A STRESS ANALYSIS OF THE M. S. C. SCHOOL OF MUSIC

Thesis for the Degree of B. S.

MICHIGAN STATE COLLEGE

O. J. Bush

1940

A Stress Analysis of the
M. S. C.
School of Music

A Thesis Submitted to

The Faculty of

MICHIGAN STATE COLLEGE

of

AGRICULTURE AND APPLIED SCIENCE

by

O. J. Bush

Candidate for the Degree of

Bachelor of Science

Civil Engineering

June 1940

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The building considered herein is the new Michigan State College School of Music which was erected on the College campus during the spring and summer of 1939 by the J. A. Utley Co. of Detroit, Michigan. The building was designed and supervised by Malcomson, Calder, & Hammond Inc., Architects and Engineers, also of Detroit, Michigan.

The stress analysis of this building was undertaken with a view toward gaining practice in analyzing problems and also to gain some knowledge of the problems which arise in actual practice and their solutions.

The author here wishes to acknowledge Professor C. A. Miller of the Civil Engineering Department and Mr. Hammond of Malcomson, Calder, & Hammond for their help and guidance in the solution of this problem.

Structura

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Specifications

Structural Steel

I. Stresses

A. Bending:

Wall bearing steel.....16,000#/sq.in.

Steel frame construction.....18,000#/sq. in.

B. Shear:

Power driven rivets.....13,500#/sq. in.

Hand driven rivets & bolts.....10,000#/sq. in.

C. Bearing:

Bearing stresses shall be taken as twice the above shearing values.

D. Tension and Compression:

Tension.....18,000#/sq.in.

Compression stress shall be determined by the formula, $17,000 - .485\frac{l^2}{r^2}$, as per American Institute of Steel Construction.

II. Rivets and Bolts

All rivets and bolts to be (3/4") three quarters inches in diameter with (13/16") thirteen sixteenths inch holes punched for them.

III. Net Sections

A. Rivet holes to be taken as (1/8") one eighth inch larger than the rivets when calculating net sections.

B. Tension Members:

In calculating net sections for tension members, allow two holes cut per angle for angles 6"x6"

Concrete.

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or larger and one hole cut per angle for angles smaller than 6"x6".

Concrete.

I. Strength requirements for various uses.

Special - Concrete requiring strength in excess of 3500 pounds per square inch minimum by reason of special design or use.

A. - Concrete requiring 3000 pounds per square inch minimum. Concrete subject to excessive exposure, wear, or in continous contact with water. Reinforced members of small sizes, floors, sidewalks pavements, curbs, and gutters. All walls with earth on one side and floors on earth are to be of class A concrete.

B. - Concrete requiring a strength of 2500 pounds per square inch minimum. Concrete for general reinforced work, slabs, columns, walks, footings subject to severe exposure, and interior subfloors.

C. - Concrete requiring a strength of 2000 pounds per square inch minimum. Concrete for mass work not reinforced and not subject to severe exposure.

D.- Concrete for any use designated as not requiring the strength or quality of class C - such as fill concrete.

Reinforced concrete

Stresses

Using class B concrete for general reinforced work, we have:

$f'c = 2500\#/sq.\ in.$

$fc = 1000\#/sq.\ in.$

$n = 12$

$v_c = 50\#/sq.\ in.$

$v = 120\#/sq.\ in.$

Bond = 125#/sq. in. for plain bars.

= 150#/sq. in. for deformed bars.

Metal reinforcement

A. All metal reinforcements shall be deformed, rolled rail steel bars to meet the American Society for Testing Materials standard specifications, designation No. A-16-35.

B. Where the principle slab reinforcement is paralell to the concrete beams, transverse reinforcement, three tenths of the gross section of the slab, shall be provided in the top of the slab and shall extend across the beam and into the slab not less than two thirds of the allowable flange hangover(six times the thickness of the slab on each side of the beam). The spacing of bars shall not exceed 18".

