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Engineering Problems
in
Lansing Auditorium

A Thesis Submitted to
The Faculty of
Michigan Agricultural College

By

L. L. Weaver // E. H. Spencer

Candidates for the Degree of
Bachelor of Science

June, 1913

THESIS
137
621
THS

Preface

In preparing this report, the authors have attempted to present the methods used, citing the authority for such methods, for arriving at their conclusions as to the safety of the building under consideration. The authors have in general proceeded along lines as is given in various text* books, but at times have resorted to methods which may prove unique, but owing to their limited experience were not able to devise better means.

For the convenience of any person perusing this report, the authors have divided the work into sections numbered with Roman numerals, paragraphs lettered with capital letters, and sub-paragraphs numbered with the Arabic notation.

The authors express grateful acknowledgement of their indebtedness to Professors H. K. Vedder, and C. A. Melick of the Michigan Agricultural College for many helpful suggestions and criticisms during the inception and progress of this work.

They desire to extend their thanks to Architect E.A.Bowd for the blue-prints which he so kindly furnished them. Their indebtedness to the authors and co-authors of the following books is acknowledged;

American Civil Engineers' Pocket Book, Mansfield Merriman
Structural Engineers' Handbook, Milo S. Ketchum.
Mechanics of Materials, Mansfield Merriman.
Cambria Steel Handbook.
Cyclopedia of Architecture and Carpentry Building.
Design of Steel Mill Buildings, Milo S. Ketchum.
A Treatise on Masonry Construction, Ira O. Baker.
Roofs and Bridges, Merriman and Jacoby.

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Part I.

Introduction.

A. General discussion of theses.

Many papers and reports of the various designs and investigations of the works of man are classed under the head of thesis. A thesis may be the advancing of a theory and its proof by mathematical formulae, again it may be an essay upon some subject which the author desires to give all the information that can be found, in fact indications show that any idea which is put forth in the character of a report, may be classed as a thesis.

B. Discussion of this thesis.

In this thesis the authors have investigated a newly constructed building and have set forth their theories as to the safety of the Lansing Auditorium from the stand-point of a student engineer. Altho the investigators have had no practical experience along this line, they have set forth their views and ideas, as to the strength of the various parts of the structure, from the ideas which they have acquired during their course as civil engineering students.

The investigators are not desirous of being quoted as to their comments, but where they have noticed defects in workmanship and material they have felt justified in setting forth that knowledge.

In those places where the authors' work has shown that the member is not safe, thay bow down to the experienced architect's knowledge. In the numerous cases where the members are found to be too large, the authors have decided that their assumptions are incorrect or else the design govern rather than strength.

The Lansing Auditorium is located on Walnut street, south of Allegan street, being to the south and west of the State Capitol. The building plan has a diamond shape, the points facing to the north and south, and the entrance being on the east side of the building. The walls are constructed of brick with a terra-cotta ledge around near the top and a sand stone course about one third the distance up the wall from the ground.

The roof is nearly flat, enough slope being provided for the running of water into the conductors. The roof is constructed of tar paper on 7/8" sheathing. This is supported by several steel trusses and I beams. All the trusses are supported on columns that carry the weight of the roof direct to the piers and foundations. The ends of a few I beams rest on plates that are laid in the walls. The ceiling underneath the roof is carried by the roof rafters, thru boards nailed to the rafters and to the ceiling joist. The balconies are supported on wooden trusses which extend from the walls out to and overhanging plate girders. The plate girders being connected to the columns supporting the roof trusses. In the

blue-print, the design of these trusses calls for anchor bolts into the wall and that the parts of the trusses be bolted together with 1/2" bolts. Altho the authors looked this part over closely, they were unable to find any bolts of any kind. The first or main floor is supported on rafters resting on I beams, the I beams being supported by concrete piers. All floors are constructed of 1" hemlock base with 1" maple covering.

The ventilating fan with its motor is between the ceiling and roof over the entrance. The fan was constructed by the American Blower Co. The motor is a 5 H.P., 60 cycle, Western Electric Motor of the induction type, taking 4 to 8 amps. at 440 volts and running at 900 R.P.M. The leads from the motor were held down by nails driven into a board and then bentover, thus pressing the exposed metal against a wood board. The blue-prints call for ventilators in the roof to provide for the ventilation of the dead space between the roof and ceiling, observation failed to show them. The fresh air heating system is located in the basement under the stage. Radiators are provided in those parts of the bui;ding where they are needed. The auditorium has a seating capacity of about 2400 people. The balconies and part of the main floor have fixed seats amounting to 1400. The rest of the main floor has movable seats and will seat approximately 1000. Lighting of the auditorium proper is by the indirect method.

Entrance and portion of South-east side.

Entrance and portion of North-east side

Part II
Procedure.

A Loads.

1. Dead

The dead weight of the structure is taken from the summation of the approximate weights of the various parts. The following assumptions and their authority quoted, were used inarriving at the values given thrucut the work.

Tar paper roof covering,	S.H.B. pg 4	2#/sq.'
Yellow pine	" " 69	40#/ft^3
Structural steel	" " 69	490#/ft.^{3}
Plaster ceiling	" " 69	10#/sq.'
Common brick work	" " 69	120#/ft^3
Maple	A.C.E.P.B. pg. 367	40#/ft^3
Hemlock	S.H.B. pg. 69	25#/ft^3
Stone concreter	" " 69	150#/ft^3

Weight of structure

Weight of roof	140,400#	or	70.2 tons
" " ceiling	278,000#	or	139.0 "
" " roof trusses	68,436#	or	34.2 "
" " girders	99,820#	or	49.9 "
" " columns	19,528#	or	9.8 "
" " column bases	3,793#	or	1.9 "
" " floors	478,500#	or	239.25 "
" " foundations, walls	6,423,100#	or	3,211.55 tons
Total			3,756.00 "
Add 50% for details			1,878.00
Total weight of structure			5,634.00 Tons

Weight of fixtures.

Weight of the fixtures was approximated, after estimating the weight of one fixture, from knowing the weight of such articles, by multiplying the weight of one fixture by the number of such fixtures and then taking the nearest round number.

Weight of radiators	5,000#	or	2.5 tons
" " piping	1,000#	or	0.5 "
" " lighting system	800#	or	0.4 "
" " Seating system	2,400#	or	1.2 "
" " Miscellaneous articles	2,000#	or	1.0 "
Total weight of fixtures	11,200#	or	5.6 "

2. Live.

Movable

The minimum live load for movable and fixed seat auditoriums given by the Chicago Building Laws 1911 and the Schneider specification is 100#/ sq. ft. (see S.H.B. pgs. 71 & 72) All live loads on the floors were figured with this assumption.

Machinery.

The only machinery in this building is the fans and their accessories which are used in the ventilating system. Since one portion of this system is in the basement, the action of the fan has no great effect on the structure. The other portion of the system is supported on two channels which rest one end on the wall and the other end on truss 2. This load was used in the checking, and was assumed as 1900#.

3. Wind

The wind load was taken as 20#/ sq.ft. of vertical projection (see S.H.B. pgs. 71 & 72) Since the roof was nearly flat and the walls of the building projected above the roof, there was not any wind load calculation for the roof.

4. Snow.

The snow load was assumed as 25#/ sq.ft. of horizontal projection for flat roofs as is required by the Chicago Building Laws 1911 and the Schneider Specifications (see S.H.B. pgs. 71 & 72)

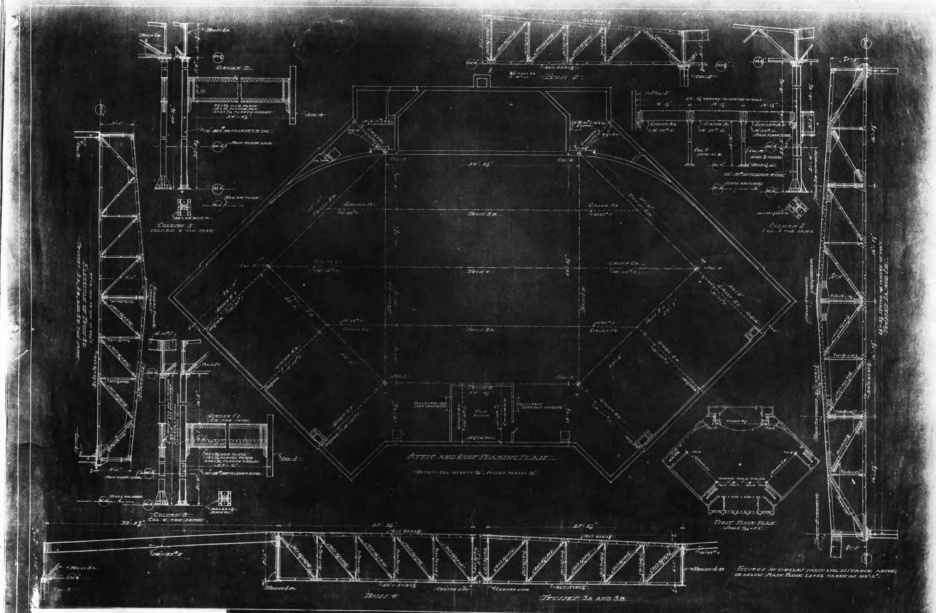
B. Roof

In arriving at the weight of the roof, the weight per sq. ft. of surface was found and this value was then multiplied by the area of the roof in sq. ft. In figuring the strength of the sheathing, the outline of C. A. Melick given in Fall Term Graphics was followed, using the value for the bending moment as $Wl^2/10$ where W equals the weight per sq. ft. and l, the span in ft. The strength of the rafters was figured along similiar lines as laid down by C.A. Melick.

1. Weight.

Area of roof over balconies.

2(12.6 x 73.66)	1860	sq. ft.
2(26.66 x 47.00)	2510	" "
2(12.70 x 14.00)	356	" "
2(12.70 x 12.70)	161	" "
18 x 18 x 54.4	980	" "
Total	5867	" "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Area of roof over main floor

54.4 x 66.4	3610	sq.ft.
2(66.4 x 33.2)/ 2	2220	" "
Total	5830	" "

Area of roof over stage

9 x 73.67	663	sq.ft.
71.48 x 54.4/ 2	630	" "
Total	1293	" "

Area of roof over lobby

71.48 x 4.19	300	sq.ft.
71.48 x 54.4/2	630	" "
Total	930	" "

Weight of roof over balconies and stage in #/ sq.ft.

Weight of tar paper roofing	2#	/sq.ft.
" " 7/8" matched Y.P. sheathing	3.5	#/sq.ft.
" " Y.P. rafters 1.625 x 40 / 12	5.4	" "
Total	10.9	" "

Weight of roof over main floor

Weight of tar paper roofing	2.0	#/sq.ft.
" " 7/8" matched Y.P. sheathing	3.5	" "
" " Y.P. rafters 1.625 x 40 x 8/ 144	3.6	" "
Total	9.1	" "

Weight of roof over lobby

Weight of tar paper roofing	2.0	#/sq.ft.
" " 7/8" matched Y.P. sheathing	3.5	" "
" " Y.P. rafters 1.625 x 40 x 10/ 144	4.5	" "
Total	10.0	" "

Total weight of roof

Weight of roof over balconies and stage	7160 x 10.9	--	78,100#
" " " " main floor	5830 x 9.1	--	53,000#
" " " " lobby	930 x 10.0	--	9,300#
Total			140,400#

Since the ceiling immediately under the roof is supported by the roof rafters, the weight of the entire ceiling is figured below.

Ceiling weight.

Lath and plaster	10#/	sq. ft.
Y.P. joists, 1.625 x 8, 16" o.c.	3.8#/"	"
Total	13. 8	" "

Use ceiling weight as 15#/sq.ft.

Area of ceiling under roof.

Area over main floor	5,830	sq.ft.
" " balconies	5,867	" "
" under "	4,571	" "
" over stage	1,302	" "
" " lobby	930	" "
Total area	18,500	" "

Weight of ceiling
18,500 x 15 ---------- 278,000#

2. Sheathing moments and shears.

Loads

Min. snow load ----------	25.0#/sq.ft.	
Tar paper roofing ----------	2.0 "	"
7/8" Y.P. sheathing ----------	3.5 "	"
Total	30.5 "	"

Bending moment--- $Wl^2/10$

M = 30.5 x 1.33 x 1.33 x 12 / 10 = 64.6"#

S = Mc/I = 64.6 x 6/12 x .875 x.875 = 42.2 #/sq. in.

Shear V = 30.5 x 1.33/2 = 20.2 #

S = 20.2/.875 x 12 = 1.9#/sq.in.

Allowed value for S is given as 1,300#/sq.in.(S.H.B.pg.58)

Deflection = 5 x W x l^3/ 384 x E x I

$$d = \frac{5 \times 30.5 \times 16 \times 16 \times 16 \times 12}{384 \times 805{,}000 \times 12 \times .875 \times .875 \times .875}$$

d = .000302 " loading deflection

d = 16/360 = .00445 " max. deflection permissable

From the above values it is shown that strength does not govern.

3. Rafter moments and shears.

Over balconies

Loads

Min. snow load ----------	25.0#/sq.ft.	
Tar paper roofing ----------	2.0 "	"
7/8" Y.P. sheathing ----------	3.5 "	"
Ceiling weight ----------	15.0 "	"
Rafters,Y.P. 1.625 x 40/12 ----------	5.4 "	"
Total	50.9 "	"

Bending moment ------ $Wl^2/8$

M = (16 x 50.9 x 23.5 x 23.5)/ (12 x 8) = 4,680'#

S =(4680 x 12 x 6)/(1.625 x 12 x 12) = 1,440#/sq.in.

Shear

V = (50.9 x 23.5 x 16)/(2 x 12) = 796#

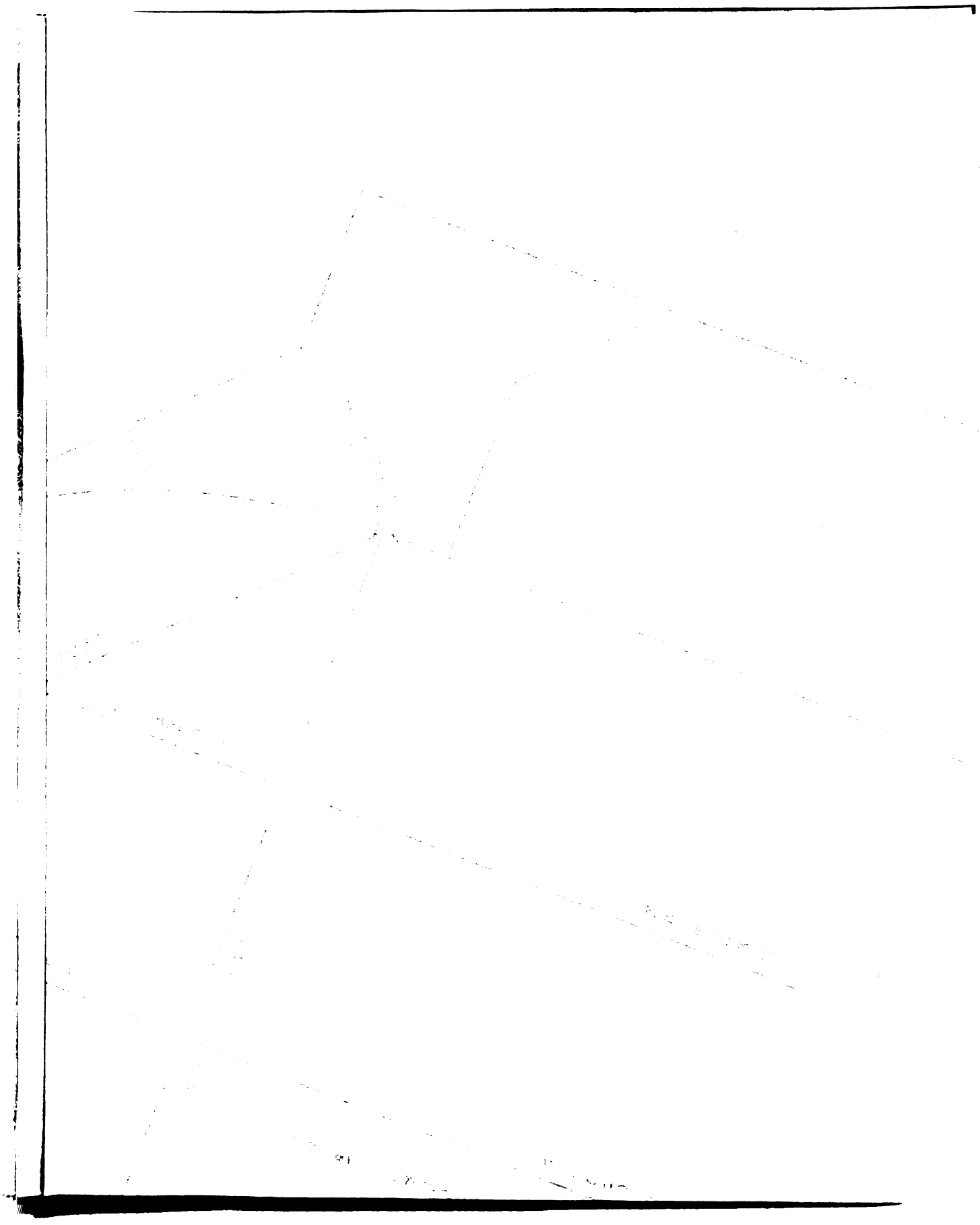
S = 796/1.625 x 12 = 40.8#/sq.in.

