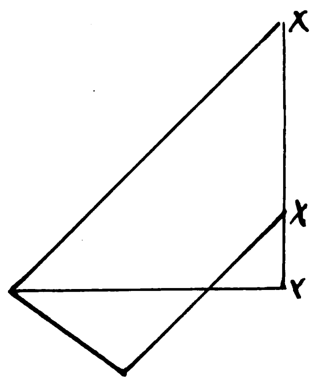
PLATE 2



SCALE: 1"=20'

MEM.	D.L.	L.L.	W.L.	D.L.+L.L.	D.L.+½L.L.+W.L.
U_2U_3	-8200	-3500	-1400	-11,700	-11,350
U_3U_4	-6200	-2500	+400	-7700	-7850
U_3L_3	-3000	-1000	-578	-4000	-4078
U_4L_3	+4400	+1400	+800	+5800	+5900
L_2L_3	+5800	+2500	+2000	+8300	+9050
1-2	-14,700	-8300	-2680	-23,000	-21,530



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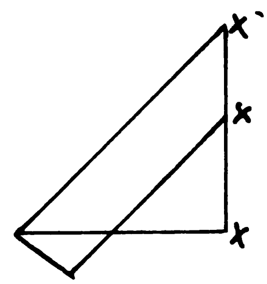

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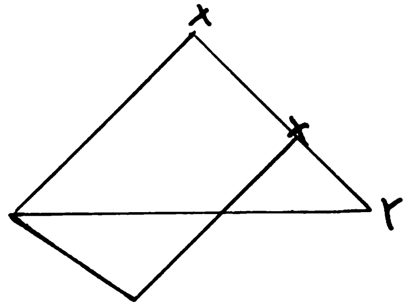
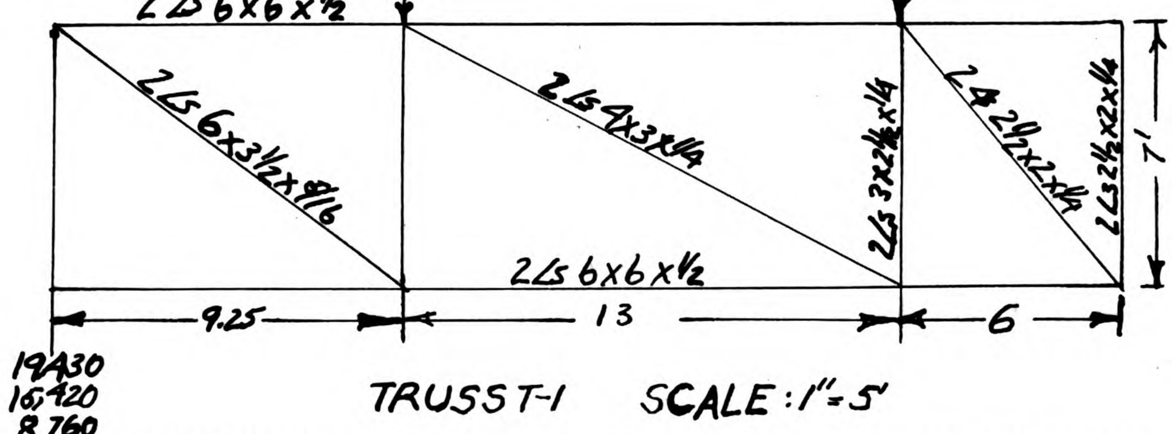


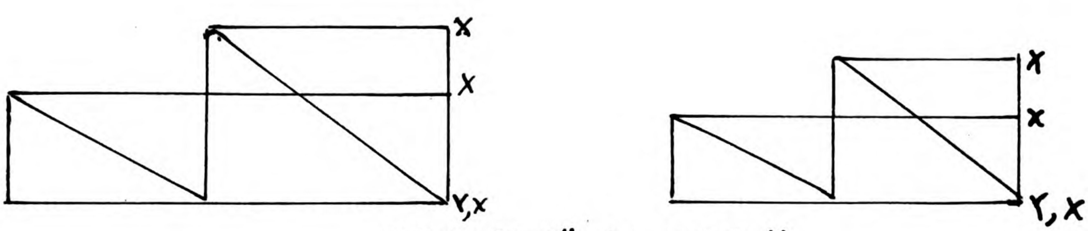
SCALE: 1"=1000#

PLATE 3



TRUSS T-1 SCALE: 1" = 5'