C. Shrinkage or temperature steel, not less than one fifth of one percent of the gross area of the slab section, and running at right angles to the main reinforcement, shall be provided

in all concrete slabs which are reinforced in one direction.

D. The shrinkage steel shall be not more than one and one half inches below the top of the slab and shall be spaced not more than 12" apart or further than five times the thickness of the slab.

E. Bars paralell to the face of any member shall be embeded not less than one bar diameter from the face.

F. Splices in walls, columns, piers, and struts shall provide sufficient lap to transfer the stress by bond. This lap shall not be less than thirty bar diameters.

G. Metal reinforcement in wall footings and column footings shall have a minimum covering of 3" of concrete. All surfaces of concrete exposed to the weather and atmosphere shall have metal reinforcements protected by not less than 2" of concrete. In all cases the thickness of the concrete covering shall not be less than one and one half times the bar diameter.

General Notes

Concrete

1. Provide #5 wire loops 5" o.c. for 6-2 slabs extended to the 1/3 point where called for.
2. Provide #6 wire loops 5" o.c. for 8-2 slabs extended to the 1/3 point where called for.
3. Provide ¼" round bars 12" o.c. or equivalent area at right angles to the main steel in all joist construction slabs.
4. All bars to be bent for shear and continuity
5. All columns to have ¼" round stays 8" o.c. for ½" round bars; 10" o.c. for 5/8" round bars; 12" o.c. for 3/4 " round bars or more unless otherwise stated.
6. All columns to have dowels extending at least 30 diameters of vertical steel into columns above and to be of smae size or equivalent area to column steel above.
7. Soil pressure - Columns = 5000# per sq. ft.
   Walls = 5,000# per sq. ft.
8. All slabs to have 8" end bearing unless otherwise noted.
9. All beams to have 12" end bearing unless otherwise noted.

Loads

1. Dead loads

Weight of concrete.....150#/cu. ft.

Gypsum.....50#/cu. ft.

Roof;

Slate.....12#/sq. ft.

3" Gypsum.....15#/sq. ft.

Ceiling load.....12#/sq. ft.

Purlins.....4#/sq. ft.

43#/sq. ft.

Add two pounds per square foot when computing rafter loads and twelve pounds per square foot when computing truss loads.

11. Live loads

Stairs and landings.....100#/sq. ft.

Auditorium floor and balcony.....80#/sq. ft.

Corridors and lobby.....80#/sq. ft.

All other rooms.....60#/sq. ft.

Roof load.....30#/sq. ft.

Purlins

The purlins are considered as simple beams, freely supported at both ends, and carrying a uniformly distributed load causing a bending moment equal to $wl^2/8$. They are checked in bending by comparing the section modulus supplied ($\frac{I}{c}$) with the section modulus required($\frac{M}{s}$).

Size.	Length	Wt. per ft.	Moment"#	Section Mod Required	Section Mod Supplied
10 CBJ 11.5	16.5'	365	149,000	8.30	10.50
6 CBJ 8.5	11.5'	"	82,500	4.03	5.07
8 CBJ 10.0	15.5'	"	132,000	7.30	7.79
6 CBJ 8.5	11.5'	"	-------	----	-----
10 CBJ 11.5	17.0'	"	176,000	9.78	10.5
6 CBJ 8.5	12.0'	"	79,000	4.40	5.07
12 CBJ 14.0	12.0'	"	254,000	14.10	14.80
13 Ch. 31.8	27.0'	"	400,000	22.20	26.90
8 CBJ 10.0	15.5'	"	-------	-----	-----

All purlins of like denomination as those given in the table but of shorter length are not checked because of their obvious safety.