Allowed value as given in table pg. 298 S.H.B. for Y.P. is 1,300#/sq.in. On the same page the following information is given: "For buildings and similiar structures, in which the timber is protected from the weatherand practically free from impact, the unit stress may be increased fifty (50) per cent." Therefore S = 1,300 x 1.5 = 1,950#/sq.in.

Over Main floor.

Loads

Min. snow load ----------	25.0#/sq.ft.	
Tar paper roofing ----------	2.0 "	"
7/8# Y.P. sheathing ----------	3.5 "	"
Ceiling weight ----------	15.0 "	"
Y.P. rafters (1.625 x 8 x 40)/144 ----------	3.6 "	"
Total	49.1 "	"



Bending moment ----- $Wl^2/8$

M = (16 x 49.1 x 16.61 x 16.61)/(12 x 8) = 2,260'#

S = (2260 x 12 x 6)/(1.625 x 8 x 8) = 1,560 #/sq.in.

Allowed value for S is 1,950 #/sq.in.

Shear

V = 49.1 x 16.61 x 16/24 = 530#

Over stage.

Since the construction above the stage can be treated as beams or as trusses, values were found by both mathods, the method of beams given here, and the method of trusses given under trusses.

Area of roof ----------	765.8 sq.ft.
Loads	
Snow --- 765.8 x 25 ----------	19,160#
Tar paper roofing---- 765.8 x 2 ----------	1,531#
Sheathing Y.P. ------ 765.8 x 3.5 ---------	2,685#
Y.P. rafters ---- 35x40x1.625x12x17.5/1728-	3,385#
Total	26,762#
26,762/765.8 ----------	35#/sq.ft.
Load per rafter 26,762/35 ----------	763#

Bending moment ------ $Wl^2/8$

M------(35 x 17.5 x 17.5/8) (16/12) ---- 1,790'#

S ----- 1790 x 12 x 6/(1.625 x 12 x 12)--- 550#/sq.in.

Allowed value for S is 1950#/sq.in.

Shear

V ---- 763/2 ---------- 381.5#

Over lobby

Loads

Min. snow load ----------	25.0#/sq.ft.
Tar paper roofing ----------	2.0 " "
½/8" Y.P. sheathing -------- ----------	3.5 " "
Y.P. rafters 1.625 x 10 ----------	3.4 " "
Ceiling weight ----------	15.0 " "
Total	48.9 " "

Bending moment

M ---- 48.9 x 17 x 17x 16/(8 x 12) -----2,350'#

S ---- 2,350 x 6 x 12/(1.625 x 100) ----1,040#/sq.in.

Allowed value 1,950#/sq.in.

Shear

V ---- 48.9 x 17/2 ---------- 553#

C. Trusses

The weights of the trusses was figured by multiplying the lengths of the various members by the weight per foot and then adding 50% for details such as rivets and gussett plates. The max. bending moment was found by finding the reactions due to the loads and then multiplying this reaction by the length and then dividing by 4. The max. shear was taken as the reaction of the truss. The resisting stresses was found by multiplying the allowed unit by the area of the piece.

The allowed unit for tension in steel was taken as16000#/sq.in. (see S.H.B. pg 57) and for compression 16000- 70 l/r (see S.H.B. pg. 80, table 9, A.R.E.A., Chicago, and Ketchum Spec.) with a max. of 14000#/sq.in. The determination of the stresses in the members of the trusses was found by the use of the three fundamental equations of equilibrium

Summation of horizontal components of forces equals 0
" " vertical " " " " 0
" " moments of forces about any point " 0

The stresses for each member of each truss using the max. loads are tabulated ubder the force diagram of the truss.

1. Weights.

Truss 1A and 1B (half-truss)

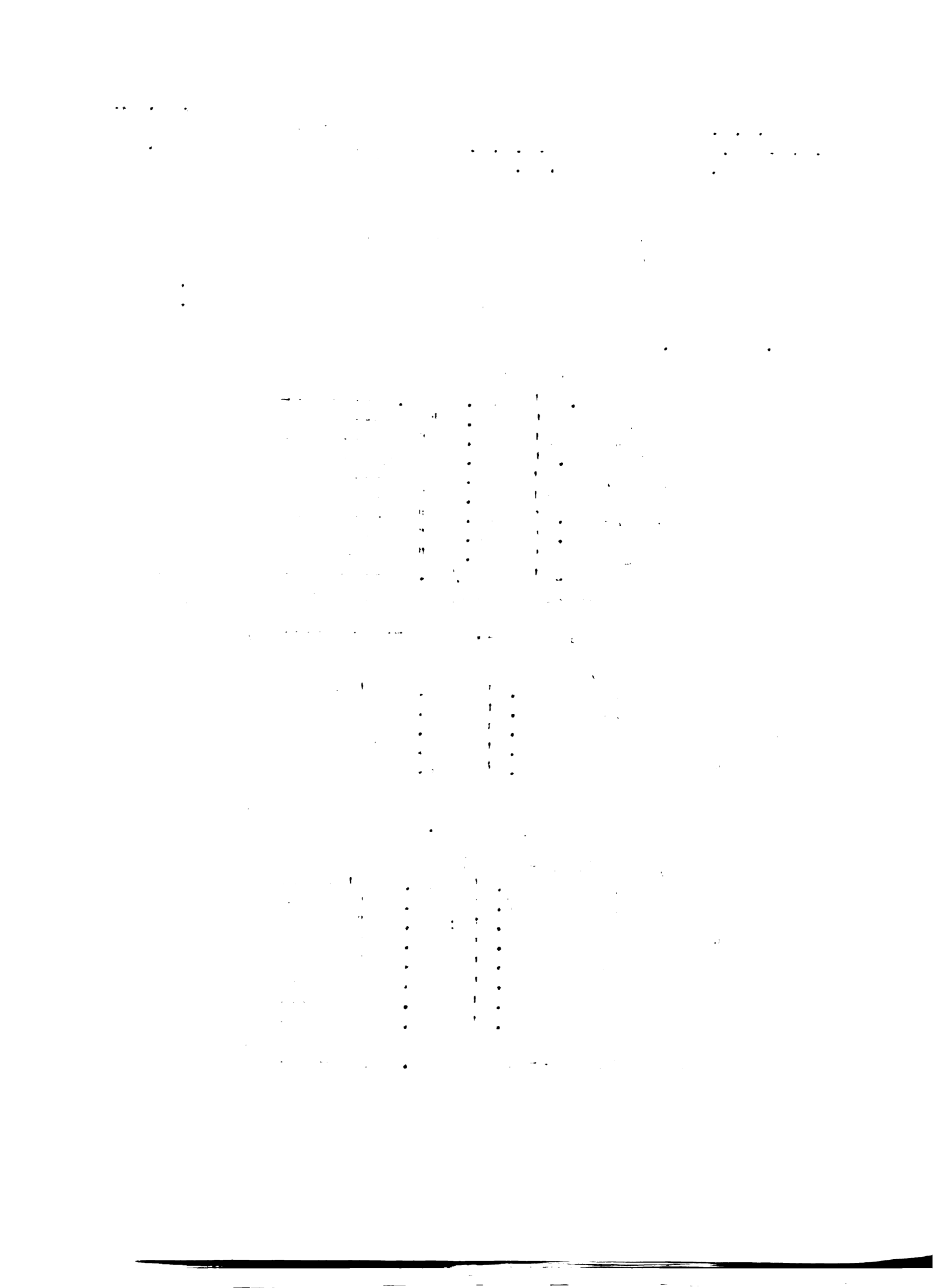
4 angles	6x6x½	35.25' @ 28.7#/ft.	———	3,820#
2 "	2½x2½x¼	7' @ 4.1 "	———	57#
2 "	2½x2x¼	9' @ 3.62 "	———	65#
2 "	"	5.5' @ 3.62 "	———	40#
2 "	2½x2x5/16	9' @ 4.5 "	———	81#
4 "	2½x2½x¼	5' @ 4.1 "	———	82#
2 "	5x3½x5/16	8.5' @ 14.3 "	———	244#
2 "	2½x2x¼	4.5' @ 3.62 "	———	33#
2 "	6x6x½	9' @ 19.6 "	———	354#
1 Plate	14x½	15' @490#/ft.^{2}	———	310#
Gussett plates	2x15x½ & 6x2x½	@ 490#/ft^2	———	470#
		Total		5,556#
Weight of truss	5,556 x 1.5 x 2		———	16,700#

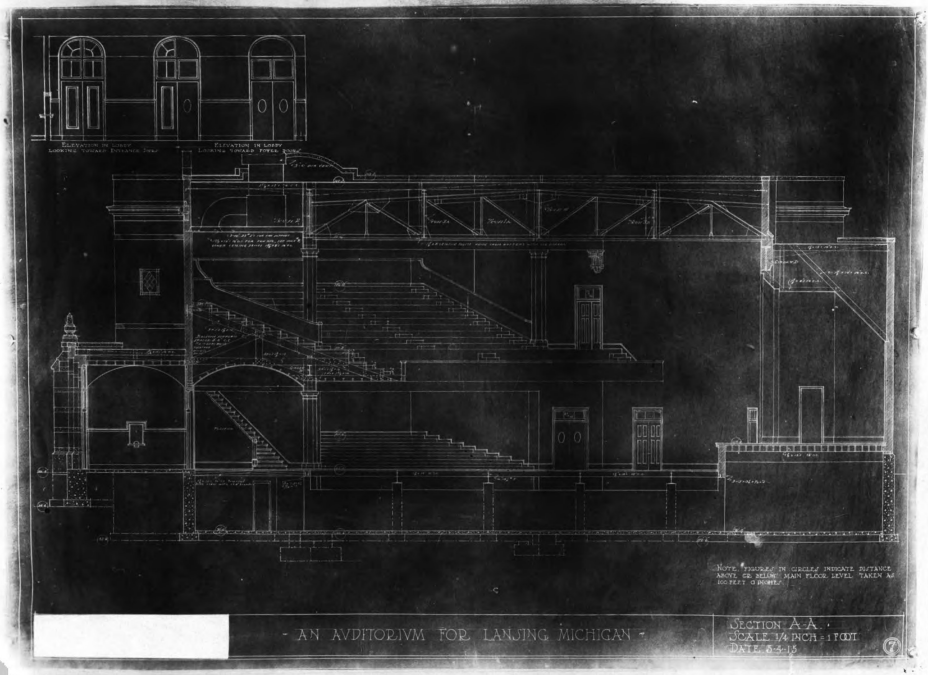
Truss 2(half-truss)

2 angles	6x4x3/8	27.5' @ 12.30#/'	———	675#
2 "	5x3x5/16	27.2' @ 8.20 "	———	415#
7 "	2½x2x¼	6.5' @ 3.62 "	———	165#
8 "	"	7.3' @ 3.62 "	———	210#
2 "	3½x2½x3/8	7.0' @ 7.20 "	———	70#
Gussett plates				350#
		Total		1,885#
Weight of truss	-- 1,885 x 1.5 x 2		———	5,660#

Truss 3A, 3B, & 4 (half-truss)

2 angles	6x4x3/8	27.2' @ 12.30#/'	———	666#
2 "	5x3x5/16	27.2' @ 8.20 "	———	444#
6 "	2½x2x¼	8.3' @ 3.62 "	———	180#
2 "	2½x2x5/16	8.3' @ 4.50 "	———	75#
2 "	3½x2½x¼	8.3' @ 4.90 "	———	81#
4 "	2½x2x¼	6.7' @ 3.62 "	———	97#
4 "	3x2½x¼	6.3' @ 4.50 "	———	113#
1 "	2½x2x¼	7.0' @ 3.62 "	———	25#
		Total		1,681#
Weight of truss	-- 1,681 x 1.5 x 2		———	2,520#




ELEVATION IN LOBBY
LOOKING TOWARD ENTRANCE DOOR
ELEVATION IN LOBBY
LOOKING TOWARD TOWER DOOR
NOTE: FIGURES IN CIRCLES INDICATE DISTANCE ABOVE OR BELOW MAIN FLOOR LEVEL TAKEN AS 100 FEET 0 INCHES
- AN AUDITORIUM FOR LANSING MICHIGAN -
SECTION A-A
SCALE 1/4 INCH = 1 FOOT
DATE 5-5-15
7

Truss 5A, 5B, 5C, & 5D. (half-truss)

2 angles 5x3½x3/8	23.5'	@ 10.40#/'	-----	490#
2 " 3½x2½x5/16	23.5'	@ 6.10 "	-----	287#
2 " 2½x2½x¼	7.0'	@ 4.10 "	-----	58#
4 " 2½x2½x¼	7.5'	@ 4.10 "	-----	168#
4 " 2½x2x¼	6.7'	@ 3.62 "	-----	97#
4 " 3x2½x5/16	7.0'	@ 5.60 "	-----	157#
Gussett plates		@ 490#/ft.²	-----	450#
Total				1707#
Weight of truss -- 1707 x 1.15 x 2			----------	3414#

Balcony truss section BB

(4.875 x1 x 38 x 40)/12	616#
(1.625 x 1 x 11 x 40)/12	60#
(3.250 x 1 x 28 x 40)/12	304#
(4.875 x 1 x 14.5 x 40)/12	236#
Total	1216#
14 trusses @ 1216#	17000#

Balcony truss section AA

40(4.875 x 1 x 38.1)/12	626.6#
40(1.625 x 1 x 32.7)/12	177.4#
40(3.250 x 1 x 21.0)/12	228.0#
Total	1032.0#
18 trusses @ 1032#	18600.0#

2. Max. Bending Moment.

Truss 1A & 1B

Loads from girder F and Truss 3A concentrated at U_3
24,720 plus 3,024 ---------- 27,744#

Loads from girder G and Truss 4 concentrated at U_5
24,720 plus 14,175 ---------- 38,895#

Reaction ---- 27,744 & 38,895/2 & 8,350 ---------- 55,541#

M =(55,541 x 33.23) - (27,536 & 8,350)16.61
---------- 1,789,400'#

Truss 2

Load from channell K concentrated at L_5 ------ 725#

Rafter loads = 1110/2 & 18.04 x 54.375 x 50.9/80 -- 1179#

Reaction = 1179 x 20 & 725 & 2830 = -- 26,135#

M = (26,135 x 27.18) - (1179 x 20 x 27.18/2) -
(725 x 5.5) -(2830 x 27.18/2) = 347,720'#

Truss 3A, 3B, 4

Rafter load (16.61 x 49.1 x 16)/12 = 1110#

Reaction = (20 x 1110) & 2520 = 24,720#

M =(24,720 x 27.2) - (1110 x 20 x 13.6) = 271,000'#

Truss 5A &5B

Load from girder F ---------- 2,648#
Rafter loads 15,000# & 6,800# ---------21,800#
Reaction = 2,648 & 7,500 & 3,400 & 1707 = 15,075#
M =(15,075 x 23.5) - (26800 & 112000 & 20200)= 195000'#

Truss 5C & 5D

Loads from girder F & E^3 (2468 & 16736) ---- 19204#
Rafetr loads ---------- 6800#
Reaction 6800/2 & 19204/2 & 1707 ---------- 14709#
M =(14709 x 23.5) - 48500 ----------297500'#

3. Max. Shears.

The reaction is taken as the max. shear.