MEMBER	D.L.	L.L.	W.L.	D.L.+L.L.	D.L.+½L.L.+W.L.
$U_1 L_2$	+32,000	+26,000	+15,000	+58,000	+60,000
$U_2 L_3$	+25,000	+21,000	+10,000	+46,000	+45,500
$U_1 U_2$	-26,000	-21,000	-12,000	-47,000	-48,500
$U_2 U_3$	+48000	-39,000	-20,000	-87,000	-87,500
$U_3 U_4$	-48 000	-39,000	-20,000	-87,000	-87,500
$L_2 L_3$	+26,000	+21,000	+12,000	+47,000	+48,500
$L_3 L_4$	+48,000	+39,000	+20,000	+87,000	+87,500
$U_2 L_2$	-19,430	-16,420	-8760	-34,858	-35,900
$U_3 L_3$	-11,650	-9,045	-4760	-20,695	-20,900



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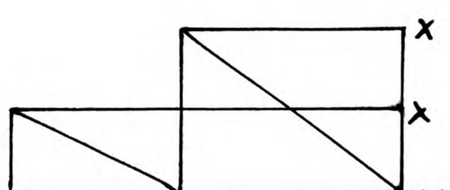
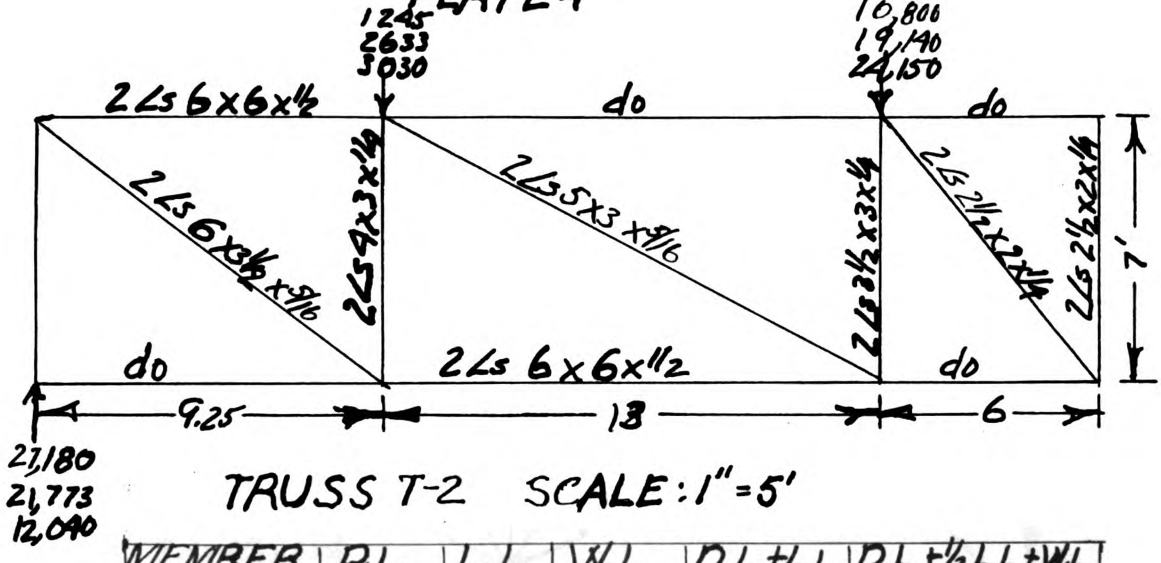