Computations:

10 CBJ 11.5 $\quad I/c = 10.50 \quad$ Length = 16.5 $\quad w = 365$

$$B.M. = wl^2/8 = \frac{365(16.5)^2}{8} \times 12 = 149{,}000''\#$$

$$\frac{M}{s} = \frac{149{,}000}{18{,}000} = 8.3 \text{(less than 10.50, therefore O.K.)}$$

Rafters & Roof Beams

No.	Size	Max. Moment	Sect. Mod needed M/s	Sect Mod. supplied L/c	Remarks
---	10"CBJ11.5	198,000"#	10.9	10.80	Shear O.K.
---	8"CBJ 10.0	101,000"#	5.7	7.76	"
---	10"WF 26.0	300,000"#	16.5	27.6	"
---	14"WF 34.0	830,000"#	46.0	48.5	"
---	10"CBJ15.0	230,000"#	12.8	13.8	"
---	10"CBJ11.5	170,000"#	9.45	10.5	"
---	"	64,800"#	3.6	10.5	"
---	10"WF 21	176,000"#	9.75	21.5	"
---	12"CBJ16.5	212,000"#	11.75	17.50	"
---	14"WF 34	586,000"#	32.6	48.5	"
R-1	10"WF 23	292,000"#	16.2	24.1	"
R-2	10"WF 29	390,000"#	21.8	30.0	"
R-3	10WF 33	495,000"#	27.5	35.0	"
R-4	12WF 40	835,000"#	46.4	51.9	"
R-5	10"WF 14	310,000"#	17.2	44.5	"
R-6	12"WF 36	648,000"#	36.0	45.9	"
R-8	10"WF 33	510,000"#	28.3	35.0	"
R-7	10"WF 29	398,000"#	22.1	30.8	"
*---	18"WF 50	1,780,000"#	98.6	89.0	"
*---	18"WF 50	1,955,000"#	107.0	89.0	"
---	10"CBL21	200,000"#	11.1	18.8	"
---	12"WF 25	509,000"#	28.2	30.9	"
---.	8"WF 17	147,000"#	8.15	14.1	"
---	10"WF 21	269,000"#	14.9	21.5	"

Rafters & Roof Beams (con't)

The two roof beams marked (*) were evidently considered as partially restrained beams because of the fact that they were framed into trusses No. 3 and No. 4 with trusses No. 5 producing a cantilever effect on the beams. If this is true the section moduli 98.6 and 107 are reduced to 85.7 and 79.0 which are just slightly smaller than the section moduli supplied and are therefore O.K.

The reasons for the variation in some of these members is beyound the author unless perhaps there are reasons which present themselves in the detailing of the steel.

Truss T-5

Plate I

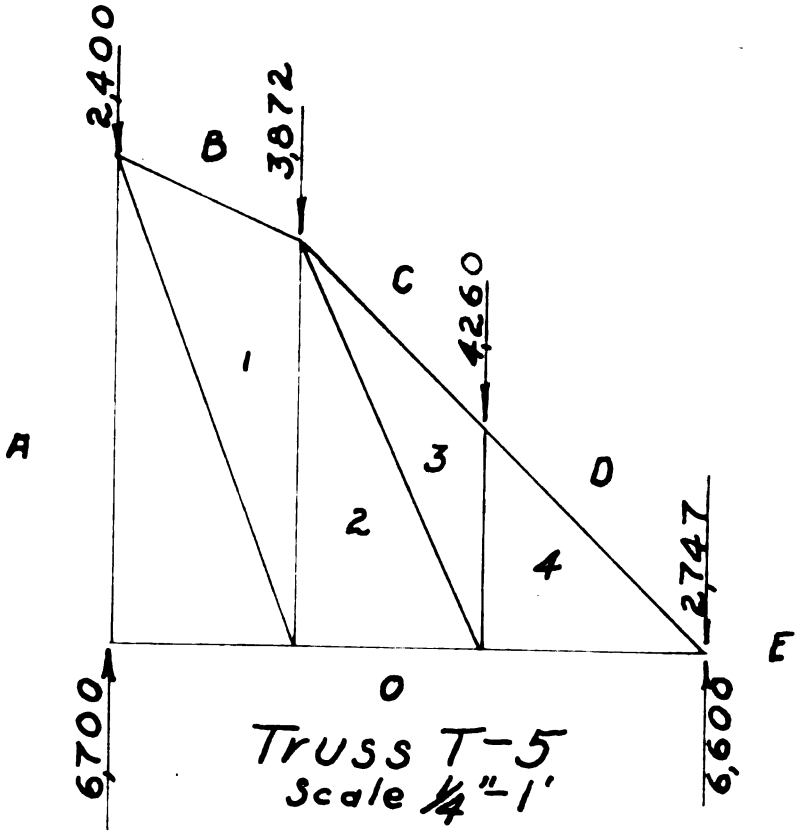
No.	Member	Length ft.in.	Load kips	Area sq.in.	Stress #/sq.in.	r	l/r	Allowable #/sq. in.
a	B-1	4-00	-02.10	2.38	900	.77	62.5	15,1000
a	C-3	5-3	-05.65	2.38	2,370	.77	82.0	13,750
a	D-4	6-03	-05.50	2.38	2,310	.77	97.5	12,500
a	O-4	4-06	3.85	1.97	1,940	.77	----	18,000
a	O-2	3-09	01.97	1.90	1,940	---	----	"
b	3-4	4-06	-05.20	2.12	2,450	---	----	14,580
b	3-2	9-00	04.90	1.71	2,860	---	----	18,000
b	1-0	10-06	05.50	1.71	3,220	----	-----	18,000
c	1-2	8-02	-05.20	2.62	2,000	---	----	11,490