Truss 1A & 1B	55541#
" 2	26135#
" 3A, 3B, 4	24720#
" 5A & 5B	15075#
" 5C & 5D	14709#

4. Resisting stresses.

Truss 1A & 1B

Panel length ---(16' 7 3/8")/2 --- 8' 3 7/16" --- 99 7/16"

Member	Make up		Area	All'd unit Stress	
L_1L_5	2 angles	6x6x$\frac{3}{4}$	14.26sq.in	16,000#/sq"	282.2Kips
U_3U_5	2 "	" pl.	23.88	12,990	311.0
U_1U_3	2 "	"	16.88	12,990	216.0
U_4L_5	2 "	2$\frac{1}{2}$x2x$\frac{1}{4}$	1.64	16,000	26.8
U_3L_4	2 "	2$\frac{1}{2}$x2x5/16	2.08	16,000	33.2
U_2L_3	2 "	5x3x9/16	8.36	16,000	133.8
U_2L_1	2 "	6x6x$\frac{7}{16}$	11.50	12,710	146.0
U_2L_2	2 "	2$\frac{1}{2}$x2x$\frac{1}{4}$	1.64	16,000	26.8
U_5L_5	4 "	"	4.24	9,950	42.2
U_4L_4	2 "	"	2.12	6,600	14.0
U_3L_3	4 "	2$\frac{1}{2}$x2$\frac{1}{2}$x$\frac{1}{4}$	4.76	11,860	56.5
U_1L_1	----------		-----	-------	------

Truss 2

Panel length 85.25"

Member	Make up		Area	All'd ubit	Stress
L_1L_6	2 angles	5x3x5/16	4.26	16,000	68.0
U_1U_6	2 "	6x4x3/8	7.22	12,050	87.0
U_5L_6	2 "	2$\frac{1}{2}$x2x$\frac{1}{4}$	1.68	16,000	27.8
U_4L_5	2 "	"	1.68	16,000	27.8
U_3L_4	2 "	"	1.68	16,000	27.8
U_2L_3	2 "	"	1.68	16,000	27.8
U_2L_1	2 "	3$\frac{1}{2}$x2$\frac{1}{2}$x3/8	4.22	7,000	29.6
U_6L_6	2 "	2$\frac{1}{2}$x2x$\frac{1}{4}$	2.12	5,800	12.5
U_5L_5	2 "	"	2.12	6,440	13.6
U_4L_4	2 "	"	2.12	7,000	14.8
U_3L_3	2 "	"	2.12	7,600	16.8
U_1L_1	----------		-----	------	----

Truss 3A & 3B
panel length 65.25"

Memebr	Make up	Area	All'd unit	Stress
L_1L_6	2 angles 5x3x5/16	4.26	16,000	68.0
U_1U_6	2 " 6x4x3/8	7.22	12,060	87.0
U_5L_6	2 " 2½x2x¼	1.64	16,000	26.8
U_4L_5	2 " "	1.64	16,000	26.8
U_3L_4	2 " "	1.64	16,000	26.8
U_2L_3	2 " 2½x2x5/16	2.08	16,000	33.3
U_1L_2	2 " 3½x2½x¼	2.44	16,000	39.0
U_1L_1	----------	----	------	----
U_2L_2	2 angles 3x2½x¼	2.88	8,850	25.3
U_3L_3	2 " "	2.88	8,600	24.8
U_4L_4	2 " 2½x2x¼	2.12	6,400	13.6
U_5L_5	2 " "	2.12	6,170	13.1
U_6L_6	2 " "	2.12	5,910	12.5

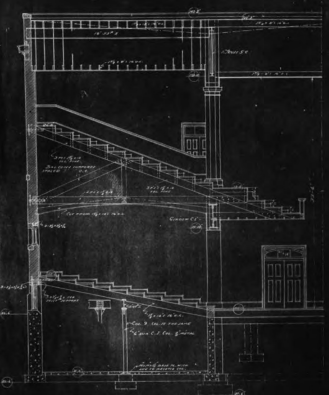
Truss 4
panel length 5.43'

Member	Make up	Area	All'd ubit	Stress
L_1L_6	2 angles 5x3x5/16	4.26	16,000	68.1
U_1L_2	2 " 3½x2½x¼	2.44	16,000	39.0
U_2L_3	2 " 2½x2x5/16	2.08	16,000	33.2
U_3L_4	2 " 2½x2x¼	1.64	16,000	26.8
U_4L_5	2 " "	1.64	16,000	26.8
U_5L_6	2 " "	1.64	16,000	26.8
U_6L_6	2 " "	2.12	5,900	13.1
U_5L_5	2 " "	2.12	5,900	13.1
U_4L_4	2 " "	2.12	5,900	13.1
U_3L_3	2 " 3x2½x¼	2.62	7,950	20.8
U_2L_2	2 " "	2.62	7,950	20 .8
U_1U_6	2 " 6x4x3/8	7.22	12,400	89.5
U_1L_1	----------	----	------	----

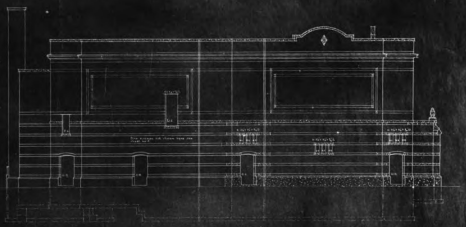
Truss 5A, 5B, 5C, & 5D.
panel length 5.87'

member	Make up	Area	All'd unit	Stress
L_1L_5	2 angles 3½x2½x5/16	3.02	16,000	48.4
U_2L_3	2 " 3x2½x5/16	2.70	16,000	43.2
U_3L_4	2 " 2½x2x¼	1.64	16,000	26.2
U_4L_5	2 " "	1.64	16,000	26.2
U_1U_5	2 " 5x3½x3/8	6.10	11,150	68.0
U_5L_5	2 " 2½x2x¼	2.22	5,900	13.5
U_4L_4	2 " "	2.12	6,880	14.6
U_3L_3	2 " "	2.12	7,880	16.7
U_2L_1	2 " 3½x2½x5/16	3.56	6,820	24.3
U_1L_1	----------	----	------	----

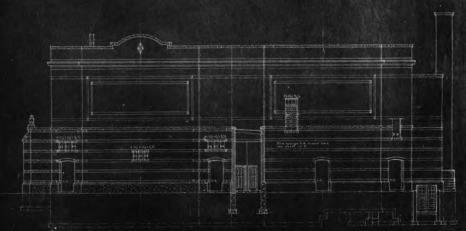
In all the above tables the member notation corresponds to the same member notation as is given on the blue prints of the truss diagrams. The size of the angles is in inches. The area is in square inshes, the allowed unit in pounds per square inch, and the stress is in kips.



SECTION D-D.



SOUTH ELEVATION



NORTH ELEVATION

NOTE-HEAVY LINES OVER OPENINGS INDICATE STEEL & R UNLESS OTHERWISE MARKED. OPENINGS WITH LIKE LETTERS ARE ALIKE.

- AN AUDITORIUM FOR LANJING MICHIGAN -

ELEVATIONS & SECTION D-D.
SCALE. 1/8 INCH = 1 FOOT.
DATE. 5-4-15.

Balcony truss, section BB.
14 trusses spaced 40¼" on centers.
Loads

Live	100#/sq.ft.
1" hemlock	2.08 #/sq.ft.
1" maple	3.33 #/sq.ft.
Total	105.41 #/sq.ft.

Use load for floor as 106#/sq.ft.

106 x 40.25/12	356#/lin.ft.of truss
Ceiling 15 x 40.25/12	51#/ " " " "

Loads as shown on diagram are as follows

CD	2698#
DE	5396#
EF	4797#
FG	2027#
GH	310#
HI	815#
IA	433#

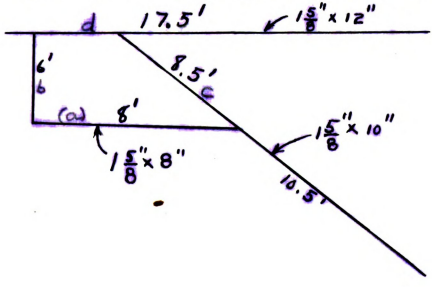
The stress as given on the diagram were figured out by the finding of the vertical and horizontal components of the various parts and then solving for their stresses.

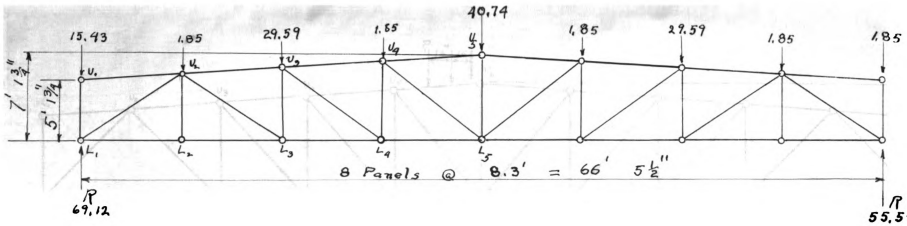
Balcony truss, section AA
18 trusses spaced 3' On centers
Floor load use 106#/sq.ft.
Loads as shown on the diagram are as follows

BC	1755#
CD	3802#
DE	3082#
EF	2450#
FG	466#
GH	393#
IJ	181#
HI	181#
dK	140#

The stresses ase given on the diagram was figured as the trusses above.

Roof truss over the stage supporting the roof and ceiling.

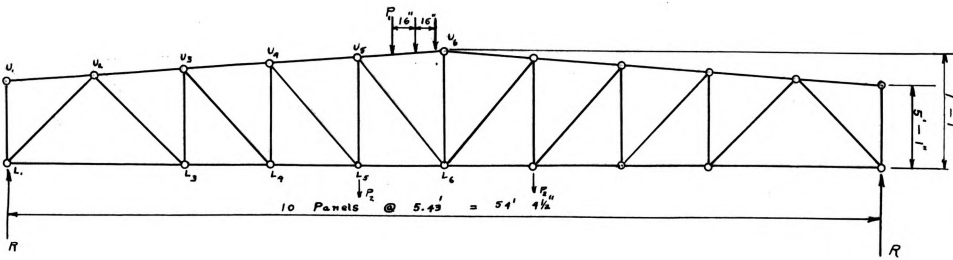




Member	Make — up	Area $\square''$	Length $'$	r $''$	l/r	Allowed unit	Stress
L_1L_2	2 ∠s 6 x 6 x 3/4	16.88	8.3	1.83	54.4	16,000 $^{\#}/_{\square''}$	77.5 $^{\#}/_{\square''}$
L_2L_3	2 ∠s 6 x 6 x 3/4	16.88	8.3	1.83	54.4	16,000	76.5
L_3L_4	2 ∠s 6 x 6 x 3/4	16.88	8.3	1.83	54.4	16,000	143.5
L_4L_5	2 ∠s 6 x 6 x 3/4	16.88	8.3	1.83	54.4	16,000	161.4
U_2L_3	2 ∠s 5 x 3 x 5/16	8.36	10.0	0.82	146.5	16,000	77.4
U_3L_4	2 ∠s 2½ x 2 x 5/16	2.62	10.2	0.58	211.5	16,000	21.9
U_4L_5	2 ∠s 2½ x 2 x 1/4	2.12	10.6	0.59	215.5	16,000	21.2
U_3L_4	2 ∠s 2½ x 2 x 1/4	2.12	5.6	0.59	113.7	16,000	0
U_1L_2			5.1				13.4
U_2L_3	4 ∠s 2½ x 2½ x 1/4	4.76	8.1	1.29	59.1	11,860	44.2
U_4L_4	2 ∠s 2½ x 2 x 1/4	2.12	6.6	0.59	134.1	6,600	12.5
U_5L_5	4 ∠s 2½ x 2 x 1/4	4.24	7.1	0.99	86.4	9,950	19.1
L_1U_2	2 ∠s 6 x 6 x 1/2	11.50	10.1	1.86	65.2	11,440	90.5
U_1U_2	2 ∠s 6 x 6 x 3/4	16.88	8.4	1.83	55.1	12,140	0.5
U_2U_3	2 ∠s 6 x 6 x 3/4	16.88	8.4	1.83	55.1	12,140	143.3
U_3U_4	2 ∠s 6 x 6 x 3/4	23.88	8.4	4.9	20.6	14,000	161.8
U_4U_5	2 ∠s 6 x 6 x 3/4	23.88	8.4	4.9	20.6	14,000	180.5

+ 1 Plate $14 \times \frac{1}{2}$

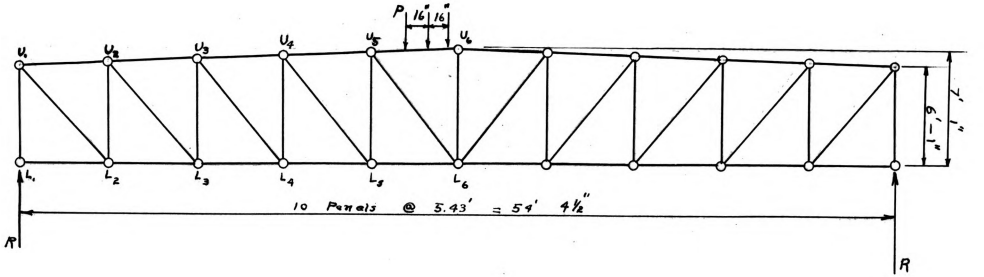
TRUSS 1 A, 1 B
C.E. THESIS M.A.C.
L. L. WEAVER E. H. SPENCER
SCALE 1" = 8' APRIL 6, 1916



Member	Make — up	Area	Length	r	l/r	Allowed unit	Stress
U_1L_1			5.1'				5.5 Kips
U_1L_3	2 ∠s 2½ x 2 x ¼	2.12 □"	5.9	0.59"	120.0	7,600 #/□"	18.3
U_2L_3	2 ∠s 2½ x 2 x ¼	2.12	6.3	0.59	128.4	7,000	19.3
U_2L_4	2 ∠s 2½ x 2 x ¼	2.12	6.7	0.59	136.8	6,440	8.5
U_4L_4	2 ∠s 2½ x 2 x ¼	2.12	7.1	0.59	144.0	5,900	2.6
L_1L_3	2 ∠s 5 x 3 x 5/16	4.80	10.8	0.85	131.1	16,000	19.9
L_3L_4	2 ∠s 5 x 3 x 5/16	4.80	5.4	0.85	73.6	16,000	38.0
L_4L_5	2 ∠s 5 x 3 x 5/16	4.80	5.4	0.85	73.6	16,000	51.1
L_5L_6	2 ∠s 5 x 3 x 5/16	4.80	5.4	0.85	73.6	16,000	59.0
U_1U_2	2 ∠s 6 x 4 x 3/8	7.22	5.5	1.17	56.4	16,000	31.2
U_2U_3	2 ∠s 6 x 4 x 3/8	7.22	5.5	1.17	56.4	12,050	40.0
U_3U_4	2 ∠s 6 x 4 x 3/8	7.22	5.5	1.17	56.4	12,050	53.2
U_4U_5	2 ∠s 6 x 4 x 3/8	7.22	5.5	1.17	56.4	12,050	61.1
U_5U_6	2 ∠s 6 x 4 x 3/8	7.22	5.5	1.17	56.4	12,050	67.6
U_5L_5	2 ∠s 3½ x 2½ x ½ x 3/8	4.22	7.7	0.72	128.4	7,000	29.4
U_4L_5	2 ∠s 2½ x 2 x ½	2.12	7.9	0.59	160.8	16,000	26.8
U_4L_4	2 ∠s 2½ x 2 x ½	2.12	8.1	0.54	165.2	16,000	19.5
U_4L_5	2 ∠s 2½ x 2 x ½	2.12	8.4	0.59	170.2	16,000	12.1
U_5L_6	2 ∠s 2½ x 2 x ¼	2.12	8.6	0.59	173.1	16,000	10.9