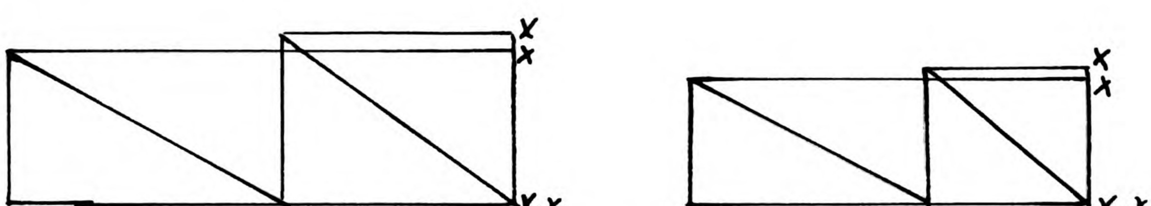
SCALE 1" EQUALS 10,000#

PLATE 4



TRUSS T-2 SCALE: 1" = 5'

MEMBER	D.L.	L.L.	W.L.	D.L.+L.L.	D.L.+½L.L+W.L.
U_1L_2	+45,000	+35,000	+20,000	+80,000	+82,500
U_2L_3	+51,000	+49,000	+22,000	+91,000	+93,000
U_1U_2	-35,000	-28,000	-16,500	-63,000	-65,500
U_2U_3	-80,000	-63,000	-36,000	-143,000	-147,500
U_3U_4	-80,000	-63,000	-36,000	-143,000	-147,500
L_2L_3	+35,000	+27,000	+16,500	+62,000	+50,000
L_3L_4	+80,000	+63,000	+36,000	+143,000	+147,500
U_1L_2	-27,180	-21,773	-12,040	-48,953	-50,106
U_2L_3	-24,150	-19,140	-10,800	-33,290	-44,520



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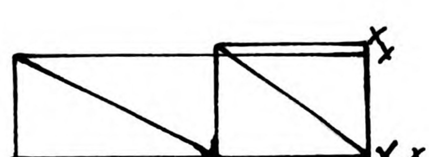

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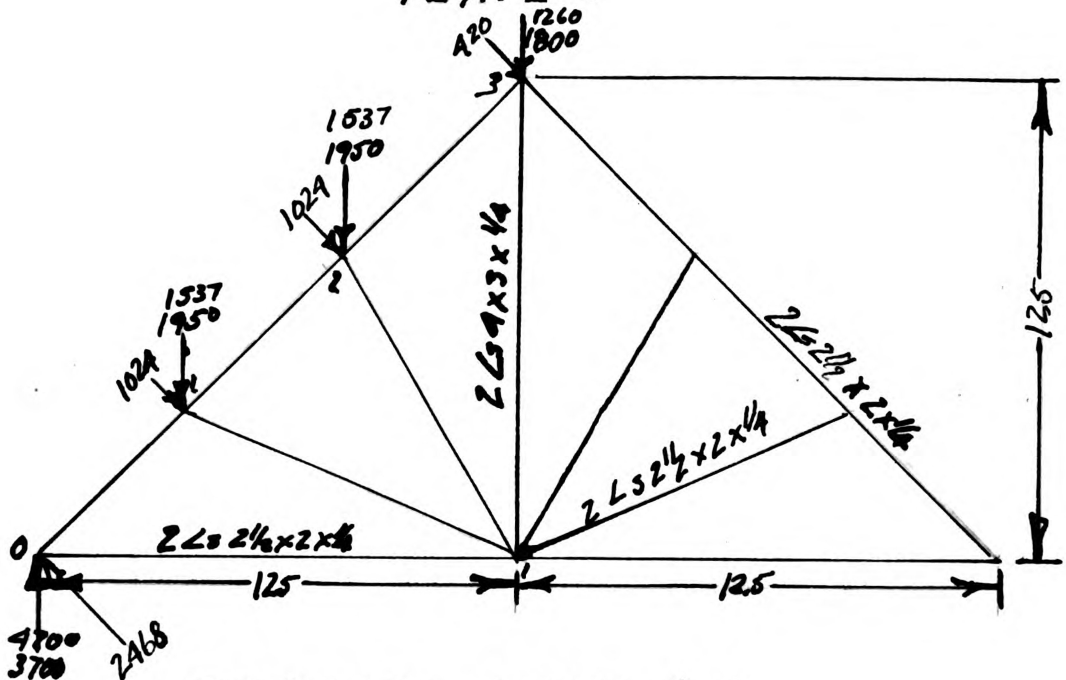
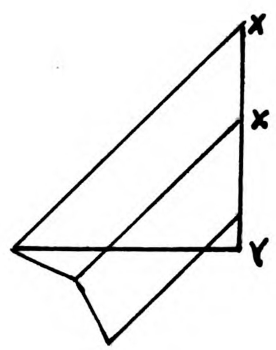
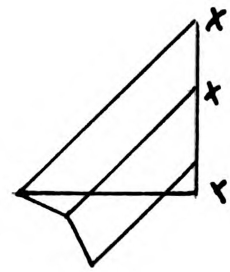
PLATE 5