For truss T-5;

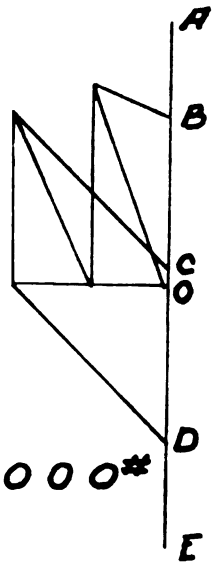
a - 2L - $2\frac{1}{2}$ x $2\frac{1}{2}$ x $\frac{1}{4}$

b - 2L - $2\frac{1}{2}$ x 2 x $\frac{1}{4}$

c - 2L - 3 x $2\frac{1}{4}$ x $\frac{1}{4}$



Truss T-5
Scale 1/4" = 1'



Scale: 1" = 5,000#

Plate I

Truss T-4

Plate II

No.	Member	Length ft.-in.	Load kips	Area sq.in.	Stress #/sq.in.	r	l/r	Allowable #/sq.in.
tc	B-1	4-08	-50.0	4.18	12,000	.89	62.9	15,100
tc	C-2	4-08	-48.0	"	11,500	"	62.9	15,100
tc	D-4	4-06	-45.0	"	10,750	"	60.4	15,250
tc	E-6	8-03	-31.3	"	7,500	"	111.0	10,850
bc	O-1	8-02	36.3	1.97	18,100	.77	-----	18,000
bc	O-3	4-04	34.0	"	17,200	"	-----	"
bc	O-5	8-04	32.2	"	16,300	"	-----	"
bc	O-7	8-04	30.2	"	15,300	"	-----	"
tc	F-8	4-06	-41.5	4.18	9,950	.89	60.4	15,250
tc	G-10	4-08	-44.0	"	10,500	"	62.9	15,100
tc	H-11	4-08	-47.0	"	11,200	"	62.9	15,100
bc	O-9	3-04	32.0	1.97	16,200	.77	-----	18,000
bc	O-11	8-02	34.0	"	17,250	"	-----	"
a	1-2	5-08	-4.0	2.12	1,900	.78	87.0	13,300
a	2-3	7-10	2.1	1.71	1,230	"	-----	18,000
c	3-4	8-06	-4.8	2.62	1,830	.95	101.0	12,100
a	4-5	11-00	4.5	1.71	2,640	.78	-----	18,000
c*	5-6	13-10	-1.7	2.62	650	.75	208.0	
c*	6-7	13-10	1.7	2.21	770	"	-----	18,000
a	7-8	11-00	4.3	1.71	2,500	.78	-----	"
c	8-9	8-06	-4.9	2.62	1,870	.95	101.0	12,900
a	9-10	7-10	2.2	1.71	1,290	.78	-----	18,000
a	10-11	5-08	-3.0	2.12	1,490	.78	87.0	15,300