R = 26,135#
P_1 = 1,179#
P_2 = 725#
W = 8,660#

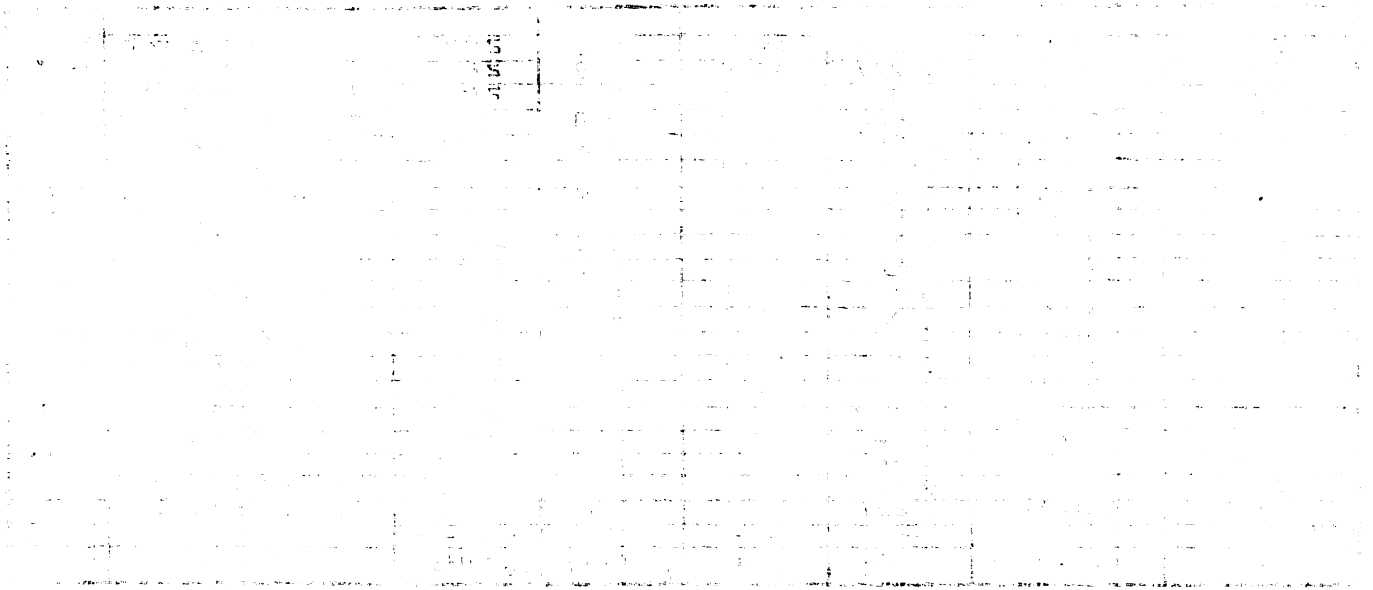
TRUSS 2
C. E. THESIS M.A.C.
L.L. WEAVER. E.H. SPENCER.
SCALE 1" = 6' APRIL 6, 1916

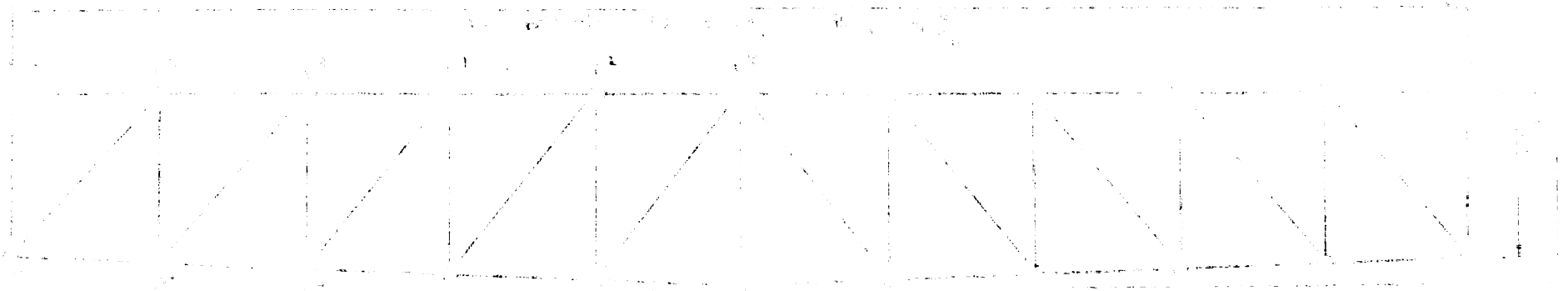


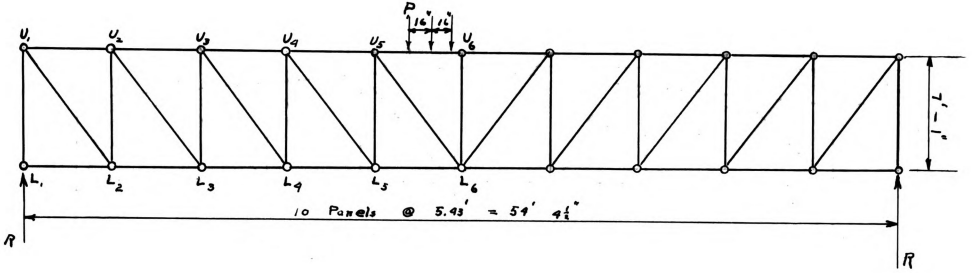
P = 1,110#
R = 29,720#
W = 50,90#

Member	Make — up	Area □"	Length'	r "	l/r	Allowed unit	Stress
U_1L_1	—	—	6.1	—	—	—	24.5 kips
U_1L_2	2 ∠s 3"x 2½" x 1/4	2.88	6.3	0.74	102.0	8,850#/□"	25.8
U_1L_3	2 ∠s 3"x 2½" x 1/4	2.88	6.5	0.74	105.6	8,600	15.2
U_2L_3	2 ∠s 2½"x 2"x1/4	2.12	6.7	0.59	136.9	6,900	24.9
U_2L_4	2 ∠s 2½"x 2"x1/4	2.12	6.9	0.59	140.2	6,770	4.0
U_3L_4	2 ∠s 2½"x 2"x 1/4	2.12	7.1	0.54	141.0	5,910	1.7
L_1L_2	2 ∠s 5"x 3"x 5/16	4.80	5.4	0.85	76.8	16,000	0
L_2L_3	2 ∠s 5"x 3"x 5/16	4.80	5.4	0.85	76.8	16,000	17.0
L_3L_4	2 ∠s 5"x 3"x 5/16	4.80	5.4	0.85	76.8	16,000	24.8
L_4L_5	2 ∠s 5"x 3"x 5/16	4.80	5.4	0.85	76.8	16,000	39.0
L_5L_6	2 ∠s 5"x 3"x 5/16	4.80	5.4	0.85	76.8	16,000	37.2
U_1U_2	2 ∠s 6"x 4"x 3/8	7.22	5.5	1.17	56.4	12,060	17.0
U_2U_3	2 ∠s 6"x 4"x 3/8	7.22	5.5	1.17	56.4	12,060	28.8
U_3U_4	2 ∠s 6"x 4"x 3/8	7.22	5.5	1.17	56.4	12,060	36.0
U_4U_5	2 ∠s 6"x 4"x 3/8	7.22	5.5	1.17	56.4	12,060	39.2
U_5U_6	2 ∠s 6"x 4"x 3/8	7.22	5.5	1.17	56.4	12,060	40.0
U_1L_2	2 ∠s 3⅜"x 2½"x 1/4	2.88	8.2	0.74	133.2	16,000	30.9
U_2L_3	2 ∠s 2½"x 2"x 5/16	2.62	8.4	0.59	174.0	14,000	18.1
U_2L_4	2 ∠s 2½"x 2"x 1/4	2.12	8.6	0.59	175.2	14,000	11.9
U_3L_4	2 ∠s 2½"x 2"x 1/4	2.12	8.8	0.59	178.9	14,000	5.1
U_3L_5	2 ∠s 2½"x 2"x 1/4	2.12	9.0	0.59	182.5	14,000	1.3

TRUSS 3A, 3B
C. E. THESIS M.A.C.
L. L. WEAVER. E.H. SPENCER.
SCALE 1"=6' APRIL 6, 1916



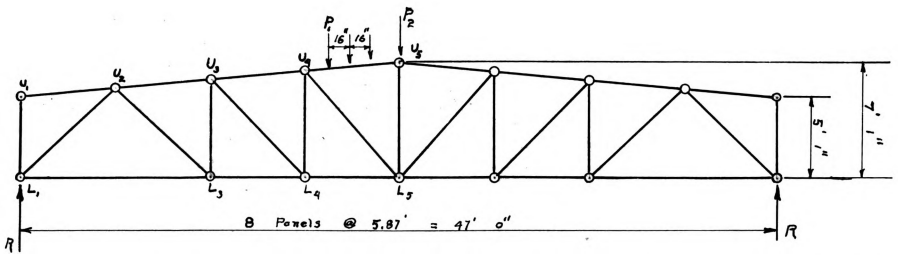




R = 21,720#
P = 1,110#
W = 5,090#

Member	Make — up	Area	Length	r	l/r	Allowed unit	Stress.
L_1L_2	2 ∠3½ x 3 x 5/16	4.80 sq in	5.4'	0.85"	76.7	16,000 #/sq in	0 Kips
L_2L_3	2 ∠3½ x 3 x 5/16	4.80	5.4	0.85	76.7	16,000	18.1
L_3L_4	2 ∠3½ x 3 x 5/16	4.80	5.4	0.85	76.7	16,000	26.5
L_4L_5	2 ∠3½ x 3 x 5/16	4.80	5.4	0.85	76.7	16,000	34.2
L_5L_6	2 ∠3½ x 3 x 5/16	4.80	5.4	0.85	76.7	16,000	38.1
U_6L_2	2 ∠3 1/2 x 2 1/2 x 1/4	2.88	8.9	0.74	144.0	16,000	24.9
U_2L_3	2 ∠3½ x 2½ x 5/16	2.62	8.9	0.58	183.6	16,000	18.9
U_3L_4	2 ∠3½ x 2½ x 1/4	2.12	8.9	0.59	181.1	16,000	12.7
U_4L_5	2 ∠3½ x 2½ x 1/4	2.12	8.9	0.59	181.1	16,000	6.5
U_5L_6	2 ∠3½ x 2½ x 1/4	2.12	8.9	0.59	181.1	16,000	0.2
U_1L_1			7.1				24.5
U_2L_2	2 ∠3 3 x 2½ x 1/4	2.88	7.1	0.74	115.0	7,950	17.7
U_3L_3	2 ∠3 3 x 2½ x 1/4	2.88	7.1	0.74	115.0	7,950	14.8
U_4L_4	2 ∠3½ x 2½ x 1/4	2.12	7.1	0.89	144.0	5,900	9.9
U_5L_5	2 ∠3½ x 2½ x 1/4	2.12	7.1	0.89	144.0	5,900	5.0
U_6L_6	2 ∠3½ x 2½ x 1/4	2.12	7.1	0.89	144.0	5,900	4.7
U_1U_2	2 ∠6 x 4 x 3/8	7.22	5.4	1.17	51.4	12,400	15.1
U_2U_3	2 ∠6 x 4 x 3/8	7.22	5.4	1.17	51.4	12,400	26.5
U_3U_4	2 ∠6 x 4 x 3/8	7.22	5.4	1.17	51.4	12,400	34.2
U_4U_5	2 ∠6 x 4 x 3/8	7.22	5.4	1.17	51.4	12,400	38.1
U_5U_6	2 ∠6 x 4 x 3/8	7.22	5.4	1.17	51.4	12,400	38.2

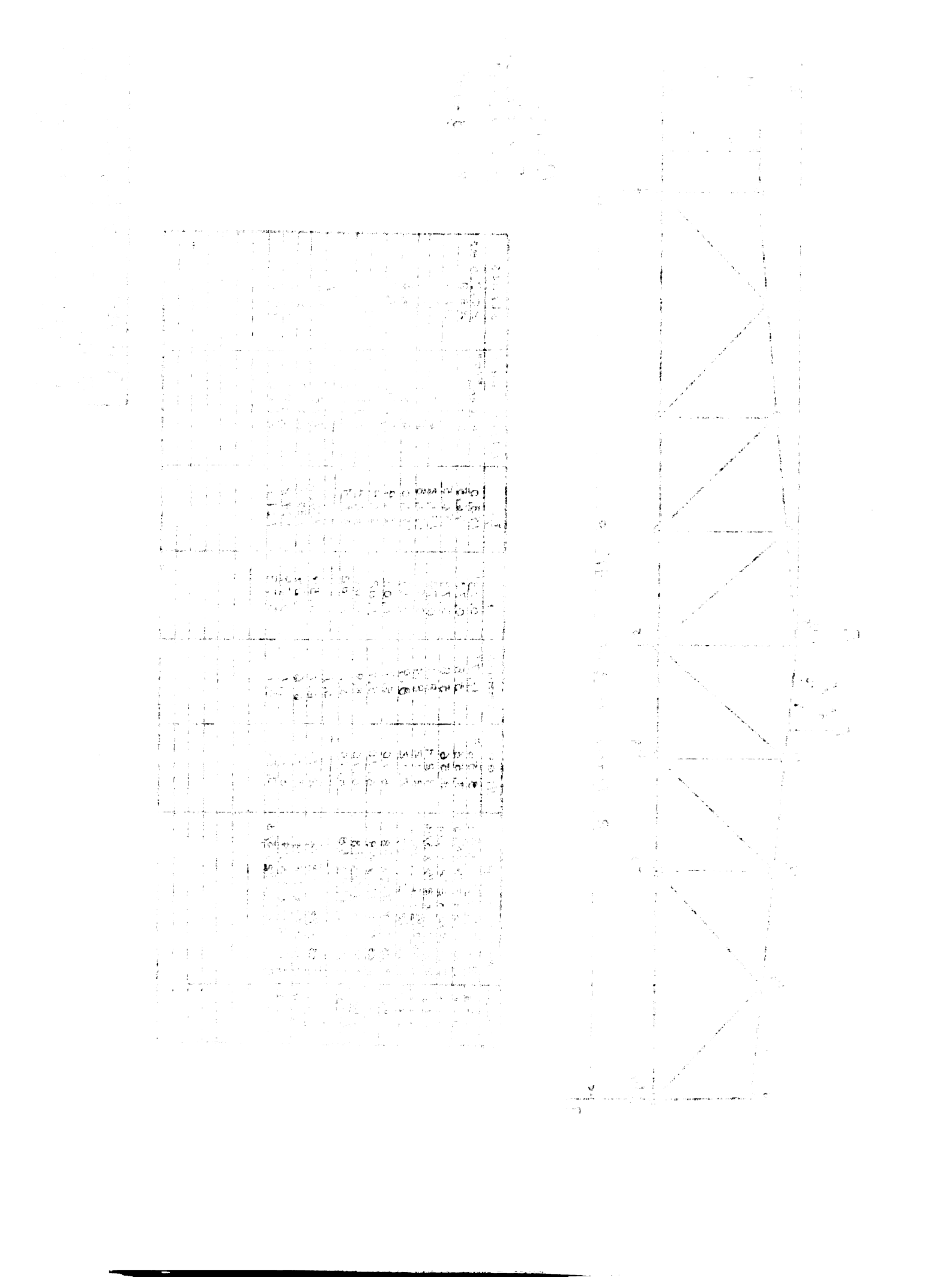
TRUSS 4
C. E. THESIS M.A.C.
L. L. WEAVER. E. H. SPENCER.
SCALE 1" = 6' APRIL 6, 1916

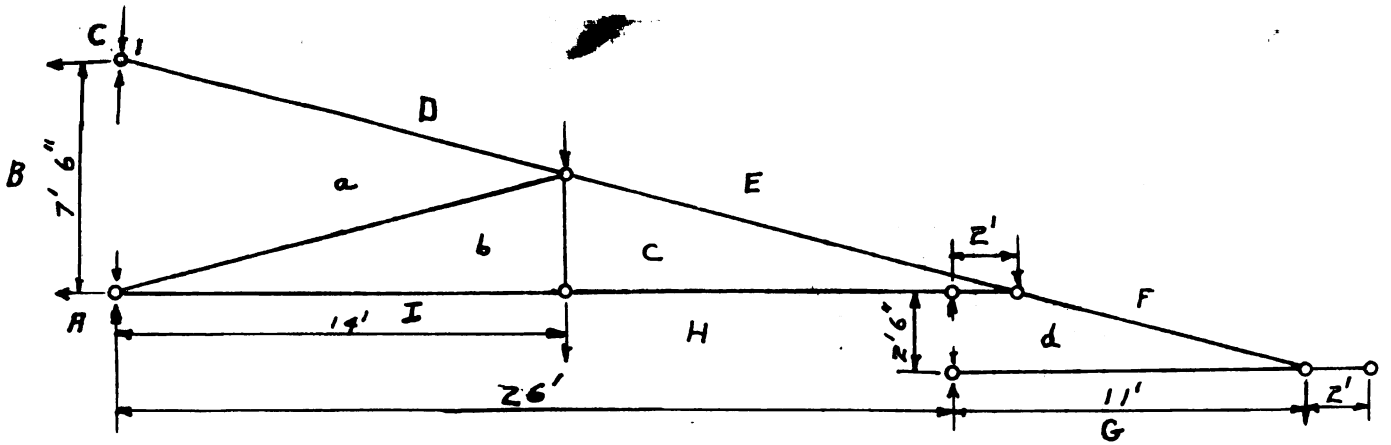


R = 14,709#
P ∝ 0" to 569#
P2 = 19,029#
W = 3,419#

Member	Make — up	Area	Length	r	l/r	Allowed unit	Stress
$L_1 L_3$	2 ∠s 3½ x 2½ x 5/16	3.56 □"	11.7'	0.73"	193.0	16,000 #/□"	20.3 K/□"
$L_3 L_4$	2 ∠s 3½ x 2½ x 5/16	3.56	5.8	0.73	96.5	16,000	23.8
$L_4 L_5$	2 ∠s 3½ x 2½ x 5/16	3.56	5.8	0.73	96.5	16,000	31.4
$U_2 L_3$	2 ∠s 3 x 2½ x 5/16	3.24	8.1	0.74	131.2	16,000	12.1
$U_3 L_4$	2 ∠s 3½ x 2 x 1/4	2.12	8.4	0.59	170.8	16,000	11.5
$U_4 L_5$	2 ∠s 2½ x 2 x 1/4	2.12	8.9	0.59	181.0	16,000	6.8
$U_1 U_2$	2 ∠s 5 x 3½ x 3/8	6.10	5.9	1.02	69.4	11,150	1.2
$U_2 U_3$	2 ∠s 5 x 3½ x 3/8	6.10	5.9	1.02	69.4	11,150	23.9
$U_3 U_4$	2 ∠s 5 x 3½ x 3/8	6.10	5.9	1.02	69.4	11,150	31.6
$U_4 U_5$	2 ∠s 5 x 3½ x 3/8	6.10	5.9	1.02	69.4	11,150	36.2
$U_1 L_1$			5.1				1.1
$U_2 L_3$	2 ∠s 2½ x 2 x 1/4	2.12	6.1	0.59	116.0	7,880	8.8
$U_4 L_4$	2 ∠s 2½ x 2 x 1/4	2.12	6.6	0.59	130.1	6,880	8.6
$U_5 L_5$	2 ∠s 2½ x 2 x 1/4	2.12	7.1	0.59	144.2	5,900	13.1
$V_2 L_1$	2 ∠s 3½ x 2½ x 5/16	3.56	8.1	0.73	131.1	6,820	19.7