TRUSS T-3 SCALE: 1"=5'

MEMBER	D.L.	L.L.	W.L.	D.L.+L.L.	D.L.+½L.L.+W.L.
$U_0\ U_1$	-6800	-5200	-2700	-12,000	-12,100
$U_1\ U_2$	-5800	-4700	-2500	-10,500	-10,650
$U_2\ U_3$	-4800	-3800	-2600	-8600	-9,300
$L_0\ L_1$	+4800	+3800	+3700	+8600	+10,400
$U_1 L_1$	-1400	-1000	-1000	-2400	-2900
$U_2\ L_1$	-800	-600	-600	-1400	-1700
$U_3 L_1$	+3000	+2400	2200	+5400	+6400





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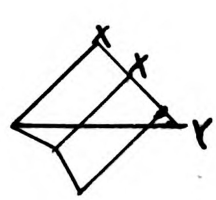
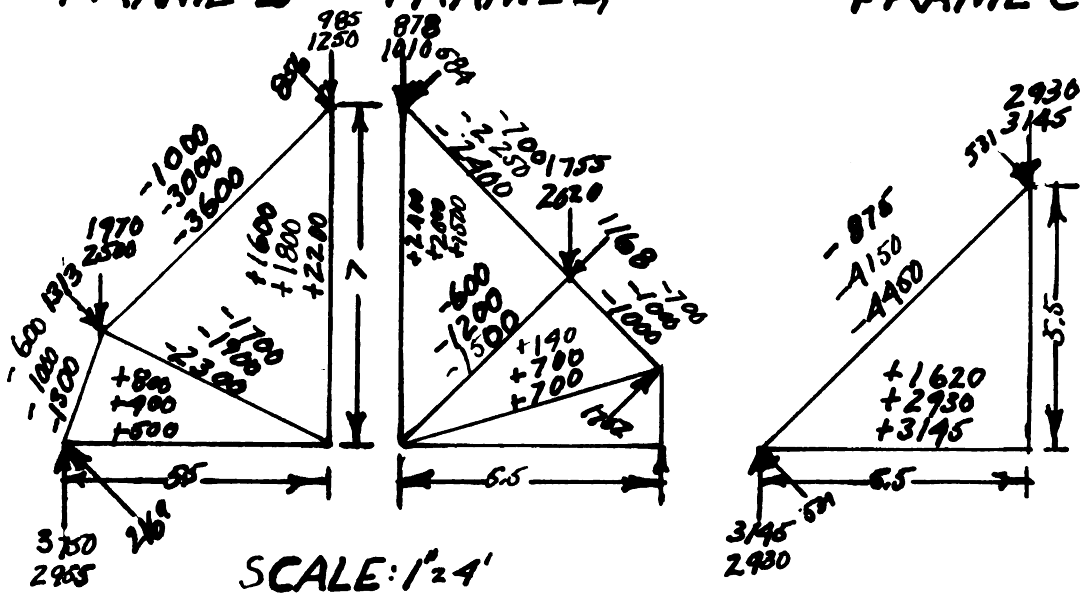