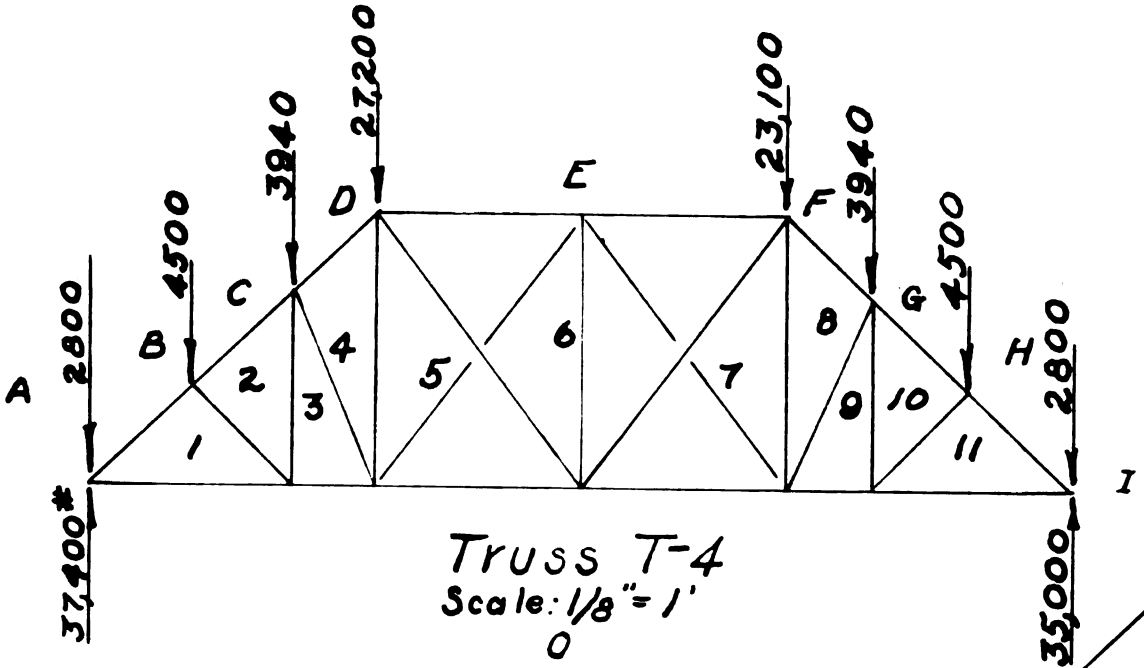
For truss T-4;

a - 2L - $2\frac{1}{2}$ x 2 x $\frac{1}{4}$ b - 1L - $2\frac{1}{2}$ x 2 x $\frac{1}{4}$