TRUSS 5 A, B, C, D.
C. E. THESIS M.A.C.
L. L. WEAVER. E. H. SPENCER.
SCALE 1" = 6' APRIL 6, 1916





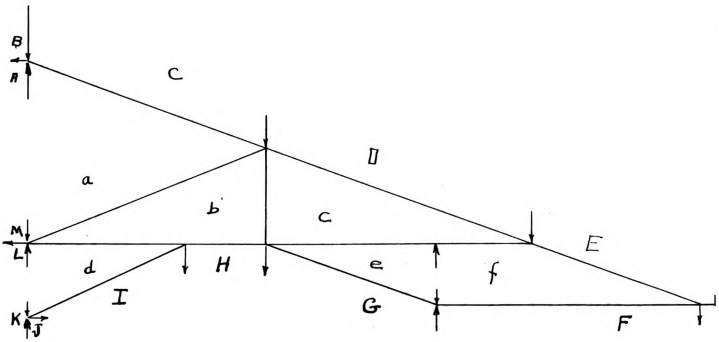
Member	a D	a E	d F	b I	c H	d G	a b	c d
Make — up	3 Pcs 1⅝ x 12"	3 Pcs 1⅝ x 12	3 Pcs 1⅝ x 12	2 Pc 1⅝ x 12	2 Pcs 1⅝ x 12	1 Pc 1⅝ x 12	3 Pcs 1⅝ x 12	2 Pc 1⅝ x 12
Area	58.5 □"	58.5	58.5	39.0	39.0	19.5	58.5	39.0
Stress	11.4 K.	14.4	7.7	12.6	12.6	7.5	7.3	12.6

BALCONY TRUSS

C. E THESIS M.A.C.

L. L. WEAVER E.H. SPENCER

SCALE 1" = 6' APRIL 5, 1916



Member	Make – up	Area "P"	Stress
aC	3 Pcs. 1⅝ x 12"	58.5	152 Kips
cD	3 Pcs. 1⅝" x 12"	58.5	16.5
fE	3 Pcs. 1⅝ x 12	58.5	7.3
ab	3 Pcs. 1⅝ x 12	58.5	19.2
bc	1 Pc. 1⅝ x 12	17.5	0.9
Hb	2 Pcs. 1⅝ x 12	39.0	16.9
ce	2 Pcs. 1⅝ x 12	39.0	15.5
cf	2 Pcs. 1⅝ x 12	39.0	18.5
dI	1 Pc. 1⅝ x 8	13.0	0.4
eG	1 Pc. 1⅝ x 8	13.0	1.5
fF	1 Pc. 1⅝ x 12	17.5	6.9

BALCONY TRUSS AA
C. E. THESIS M. A. C.
L. L. WEAVER E. H. SPENCER
SCALE 1" = 4' APRIL 8, 1916

Member (a) length 8' make up 1 5/8" x 8" wood.

35 x 8 x 13/96 x 8/12 x 40 ---------- 10,150# wt. of rafters
9 x 10 x 15---------- 1,350 " " ceiling
Total 11,500 " on rafters
P ------ 11,500/70 ---------- 164.5#rafter reaction
M ------ 8 x 164.5/4 ---------- 329'#
M ------ 329 x 12 ---------- 3,957 "#
S ------ 3957 x 4/69.4 ---------- 228 #/sq.in.

Member (b) length 6' make up 1 5/8" x 8" wood

6 x 13/96 x 8/12 x 40 ----------21.7# wt. of member
82.3 & 21.7 ---------- 104 # load
104/5.25 ---------- S------ 19.8#/sq.in.

Member (c) length 19' make up 1 5/8" x 10"

19 x 10/12 x 13/96 x 40 ---------- 86 # wt. of one rafter
10.5 x 32 x 15/24 ---------- 26.2 " " ceiling
Load on (c) from (a) ---- 82.3 applied @ 10½' from rt. end
13.5/16.5 ---------- .82
.82 x 19 ---------- 15.55- horizontal distance between supports
.82 x 9.5 -------- 7.77 " " to force 86#
.82 x 5.25 ------- 4.3' " " " " 26.2#
.82 x 10.5 ------- 8.6 " " " " 82.3#
Summation moments about rt. support equals 0
26.2 x 4.3 ---------- ---------- 112.5 '#
86.0 x 7.77 ---------- 668.5 '#
82.3 x 8.6 ---------- 709.0 '#
Total 1490.0 '#
1490/15.5 ----------Reaction ------- 96.0#
M ---- (98.0 x 7.77) - (26.2 x .8) ---------- 8700"#
S ---- 8700 x 5/135.4 ---------- 321 #/sq.in.

Member (d)

$$R = \frac{(38.4 \times 16) + (14.5 \times 82.3)}{17.5} = 103\#$$

M ---- 103 x 3 x 12 ---------- 3708"#
S ---- 3708 x 6/234 ---------- 95 #/sq.in.
Max. S ----- 95 x 515 ---------- 610 #/sq.in.

D. Girders

The weight of all I beams was assumed as the weight per foot times the span. The weights of the plate girders was found by figuring the weight of the web plate , the cover plates, flange angles and joist support angles and then adding 50% to this value. The 50% takes care of the details such as rivets and stiffner angles.

The max. bending moments were figured from the assumption that the reaction caused by the loads was a single concentrated load applied at the center of the span. This does not hold true for all cases but after several trials the authors came to the condlusion that the results by this method were as accurate as by the other methods.

The max. shear on the girders was taken as the reaction. The resisting moment was taken from S.H.B. Table 7, pg. 23 for all I beams, and the resisting shear from Table 11, pg. 28 S.H.B. The resisting moment for the plate girders was found by the methods as explained in table 87, pg.204 S.H.B. For all beams acting as lintels, the wall load was assumed as a triangle whose base and altitude equaled the span. Where the span was great the full load on the beam was taken. The authority for this action is given on pgs. 90 - 100 of the Cyclopedia of Architecture and Carpentry Building, Vol. 5.

1. Weights.

A1	55 x 18.1		1000#
A2	55 x 18.1		1000#
B	55 x 18.1		1000#
C1	1 web plate 42x5/8x47	4190#	
	2 cov. " 47x14x½	2236#	
	4 angles 6x6x7/16 x47	3240#	
	8 " 4x3x½ x 3.4	302#	
	1 " " x 47	522#	
	Total	10490#	
	Add 50% for details	5245	
	Total		15735#
C2	same as C1		15735#
D	" " C1	10490	
	less 1 angles 4x3x½ x47	522	
	Total	9968	
	Add 50% for details	4984	
	Total		14952#
E1	55 x 26.67		1473#
E2	55 x 26.67		1473#
E3	55 x 26.67		1473#
E4	55 x 26.67		1473#
E5	55 x 26.67		1473#
E6	55 x 26.67		1473#
F1	25 x 16.6		416#
F2	25 x 16.6		416#
F3	25 x 16.6		416#
F4	25 x 16.6		416#
G1	55 x 33.23		1830#
G2	55 x 33.23		1830#
H1	21 x 12.5		526#
H2	21 x 12.5		526#
I1	18 x 6.3		228#
I2	18 x 6.3		228#
K1	15 x 18.1		270#
K2	15 x 18.1		270#
L1	42 x 18.8		788#
L2	42 x 18.8		788#
L3	42 x 18.8		788#
L4	42 x 18.8		788#

M1 --------- 2x18x10.5 ---------- 380#
M2 --------- 2x18x10.5 ---------- 380#
N --------- 2x18x 7.5 ---------- 270#
O1 --------- 2x18x13.0 ---------- 468#
O2 --------- 2x18x13.0 ---------- 468#
P1 --------- 65 x 26.0 ---------- 1690#
P2 --------- 65 x 26.0 ---------- 1690#
Assembly floor ------ 31½x14x24 ---------- 10580#
" " ------ 12¼x6.6x4 ---------- 252#

2. Max. Bending Moment.

Girders A1, A2, & B
Span 18.13', balcony trusses section AA spaced 3' o.c.
Truss reaction top of girder ---------- 4.1 Kips
Girder reaction 4.1 x 6 ---------- 24.6 "
Max. Moment 24.6 x 18.13/4 ----------112.0 " ft.

Girders C1 & C2
P1 ---------- 8 kips P2 ---------- 0.3 kip
Loads equals 14x8.3 ---------- 116 Kips
Reaction --- 116/2 plus 15.72/2 ---------- 65.8 "
Moment ------- 65.8 x 47/4 ---------- 775 Kip ft.

Girder D
Masonry above girder(1.5x5) plus (1x8.5) ---------16 sq.ft.
Weight 53.49 x 16 x 120 ---------- 103500#
Load 485 x 40 ---------- 19400#
Weight of girder ---------- 14950#
Reaction (103500 & 19400 & 14950)/2 ---------- 68.9 kips
Moment 68.9 x 53.49/4 ---------- 920 ft. "

Girder E1 & E2
P1 ----------1600/2---------- 800#
P2 ------ 12.5 x 26.66 x 50.9/(20 x 2) -------- 423#
Reaction(423 & 800)/10 plus 736 ---------- 12886#
Moment --(12,886 x 13.33) - (1223 x10 plus 736) 6.66 ---
---------- 88,500'#

Girder E3 & E4
Loads 23.5 x 26.66 x 50.9 ---------- 32000#
P ---------- 32000/20 ---------- 1600#
Reaction (16000x 2)/2 plus 736 ---------- 16736#
Moment --(16736 x 13.33) - (16000 plus 736) 6.66-- 111500'#

Girder E5 & E6
P1 ---------- 800#
P2 --- 12.7x13.9x50.9/20 ---------- 450#
P3 varies from 1250# to 800#
R_1 ---(12500x7) & (1473x13.33) & (2250x17.5) & (8000x19.6) &
divided by 26.66 ---------- 11380#
R_2 ------ 25,696 - 11,380 ----------13580#

Moment -----(13580 x 13.33) - (12500 x 6.33) - (736 x 6.66)
---------- 96,900'#

Girders F1, F2, F3, & F4
P2 --------- 16.61 x 16.61 x 49.1/24 ---------- 564#
P1 varies from 0 to 564#
R_2 ---- (564x6x8.4) & (282x6x5.5) & (416x8.3)/16.61 -- 2468#
R_1 ---------- 3024#
Moment --- (3084x8.4) - (3384 & 208) x 4.2 ---------- 10300'#

Girders G1 & G2
P2 ---------- 564 x 2 ---------- 1128#
P1 varies from 0 to 1128#
R_2 -- (1128x13x8.4) & (1128x6x22.1) & 915/33.2------ 9115#
R_1 ---------- 14175#
Moment (14175 x 16) - (1128x12x7.6) - (915x8.3) -- 109,000'#

Girders H1 & H2 act as lintels.
Span 12.5' 12" brickwall 9.5' high supported by girders
Weight of wall 9.5 x 12.5 x 1 x 120 ---------- 14,250#
Reaction 14250/2 & 263 x 2/2 ----------7,388#
Max, moment 7388 x 12.5/4 ---------- 23,100'#

Girders I1 & I2 acts as lintels.
Span 6' 4" 12" brickwall 7' 8" high supported
Weight of brick 7.66 x 1 x 6.33 x 120 ---------- 5,820#
Reaction 5820/2 & 114 x 2/2 ---------- 3,024#
Moment ---- 3024 x 6.33/4 ---------- 4,760'#

Girders K1 & K2
Weight of fan ---------- 1,500#
" " motor ---------- 400#
R_2 ---- (4 x 400) & (750 x 12)/ 18 &- 590 & 135 --- 725#
R_1 ---- 1150& 135 - 590 ---------- 695#
Max. moment occurs at 695y -- 725(18 - y)
y ---------- ---- 9.2'
Max. moment ------695 x9.2 - (400 x 5.2) ------- 4,320'#

Girders L1, L2, L3, & L4.
P ---------- ---------- 1947#
Reaction (1947 x 14) & 788/2 ----------14,000#
Moment 14,000 x 18.79/4 ---------- 65,800'#

Girders M1 & M2
Span 10.55' supporting 8" brickwall 29' high.
Weight of wall 10.55 x .666 x 29 x 120 --------- 23,200#
Reaction 190 & 23200/2 ---------- 11,790#
Moment 11790 x 10.55/4 ---------- 31,100'#

Girder N
Span 7.5' supports 8" brickwall 29' high
Weight of wall 7.5 x .666 x 29 x 120 ---------- 16,500#
Reaction 135 & 16,500/2 ---------- 8,385#
Moment --8,385 x 7.5/4 ---------- 15,700'#

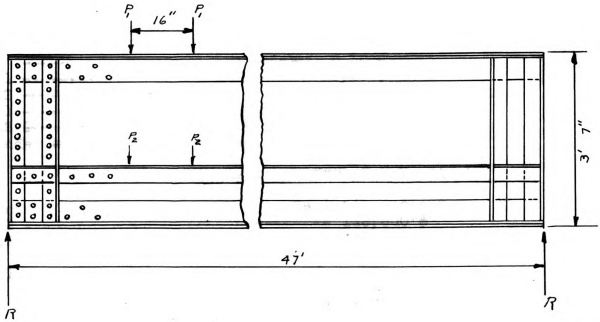
Girder 01 & 02
Span 13' supporting 12" brickwall 7.66' high
Weight of wall 7.66 x 1 x 13 x 120 ---------- 12,000#
Reaction --- 234 & 12,000/2 ---------- 6,234#
Moment ---------- 6,234 x 13/4 ---------- 20,200'#

Girders P1 & P2
Span 26' supporting rafters 12" o.c.
Loads 1104 & (105.41 & 54) x 13.5/2 ---------- 2,170#
Reaction 2170 x 26/2 ---------- 28,200#
Moment ---------- 28,200 x 26/4 ---------- 183,000'#
This exceeds the allowed value by nearly 30,000'# therefore the assumption as to the live load on the stage floor must be wrong.