PLATE 6

FRAME B FRAME B$_1$ FRAME C



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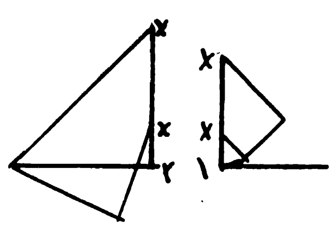

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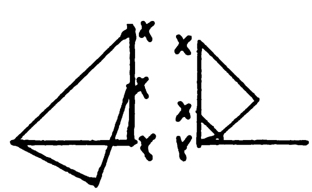


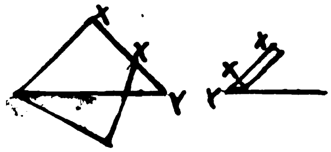
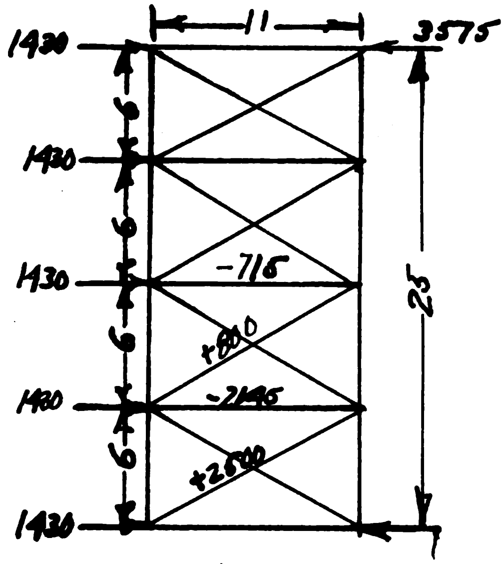
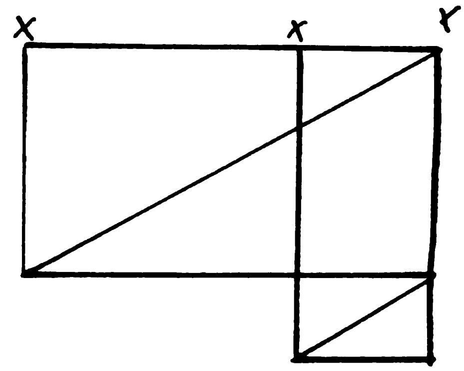


PLATE 7

LOWER CHORD BRACING



SCALE: 1"=10'



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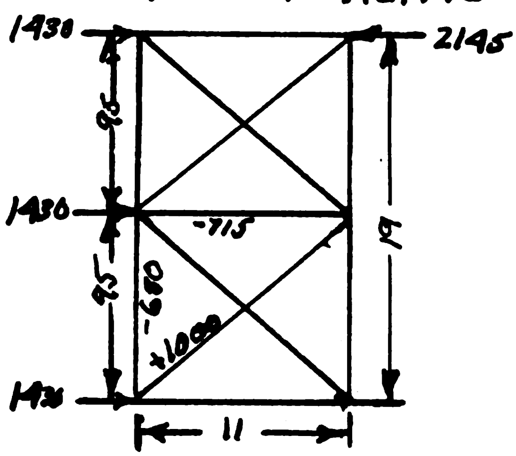
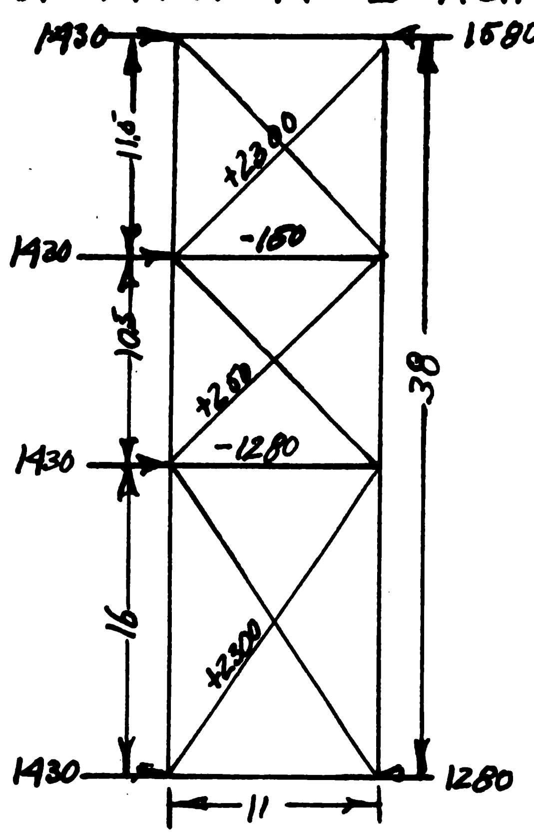
VERT. BRACING



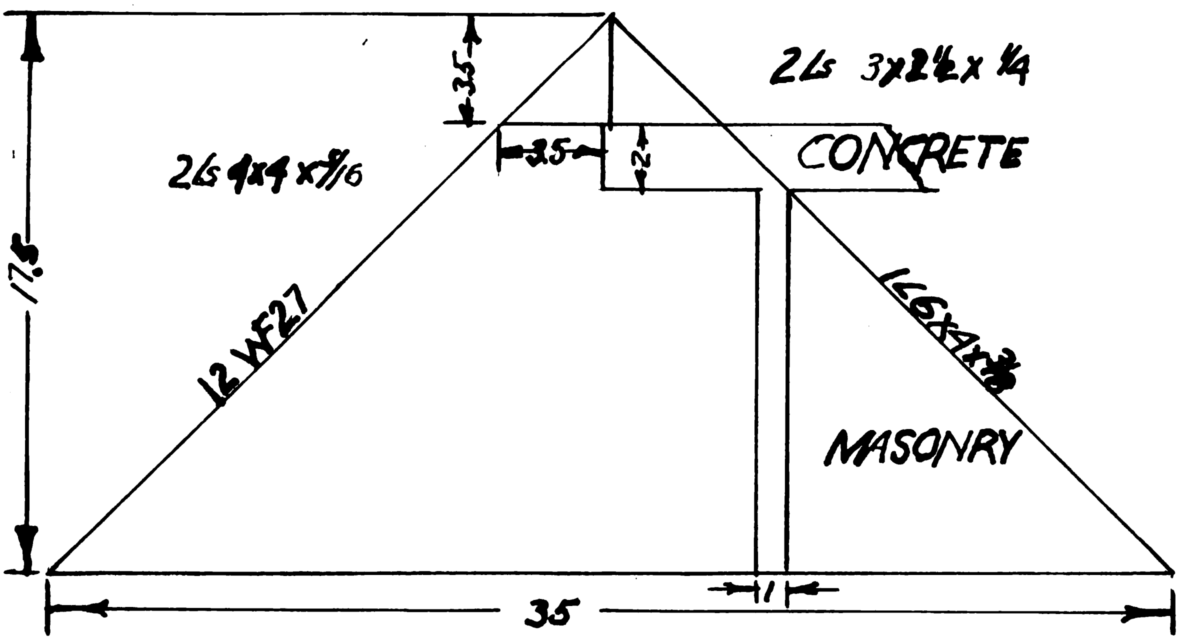
SCALE: 1"= 10'

UPPER CHORD BRACING



SCALE: 1"=10'

PLATE 8
SECTION E-E



SCALE: 1"=6'

ROOM USE ONLY