Girders under main floor
P ---------- 955 x 2 ---------- 1910#
14' span of girders 16" spacing of rafters
14 x 12/16 ---------- 10.5 rafters supported
1910 x 10.5/2 ---------- 10,000 & 220 ---- 10220 -- Reaction
Moment 10220 x 14/4 ---------- 36,000'#

3 Max. shears taken as greatest reaction

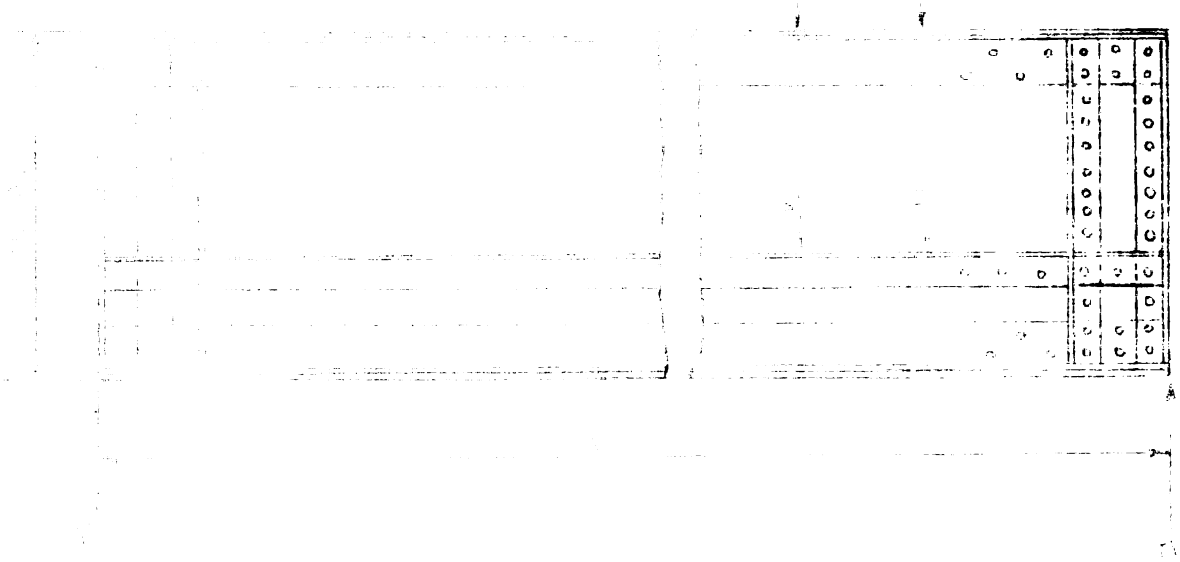
Member	Max. Shear
A1	24,600#
A2	24,600#
B	24,600#
C1	65,800#
C2	65,800#
D	68,900#
E1	12,886#
E2	12,886#
E3	16,736#
E4	16,736#
E5	13,580#
E6	13,580#
F1	3,024#
F2	3,024#
F3	3,024#
F4	3,024#
G1	14,175#
G2	14,175#
H1	7,388#
H2	7,388#
I1	3,024#
I2	3,024#
K1	695#
K2	695#
L1	14,000#
L2	14,000#
L3	14,000#
L4	14,000#

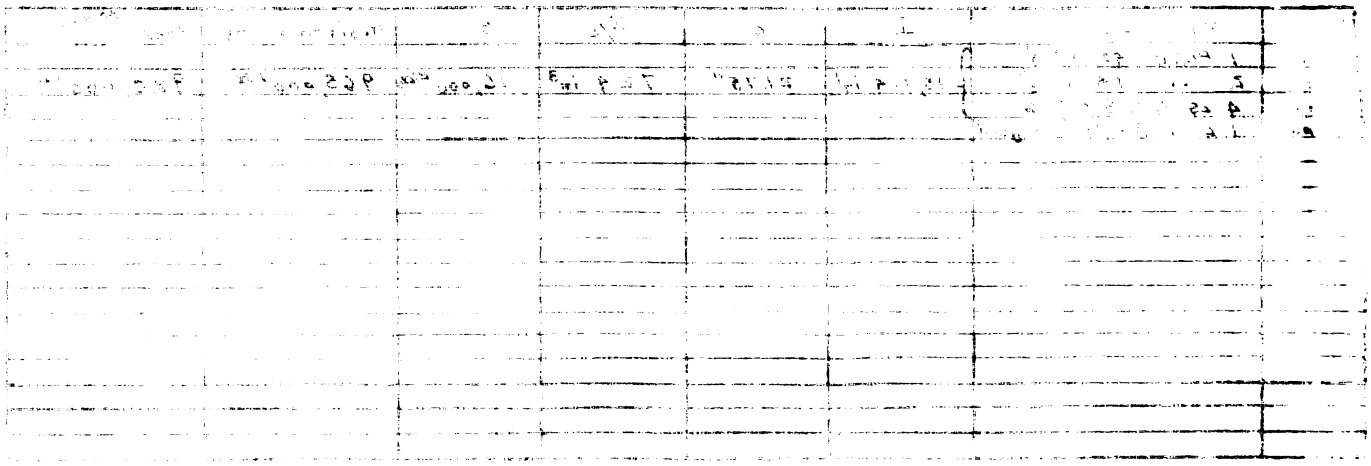


$R = 68,900^{\#}$
$P = 8,000^{\#}$
$P_2 = 300^{\#}$

Member	Make — up	I	c	I/c	S	Resisting M	Max M Bending
Web	1 Plate 42" x 5/8"						
Cover	2 " 18" x 1/2"	13,719 in⁴	21.75"	729 in³	16,000 #/in²	965,000 in#	920,000 in#
Flange	4 ∠s 6 x 6 x 7/16						
Support	1 ∠ 9 x 3 x 1/2 for joist						

PLATE GIRDER
C. E. THESIS M. A. C
L. L. WEAVER E. H. SPENCER
SCALE 1" = 2' APRIL 7, 1916





Member	Max. shear
M1	11,790#
M2	11,790#
N	8,385#
O1	6,234#
O2	6,234#
P1	28,200#
P2	28,200#
Under main floor	10,220#

4. Resisting moments as taken from S.H.B. pg. 23

Member	Resisting moment
A1	118,000'#
A2	118,000'#
B	118,000'#

C1

back to back of angles 42.5" ----- d
dist. to extreme fibre 21.75# ---- c

Moment of inertia 4 angles 6x6x7/16	-------	7838
" " " 1 web pl. 42x5/8	------	3858.8
" " " 2 cover pl. 14x½	--------	6472.2
Total I		18169

Moment of inertis of rivet holes (¾ rivets, 7/8 holes)

location	number	size	area	dis.to N.A. of hole	I
web	2	1½x7/8	2.63	19.25	975
flange	4	15/16x7/8	3.28	21.25	1480
				Total	2455

I = 18,169 - 2,455 ---------- 15,714 in^4
I/c = 15,714/21.75 ---------- 724 in^3
S ------- 16,000#/ sq/in.
M ------- 16,000 x 724/12 ---------- 965,000'#

C2	965,000'#
D same as C1	965,000'#
E1	118,000'#
E2	118,000'#
E3	118,000'#
E4	118,000'#
E5	118,000'#
E6	118,000'#
F1	33,000'#
F2	33,000'#
F3	33,000'#
F4	33,000'#
G1	118,000'#
G2	118,000'#
H1	50,000'#
H2	50,000'#
I1	38,000'#
I2	38,000'#

Member	Resisting moment
K1	18,000'#
K2	18,000'#
L1	79,000'#
L2	79,000'#
L3	79,000'#
L4	79,000'#
M1	38,000'#
M2	38,000'#
N	38,000'#
O1	38,000'#
O2	38,000'#
P1	156,000'#
P2	156,000'#
Under main floor	48,000'#

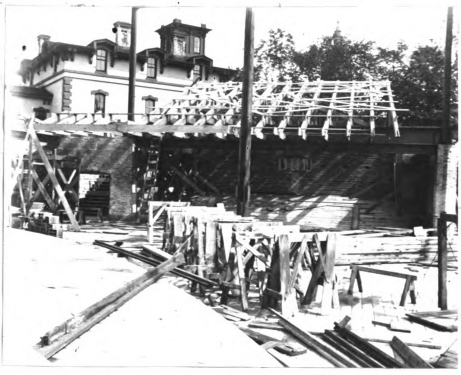
5 Resisting Shear.(see pg. 28, table 11, S.H.B.)

Member	Resisting shear
A1	82,800#
A2	82,800#
B	82,800#
C1	
Area of web--- (42.5 - 6)5/8	22.8sq.in.
Shear----- 22.8 x 10,000	228,000#
C2	228,000#
D	228,000#
E1	82,800#
E2	82,800#
E3	82,800#
E4	82,800#
E5	82,800#
E6	82,800#
F1	31,000#
F2	31,000#
F3	31,000#
F4	31,000#
G1	82,800#
G2	82,800#
H1	26,100#
H2	26,100#
I1	21,600#
I2	21,600#
K1	24,000#
K2	24,000#
L1	61,500#
L2	61,500#
L3	61,500#
L4	61,500#
M1	21,600#
M2	21,600#
N	21,600#
O1	21,600#

Member	Resisting shear
O2	21,600#
P1	100,000#
P2	100,000#
Under main floor	42,000#

Comparison table for girders

Member	Depth	wt/ft	span	wt.	Bend.M	Res.M	Shear	Res. S
A1	18"	55#	18.1'	1000#	112.0	118.0	24.6	82.8
A2	18	55	18,1	1000	112.0	118.0	24.6	82.8
B	18	55	18.1	1000	112.0	118.0	24.6	82.8
C1	43		47.0	15.7K	775.0	965.0	65.8	228.0
C2	43		47	15.7	775.0	965.0	65.8	228.0
D	43		54.4	14.9	920.0	965.0	68.9	228.0
E1	18	55	26.6	1.473	88.5	118.0	12.9	82.8
E2	18	55	26.6	1.473	88.5	118.0	12.9	82.8
E3	18	55	26.6	1.473	111.5	118.0	16.7	82.8
E4	18	55	26.6	1.473	111.5	118.0	16.7	82.8
E5	18	55	26.6	1.473	96.9	118.0	13.5	82.8
E6	18	55	26.6	1.473	96.9	118.0	13.5	82.8
F1	10	25	16.6	.416	10.3	33.0	3.0	31.0
F2	10	25	16.6	.416	10.3	33.0	3.0	31.0
F3	10	25	16.6	.416	10.3	33.0	3.0	31.0
F4	10	25	16.6	.416	10.3	33.0	3.0	31.0
G1	18	55	33.2	1.83	109.0	118.0	14.2	82.8
G2	18	55	33.2	1.83	109.0	118.0	14.2	82.8
H1	2- 9	21	12.5	.263	23.1	25.0	7.4	26.1
H2	2- 9	21	12.5	.263	23.1	25.0	7.4	26.1
I1	2- 8	18	6.3	.114	4.7	19.0	3.0	21.6
I2	2- 8	18	6.3	.114	4.7	19.0	3.0	21.6
K1	10	15	18.0	.27	4.3	18.0	.7	24.0
K2	10	15	18.0	.27	4.3	18.0	.7	24.0
L1	15	42	18.8	.788	65.8	79.0	14.0	61.5
L2	15	42	18.8	.788	65.8	79.0	14.0	61.5
L3	15	42	18.8	.788	65.8	79.0	14.0	61.5
L4	15	42	18.8	.788	65.8	79.0	14.0	61.5
M1	2- 8	18	10.5	.19	31.1	38.0	11.8	21.6
M2	2- 8	18	10.5	.19	31.1	38.0	11.8	21.6
N	2- 8	18	7.5	.135	15.7	38.0	8.4	21.6
O1	2- 8	18	13.0	.234	20.2	38.0	6.2	21.6
O2	2- 8	18	13.0	.234	20.2	38.0	6.2	21.6
P1	20	65	26.0	1.69	185.0	156.0	28.2	100.0
Under main floor	24- 12	31.5	14.0	.441	36.0	48.0	10.2	42.0



North Balcony during construction

South Balcony during construction

E. Columns

The weight of the Bethlehem H columns was figured by multiplying the weight per foot by the length. The weight of the cast iron columns was figured by first finding the area of the metal and then multiplying this value by 1 foot and then by the weight of cast iron per cu. ft., this gave the weight per lineal foot. This last value was multiplied by the length thus giving the weight of the columns.
In analyzing the stresses in the columns there was two classes of loading to be considered. Some of the columns were just centrally loaded, hence the stress was computed by the column formula given in the Cyclopedia of Architecture and Carpentry Building, other columns had loads applied centrally at the top with eccentric loads applied on the sides. The latter loads not only increased the compression stress, as diid the centrally applied loads, but also produced a stress in the column that resisted the the moment produced by the eccentrically applied loads. From the formula, $S = M\ c/I$ the fibre stress due to the moment was determined and added to the stress produced by the direct compression of the loads. In the case where two eccentric loads were placed on opposite sides of the column, their respective moments are of opposite sign and the fibre stress developed depends on the resultant moment.

1. Weight

Column	Wt./ft/	Bethlehem height	weight
1	71#	43.23'	3.07 Kips
2	71	43.23	3.07
3	54	43.27	2.34
4	54	43.27	2.34
5	82.5	40.20	3.42
6	82.5	40.20	3.42

Cast iron

Area of column. .785(36 - 20.2) ---------- 12.3 sq. in.
Weight per foot 12.3 x 12 x 450/1728 ------- 38.4#/lin.ft.

7	38.4	14.87	.57
8	38.4	14.87	.57
9	38.4	9.5	.36
10	38.4	9.5	.36

Total weight 19.528 Kips.

The following formulas as given in Vol. 8 of the Cyclopedia Of Architecture and Carpentry Building pgs. 87 - 105 were used in the calculation of the stresses in the columns.

$S = P/A\ (1 \text{ plus } k\ L^2/r^2)$

Where P is the ultimate load, S the unit stress, L the length and r the least radius of gyration. k has the following values 1/18,000 for columns with two pin ends
1/24,000 " " " one flat and one pin end
1/36,000 " " " two flat ends.

For cast iron columns- hollow - with comparative thin walls The following a modification of Rankine's formula is often used

$P/A = 80,000/(1 \text{ plus } L^2/800d^2)$

Where P is the load L the length and d the outside diameter.

2. Stresses

Columns 1 & 2.

L ---------- 43' $2\frac{3}{4}$" ---------- 528.75"
L^2 ---------- 280,000
r ---------- 2.56"
r^2 ---------- 6.55
I/c ---------- 27.0
Area ---------- 20.91 sq. in.
k ---------- flat ends ----------1/36,000
Direct central applied loads

From truss 2 ----------	26,135#
" " 5D ----------	14,709#
" " 1B ----------	55,541#
" girder E6 ----------	12,116#
Total	108,501#

Eccentric loads

From girder C1 ---- ----------	65,800#
" " A1 ----------	24,600#
Total direct compression loads	198,901#

S ---- $\frac{198,901(1 \text{ plus } 280,000/(36,000 \times 6.55))}{20.91}$ = 2,525#/sq. in.

Resultant moment set up in column due to eccentric loads
(65,000 - 24,600) x 5 3/16 ---------- 213,740"#
S from eccentric loads --- 213,740/27 ------ 7,930
Total stress ---- 7,930 plus 2,525 -------- 10,455#/sq.in.

Columns 3 & 4
Top load (12,886 & 9,115 & 15,075) -------- 37,076 #
Eccentric load ---------- 65,300#
Eccentricity of load - ---------- 5"
Length of column 43' $3\frac{1}{4}$" ---------- 499.25"
Make up 10" 54# Bethelhem H column
Area ---------- 15.91sq.in.
least r ---------- 2.51"
S --- $\frac{37,076(1 \text{ plus}(1/36000(499.25/2.51)^2))}{15.91}$ = 5,010#/sq.in.
I/c ---------- 20.1
M ---------- 65,300 x 5 ---------- 326,500"#
S ---- 326,560/20.1 ---------- 16,100#/sq.in.
Total S ------ 16,100 & 5,010 ---------- 21,110#/sq.in.

Columns 5 & 6
Loads ---------- 68,900#
Length of column --------- 40' $2\frac{1}{2}$" --------- 482.5"
Make up ------ 10" 82.5# Bethlehem H column
Least r ---------- 2.58
Area ---------- 24.29 sq.in

$$S = \frac{68{,}900(1 + 1/36000(482.5/2.58)^2)}{24.29}$$

S ---------- 5,600#/sq.in

Columns 7 & 8
Load 224,000#
Length of column ---------- 14' 10½" -- 178.5
Make up ---------- Cast iron 6" dia. ¾" metal.
Area ---------- 12.3 sq.in.
P ------- 12.3 x 80,000/ (1 plus 178.5^2/800x36) -- 474,000#
474,000/224,000 ------- 2.1 factor of safety

Columns 9 & 10
Load ---------- 28,000#
Length of column ------- 9' 6" ---------- 114"
Make up ---------- Cast iron 6" dia. ¾" metal
Area ---------- 12.3 sq.in.
P ----- 12.3 x 80,000/(1 plus 114^2/800x36) ----- 680,000#
680,000/28,000 ---------- 24.6 factor of safety .

F. Column bases and bearing plates.

The weights of the bases and bearing plates was figures from table 2 pg. 12 S.H.B. The resisting stress was not figured out directly but the required thickness for the plate was found and compared with the thickness used. For the above calculation the following formula, taken from the Cyclopedia of Architecture and Carpentry Building vol. 5, pg. 96, was used.

$$t = .866(L - b)(R/b'Lf)^{\frac{1}{2}}$$

where t is the required thickness
L the length of the plate perpendicular to the beam.
b' the width of the plate parallel to the beam
b the width of the beam resting on the plate.
R the end reaction of the beam.
f is a constant 16,000 for steel and 2,500 for cast iron

1. Weight of bearing plates.

Main floor	32x1.33x34	1,540#
Girders I & O	8x1x17	136#
Girders L	4x1.33x40.8	218#
Girders P	4x1.33x54.4	291#
Girders M & N	6x1x17	102#
Girders E	6x1x40.8	245#
Girders K	2x1x17	34#
Girders H	4x1.33x17.	88#
Columnss 7&8	2x1.3x76.5	198#
Columns 9 & 10	2x1,2x59.5	143#
Columns 1 & 2	2x2.33x58	270#
Columns 3 & 4	2x1.83x58	212#
Columns 5 & 6	2x2x79	316#
Total		3,793#

$$S = \frac{68{,}900(1 \text{ plus } 1/36000(482.5/2.58)^2)}{24.29}$$

S ---------- 5,600#/sq.in

Columns 7 & 8
Load 224,000#
Length of column ---------- 14' 10½" -- 178.5
Make up ---------- Cast iron 6" dia. ¾" metal.
Area ---------- 12.3 sq.in.
P ------- 12.3 x 80,000/ (1 plus $178.5^2/800x36$) -- 474,000#
474,000/224,000 ------- 2.1 factor of safety

Columns 9 & 10
Load ---------- 28,000#
Length of column ------- 9' 6" ---------- 114"
Make up ---------- Cast iron 6" dia. ¾" metal
Area ---------- 12.3 sq.in.
P ----- 12.3 x 80,000/(1 plus $114^2/800x36$) ----- 680,000#
680,000/28,000 ---------- 24.6 factor of safety .

F. Column bases and bearing plates.

The weights of the bases and bearing plates was figures from table 2 pg. 12 S.H.B. The resisting stress was not figured out directly but the required thickness for the plate was found and compared with the thickness used. For the above calculation the following formula, taken from the Cyclopedia of Architecture and Carpentry Building vol. 5, pg. 96, was used.

$$t = .866(L - b)(R/b'Lf)^{\frac{1}{2}}$$

where t is the required thickness
L the length of the plate perpendicular to the beam.
b'the width of the plate parallel to the beam
b the width of the beam resting on the plate.
R the end reaction of the beam.
f is a constant 16,000 for steel and 2,500 for cast iron

1. Weight of bearing plates.

Main floor ----------	32x1.33x34 ----------	1,540#
Girders I & O ----------	8x1x17 ----------	136#
Girders L ----------	4x1.33x40.8 ----------	218#
Girders P ----------	4x1.33x54.4 ----------	291#
Girders M & N ----------	6x1x17 ----------	102#
Girders E ----------	6x1x40.8 ----------	245#
Girders K ----------	2x1x17 ----------	34#
Girders H ----------	4x1.33x17. ----------	88#
Columnss 7&8 ----------	2x1.3x76.5 ----------	198#
Columns 9 & 10 ----------	2x1,2x59.5 ----------	143#
Columns 1 & 2 ----------	2x2.33x58 ----------	270#
Columns 3 & 4 ----------	2x1.83x58 ----------	212#
Columns 5 & 6 ----------	2x2x79 ----------	316#
Total		3,793#

$$S = \frac{68{,}900(1 \text{ plus } 1/36000(482.5/2.58)^2)}{24.29}$$

S ---------- 5,600#/sq.in

Columns 7 & 8
Load 224,000#
Length of column ---------- 14' 10½" -- 178.5
Make up ---------- Cast iron 6" dia. ¾" metal.
Area ---------- 12.3 sq.in.
P ------- 12.3 x 80,000/ (1 plus $178.5^2/800x36$) -- 474,000#
474,000/224,000 ------- 2.1 factor of safety

Columns 9 & 10
Load ---------- 28,000#
Length of column ------- 9' 6" ---------- 114"
Make up ---------- Cast iron 6" dia. ¾" metal
Area ---------- 12.3 sq.in.
P ----- 12.3 x 80,000/(1 plus $114^2/800x36$) ----- 680,000#
680,000/28,000 ---------- 24.6 factor of safety .

F. Column bases and bearing plates.

The weights of the bases and bearing plates was figures from table 2 pg. 12 S.H.B. The resisting stress was not figured out directly but the required thickness for the plate was found and compared with the thickness used. For the above calculation the following formula, taken from the Cyclopedia of Architecture and Carpentry Building vol. 5, pg. 96, was used.

$$t = .866(L - b)(R/b'Lf)^{\frac{1}{2}}$$

where t is the required thickness
L the length of the plate perpendicular to the beam.
b'the width of the plate parallel to the beam
b the width of the beam resting on the plate.
R the end reaction of the beam.
f is a constant 16,000 for steel and 2,500 for cast iron

1. Weight of bearing plates.

Main floor ----------	32x1.33x34 ----------	1,540#
Girders I & O ----------	8x1x17 ----------	136#
Girders L ----------	4x1.33x40.8 ----------	218#
Girders P ----------	4x1.33x54.4 ----------	291#
Girders M & N ----------	6x1x17 ----------	102#
Girders E ----------	6x1x40.8 ----------	245#
Girders K ----------	2x1x17 ----------	34#
Girders H ----------	4x1.33x17. ----------	88#
Columnss 7&8 ----------	2x1.3x76.5 ----------	198#
Columns 9 & 10 ----------	2x1,2x59.5 ----------	143#
Columns 1 & 2 ----------	2x2.33x58 ----------	270#
Columns 3 & 4 ----------	2x1.83x58 ----------	212#
Columns 5 & 6 ----------	2x2x79 ----------	316#
Total		3,793#

2 Required and actual thicknesses of plates.

Main floor
Make up ---- 32--- 16"x16"x5/8" plates
Reaction ---------- 10,220x2 -- 20440#
b ----------5"
b' ---------- 16"
L ---------- 16"
f ---------- 16,000
t ---- .866(16 -5)(20440/16x16x16000)$^{\frac{1}{2}}$ ---------- .670
Required thickness is .670"
Actual thickness is .625" plate should be .035" thicker.

Girders I1 & I2
Make up ------- 8 --- 8x12x5/8 plates
Reaction ---------- 3,024#
b ---------- 8.5"
b' ---------- 8"
L ---------- 12"
f ---------- 16,000
t--- .866(12 - 8.5)(3,024/8x12x16000)$^{\frac{1}{2}}$ ---------- .133"
Required thickness is .133"
Actual thickness is .625" plate is .492" thicker than necessary

Girder O1 & O2
Make up ---------- same as for I1 & I2 above
Reaction ---------- 6,234#
t ---- .866(12 - 8.5)(6,234/8x12x16000)$^{\frac{1}{2}}$ ---------- .193"
Required thickness is .193"
Actual " " .625 or an excess of .432"

Girders L1, L2, L3, & L4.
Make up ----- 4 plates 16"x16"x$\frac{3}{4}$"
Reaction ---------- 14,000#
b ---------- 5$\frac{1}{2}$"
b' ---------- 16"
L ---------- 16"
f ---------- 16,000
t --- .866(13 - 5.5)(14000/16x16x16000)$^{\frac{1}{2}}$--------- .533"
Required thickness .533"
Actual " .750 or an excess of .217"

Girders P1 & P2
Make up ---------- 4 plates 16x16x1
Reaction---------- 28,200#
b ---------- 6.25"
b' ---------- 16"
L ---------- 16"
f ---------- 16,000
t --- .866(16 - 6.25)(28200/16x16x16000)$^{\frac{1}{2}}$--------- .7"
Required thickness .7"
Actual " 1.0 or an excess of.3"

Girders M1 & M2
Make up ---------- 4 plates 8"x12"x5/8"
Reaction ---------- 11,790#
b ---------- 8.5"
b' ---------- 12.0"
L ---------- 8.0"
f ---------- 16,000
t --- .866(12 - 8.5)(11790/8x12x16000)$^{\frac{1}{2}}$ ---------- .266"
Required thickness .266"
Actual " .625 or an excess of .359

Girders N
Make up ---------- 2 plates 8"x12"x5/8"
Reaction ---------- 8385#
b, b' ,L, & f. same as above
t ---------- .866(12 - 8.5)(8385/8x12x16000) -- .224"
Required thickness .224"
Actual " .625 or an excess of .401"

Girders K1 & K2
Make up ---------- 8"x12"x5/8"
b ---------- 2.6"
b' ---------- 12"
L ---------- 8"
f ---------- 16000
t --- .866(12 - 2.6)(695/8x12x16000)$^{\frac{1}{2}}$ ---------- .172"
Required thickness .172
Actual " .625 or an excess of .453"

Girders E5 & E6
Make up ---------- 12"x16"x$\frac{3}{4}$" plate
Reaction ---------- 13,580#
b ---------- 6"
b' ---------- 12"
L ---------- 16"
f ---------- 16,000
t --- .866(16 - 6)(13580/12x16x16000) ---------- .575"
Required thickness .575"
Actual " .750 or an excess of.175"

Girders E4 & E3
Make up ---------- 12x16x$\frac{3}{4}$ plate
Reaction ---------- 16,736#
b, b', L, & f are the same as above.
t --- .866(16 - 6)(16736/12x16x16000) ---------- .640"
Required thickness .640"
Actual " .750" or an excess of .110"

Girders E1 & E2
Make up ---------- 12x16x$\frac{3}{4}$ plate
Reaction ---------- 12,886
b, b', L, & f are the same as above
t --- .866(16 - 6)(12886/12x16x16000)$^{\frac{1}{2}}$ ---------- .562"
Required thickness .562"
Actual " .750" or an excess of .188"

Girders H1 & H2

Make up	2 plates 8"x16"x5/8"
Reaction	7,388#
b	9¼"
b'	16"
L	8"
f	18,000
t --- .866(16 - 9.25)(7388/16x8x16000)$^{\frac{1}{2}}$	.350"

Required thickness .350"
Actual " .625 or an excess of .275"

G. Floors(floor beams and covering)

The weight per sq. ft. of floor surface was found and then multiplied by the no. of sq. ft. of floor, thus giving the weight of the floor covering, to this value was added the weights of the joists.
The bending moment for the floor covering was taken as $Wl^2/10$ being the same formula as was used for the roof sheathing.
The resisting-stress was figured from the relation $S = M\ c/I$
The resisting stress was taken as 1300#/sq.in. with an increase of 50% for unexposed building material

1. Weight

Weight per sq.ft.

1" hemlock	3.08#/sq.ft.
1" maple	3.32#/sq.ft.
Total	5.4 #/sq.ft.

Balcony section BB.

Area of floor	4887 sq.ft.
Weight 4887x5.4	26,400#

Balcony section AA.

Area of floor	980sq.ft.
Weight 980x5.4	5,300#

Main floor

Area of floor	10,687sq.ft.
Weight 10687x5.4	57,800#

Joists

Number	length	Total
84	13'	1090'
45	6'	270'
84	13'	1090'
64	13'	834'
42	13'	546'
80	10'	800'
56	30'	1680'
40	19'	760'
	Agregate	7170'

Weight 7170 x(1.625/12)x 40	389,000#
Total weight of floor	478,000#

Test of flooring
Loads
Live ---------- 100#/sq.ft.
Dead ---------- 5.4#/sq.ft.
Use 106#/sq.ft. as the load
M --- $Wl^2/10$ ---------- 106x9/10 ---------- 95.4'#
S ----- 95.4 x 12 x 6/(12 x 4) ---------- 143#/sq. in.
Shear 106 x 3 ---------- 418#
The floor is safe for the resisting stress is 1950#/sq.in.

Test of joists
1st. floor under balcony, section BB.
Load 106#/sq.ft.
Load on rafter 106x(16/12)x13.25 ---------- 1,875#
Weight of " (1.625/12) x 1x13.25x40 --- 72#
Total 1,947#
Reaction --- 1,947/2 ----------974#
M --- 974x13/4 ---------- 3,170'#
S --- 3170x12x(6/(1.625x144)) ---------- 975#/sq.in.
Allowed unit is 1950#/sq.in. therefore joist is safe.

Joist under main floor
Loads on floor 106#
Weight of joist --(1.625/12)x1x40x13 ---------- 70.5#
" " floor on joist 106(16/12)13 ---------1,840#
Reaction 70.5 & 1,840 divided by 2 ---------- 955#
M --- 955x13/4 ---------- 3,100'#
S --- 3100x6x12/(1.625x144) ---------- 955#/sq.in
Allowed unit is 1950#/sq.in. therefore floor is safe.

Joist under stage floor.
Load on joist 106#/sq.ft.
Weight of joist --(1.625/12) x1x40x20 ---------- 108#
Weight of floor on joist 106 x 20 ---------- 2120#
Reaction 2120 & 108 divided by 2 ---------- 1114#
M --- 1114x20/4 ---------- 5,570'#
S --- 5,570 x 6 x 12/(1.625x144) ---------- 1,700#/sq.in,
Therefor the joist is safe since the allowed unit is 1950#/sq in

H. Foundations (including walls and partitions)

The volumns of the walls was figured both for the brick and for the concrete. The volumns was multiplied by 120 for the brick walls which gave the weight of the brick walls. For the concrete the volumns was multiplied by 150 and this result was the weight of the concrete walls. The sum of the two results was taken as the weight of the foundations, walls and partitions. The bearing stress on the walls was figured by dividing the area of the plates by the load on the plate The resisting stress was taken from S.H.B. pg.56.
The earth bearing was found by dividing the weight of the structure and fixtures by the sq. ft. of foundation resting on the earth. This value was compared with the value in S.H.B.pg56

Entrance during construction,
showing forms for door-ways, also the steel roof frames in the background.

L. Weight

Weight of brick walls and partitions.

Under front part of stage --- 4x1x130x120 -------	62,400#
Outside walls ----------120x 44x17½12 x192 -----1,	434,000#
44x1x104x120 ----------	550,000#
54x1xx91x120----------	381,000#
Partitions 18x1x150x120 ----------	324,000#
44x1x82x120 ----------	433,000#
18x.66 x32x120 ----------	46,100#
18x6x16x120 ----------	208,000#
40x.66x156x120 ----------	500,000#
Wall over girder D 53.5x1x16x120 ----------	103,500#
Total weight of brick masonry	4,032,000#
or 2,016 tons.	

Weight of concrete walls and foundations.

2(2x48.66x11.5)150 ----------	335,000#
(20/12)11.5x274.5x150 ----------	788,000#
2(39x11.5x2)150 ----------	270,000#
2(4x11.5x1.5)150 ----------	20,700#
(11.5x.66x361.6)x150 ----------	415,000#
(11.5x1x261)150 ----------	45,000#
(11.5x2.83x91)150 ----------	444,000#
24(1.33x1.33x11.5)150 ----------	73,000#
Total weight of concrete	2391,100#
or 1,195,55 tons.	

2. Bearing stress

Under columns 1 & 2.on concrete

Load ----------	198,901#
Area ---------- 30x24 -------	720 sq.in.
Pressure on masonry 198,901/720 ----------	276#/sq.in.

Allowed value as given in S.H.B. pg 56 is 280#/sq.in.

Under columns 3 & 4 on concrete

Load ----------	102,376#
Area ---------- 22x22 -------	484sq.in.
Pressure on masonry 102,376/484 ----------	211.5#/sq.in

Allowed value 280#/sq.in.

Under columns 5 & 6 on concrete

Load ----------	68,900#
Area ---------- 22x28 ------	616 sq.in.
Pressure on masonry -- 68,900/616 ----------	112#/sq.in

Allowed value 280#/sq.in.

Under columns 7 & 8 on concrete

Load ----------	224,000#
Area ---------- 15x15 -------	225sq.in.
Pressure on masonry -- 224,000/225 ----------	99.5#/sq.in.

Allowed value 280#/sq.in.

Undre columns 9 & 10 On concrete
Load ---------- 28,000#
Area ---------- 14x14 ----- 196sq.in.
Pressure on masonry 28,000/196 ---------- 145#/sq.in.
Allowed value 280#/sq.in.

Under girders E5 & E6 on brick
Load ---------- 13,580#
Area ---------- 16x12 ------- 182sq.in.
Pressure on masonry --- 13,580/182 --------- 74.5#/sq.in.
Allowed value S.H.B. pg. 56 is 168#/sq.in.

Under girders E4 & E3 on brick
Load ---------- 16,736#
Area ---------- 16x12 -------- 182sq.in.
Pressure on masonry 16736/182 ---------- 91.5#/sq.in.
Allowed value 168#/sq.in.

Under girders E2 & E1 on brick
Load ---------- 12,886#
Area ---------- 16x12 ------ 182 sq.in.
Pressure on masonry 12,886/182 ---------- 71#/sq.in
Allowed value 168#/sq.in.

Under girders H1 & H2 on brick
Load ---------- 7,388#
Area ---------- 16x8 ---------- 128sq.in.
Pressure on masonry 7,388/128 ---------- 59#/sq.in.
Allowed value 168#/sq.in.

Lintels in walls composed of 2 angles $3\frac{1}{2}x3\frac{1}{2}x5/16$
Span 6 feet; use load as triangular load of 6ft. base and 6 ft altitude.
Reactions $6'x17/12\ x3x120x\frac{1}{2}$ ---------- 1530#
Max moment 1530 x 6/4 ----------2300'#
Max moment as taken from S.H.B. is 2x1310 -- 2620'#
Since these lintels are supported in between the ends the angle irons are more than strong enough.

3

3. Resisting stresses

The resisting stress for common brick as taken from S.H.B. pg. 56 is 12 tons per sq.ft, or 168#/sq.in.
The resisting stress for concrete as taken from S.H.B. pg. 56 is 20 tons per sq.ft,-or 280#/sq.in.

4. Earth bearing

Area covered by cross section of walls -------- 1,600sq.ft.
weight of structure ---------- 5,630 tons
Bearing ----- 5,630/1,600 ---------- 3.5 tons/sq.ft.
The earth on which the building is located is clay and in S.H.B.pg.56 the allowed bearing value for clay is 4tons/sq.ft.

I. Fire resisting properties

The investigators, after inspecting the Lansing Auditorium, offer as their humble opinion, that little or no attempt has been made to produce a fire-proof or slow burning structure. The floors, interior finish and roof are constructed of wood, and for the most part, those parts enumerated are constructed of soft woods and are of comparatively thin dimensions, thus making the parts favorable for rapid consumption by fire. None of the wood has been treated with a fire resisting preparation in order that the fire hazard may be reduced.

The walls being of brick and containing very few openings would act similiar to a stack, thus allowing the interior to burn like fuel in a furnace. These openings in the wall are so placed and of such a nature, that if a fire started in the fan room, under the balcony floors, or under the roof in the dead space between the roof and ceiling, it would be practically impossible to reach the source of the fire. The fan room is a very favorable place for the starting of a fire. In this room are the fans, motors, and electric wiring which might ~~igni~~ ignite the wood thru heating or sparking. As noted in a preceding paragraph, at the time of the inspection by the investigators, the leads from the motor in the fan room were held to a pine board by nails bent over. The leads were not very widely separated thus offering favorable conditions for sparking between the leads if an excess of current should pile up at this point.

However, the building is amply equipped with fire escapes and exits thus preventing any serious calamity in case a fire occured when the auditorium was filled to its capacity.

Taking into account the increased cost of a fire proof structure and the small risk of destruction by fire; financially, the present construction is perhaps better engineering than an increase of expenditure by fire-proofing the structure.

Part III

Conclusion.

A. Concluding remarks on this thesis.

The report of the investigation is finished. Whether the authors have tested all the parts of the structure, they can not postively state, altho they are of the opinion that all parts of vital importance have been touched upon. The investigators have received a number of ideas from books, due to the fact that the design called for some research. As the authors noted in a previous paragraph, some of their methods may prove unique, but as a whole they are satisfied that they have done justice to the work, by attempting to investigate a structure in which so many stresses may occur. The authors are confident that in some parts, they have ommitted the stresses due to peculiar loadings, but as they did not know how to complete that part of the investigation, they did not tackle those portions.

B. Concluding remarks on theses in general.

The authors are here offering some suggestions that they would like to hear discussed by all other students who work theses. In leading up to this topic the authors will state some of the conclusions that they have drawn during the progress of their work. First; altho the authors have acquired considerable book-knowledge along lines of investigation, they feel that they did not go deep enough into the investigations of the various parts; second, the methods used for the investigation of the structure were acquired partly from books and partly from ideas of the investigators. The authors are not postive that the means so devised are correct or even close enough as to give values within the prescribed limits.

For the elimination of this trouble, the following idea is suggested; first, allow the student to name the subject which he thinks would be of interest to himself. Some of the subject might be Sewage Disposal, Water Supply, or Investigation of Some Structure. Second, after this choice has been made place all those desirous of taking one subject into a class, all those taking another subject into another class and so on. Third, assign an instructor to each class. Fourth, have these classes meet once a week for four weeks. At these meetings, each student must give a report of the time and methods used in conducting the work. The class can then comment on the methods employed and the instructor can correct errors in the methods employed. After the four weeks has passed the work should have progressed far enough so as to be finished in a proper manner.

It is with regret that the authors here write

FINIS

Part III

Conclusion.

A. Concluding remarks on this thesis.

The report of the investigation is finished. Whether the authors have tested all the parts of the structure, they can not postively state, altho they are of the opinion that all parts of vital importance have been touched upon. The investigators have received a number of ideas from books, due to the fact that the design called for some research. As the authors noted in a previous paragraph, some of their methods may prove unique, but as a whole they are satisfied that they have done justice to the work, by attempting to investigate a structure in which so many stresses may occur. The authors are confident that in some parts, they have ommitted the stresses due to peculiar loadings, but as they did not know how to complete that part of the investigation, they did not tackle those portions.

B. Concluding remarks on theses in general.

The authors are here offering some suggestions that they would like to hear discussed by all other students who work theses. In leading up to this topic the authors will state some of the conclusions that they have drawn during the progress of their work. First; altho the authors have acquired considerable book-knowledge along lines of investigation, they feel that they did not go deep enough into the investigations of the various parts; second, the methods used for the investigation of the structure were acquired partly from books and partly from ideas of the investigators. The authors are not postive that the means so devised are correct or even close enough as to give values within the prescribed limits.

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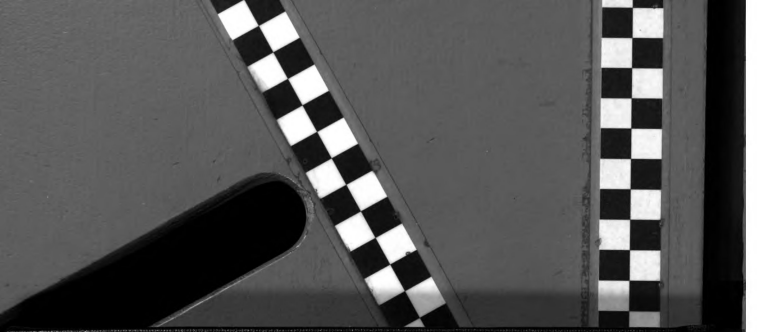

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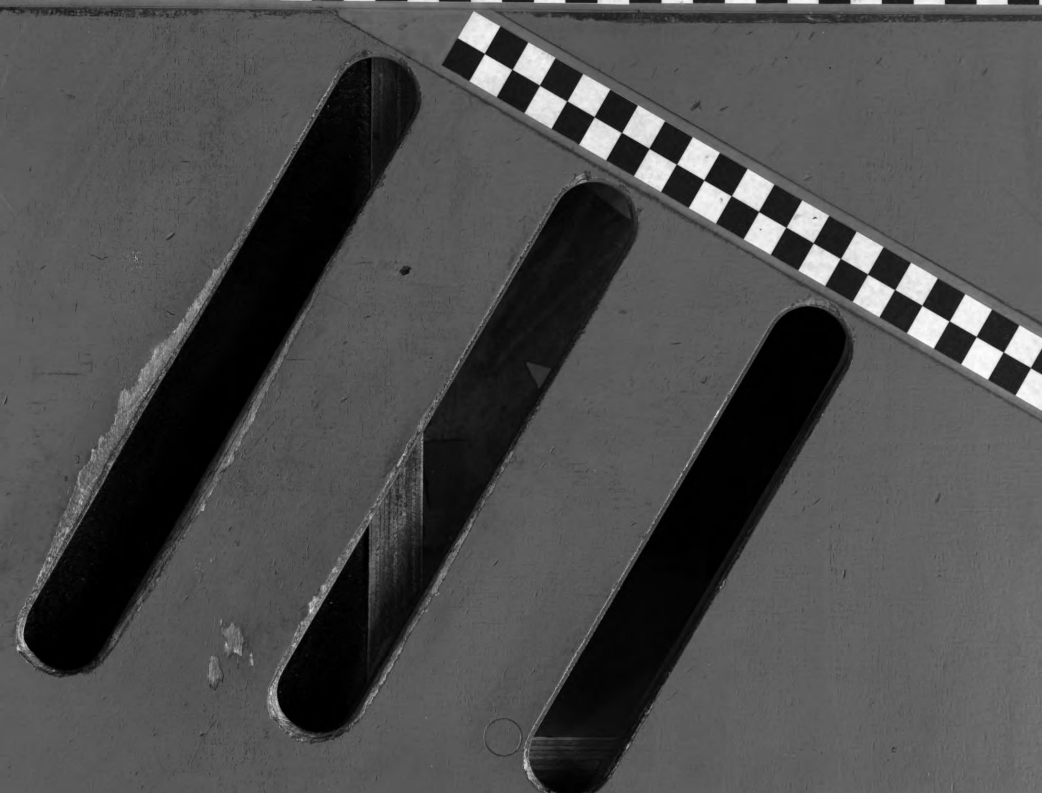

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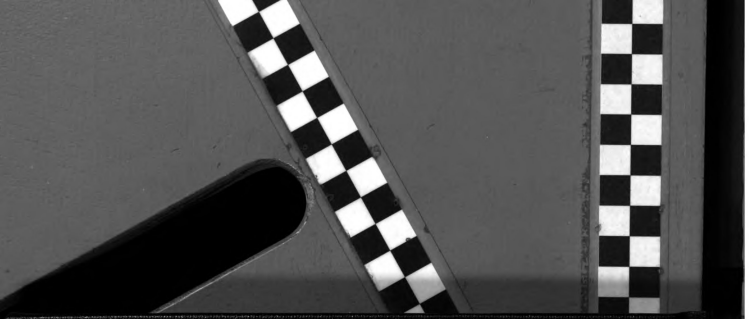

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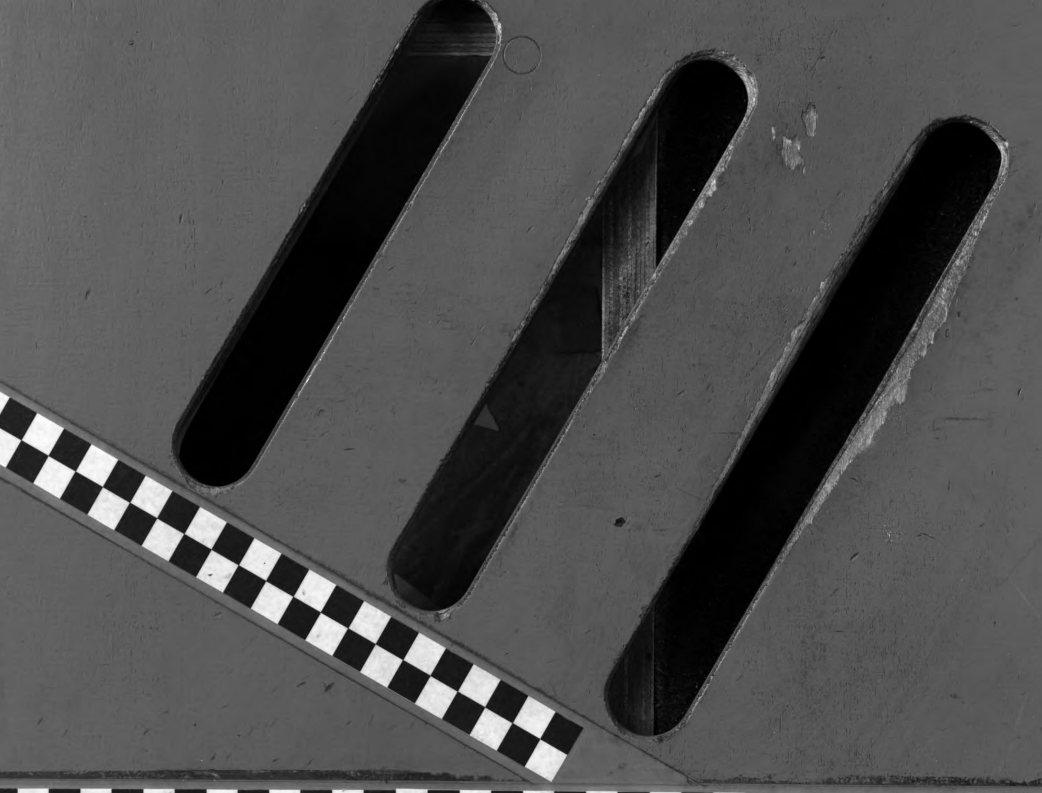

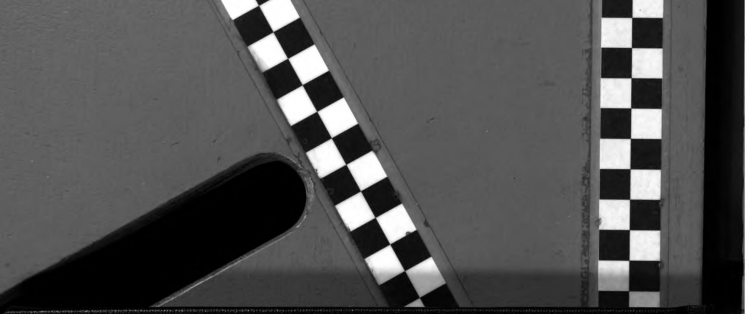

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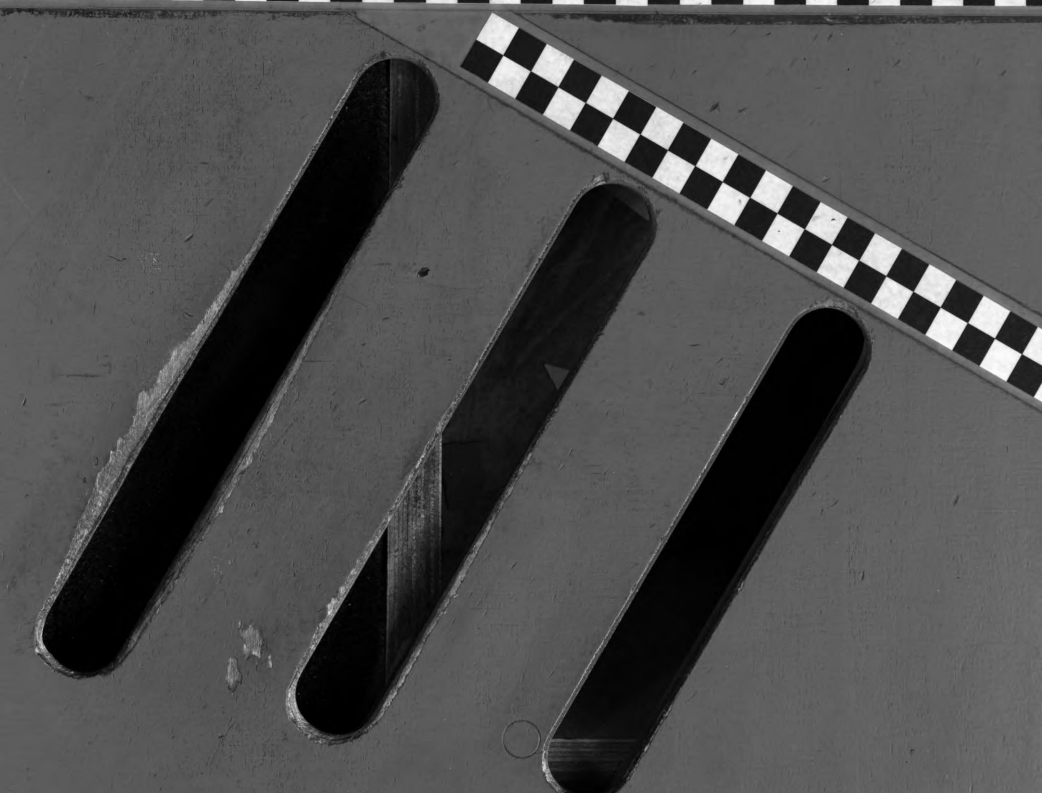

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