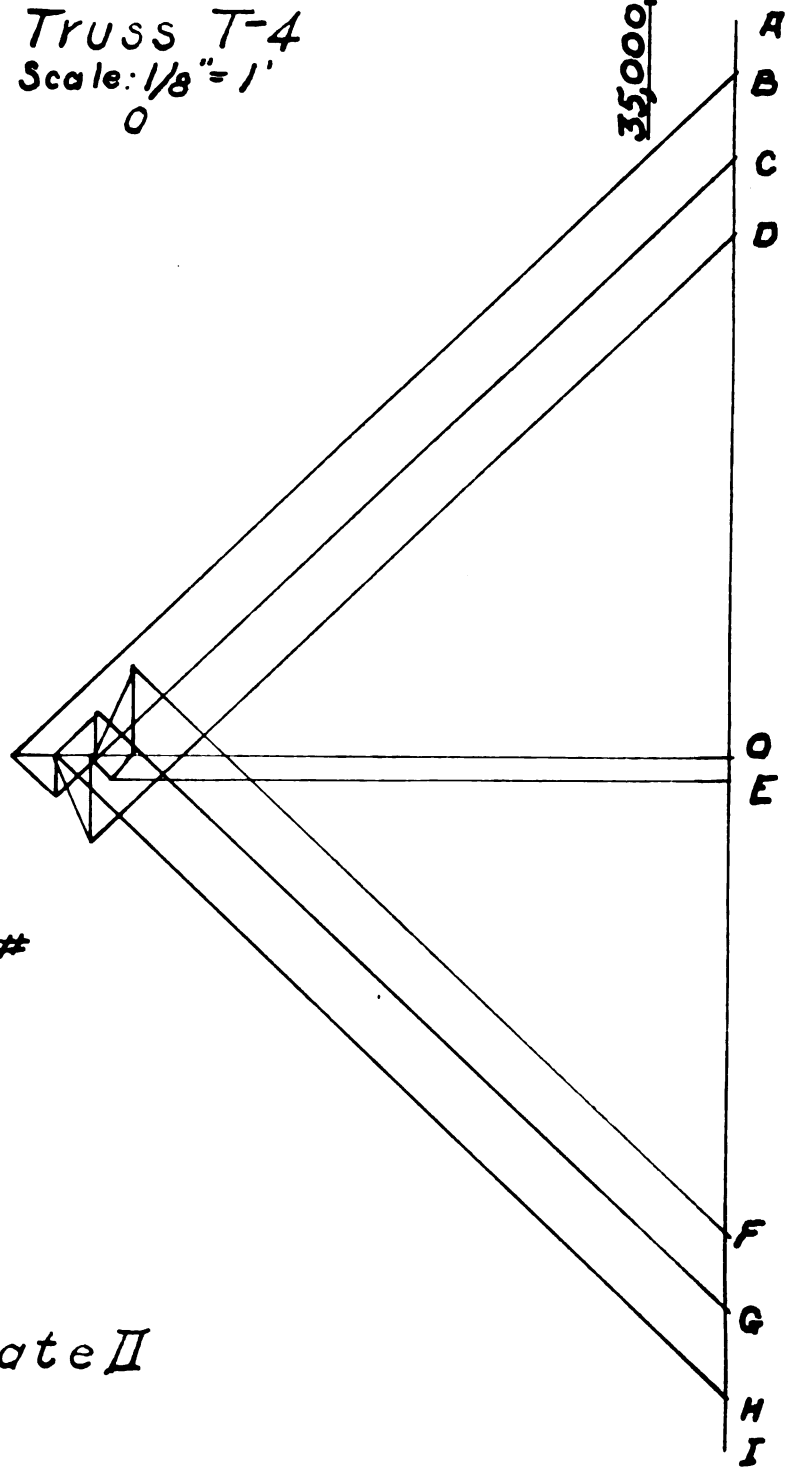
c - 2L - 3 x $2\frac{1}{2}$ x $\frac{1}{4}$ tc - 4 x 3 x 5/16 LLO

bc - 2L - $2\frac{1}{2}$ x $2\frac{1}{2}$ x $\frac{1}{4}$

Member 5-6 of this truss, a compression member, has an l/r ratio of 208 which is greater than that prescribed in the specifications for compression members. However, the limiting value of l/r, 120, is an arbitrary value and need not be strictly adhered to in extreme cases of stress, and since this member has a stress of only 650 pounds per square inch it is undoubtedly sufficient.



Truss T-4
Scale: 1/8" = 1'



Scale: 1" = 10,000#

Plate II

Truss T-3

Plate III

No.	Member	Length ft.-in.	Load kips	Area sq. in.	Stress #/sq.in.	r	l/r	Allowable #/sq.in.
a	5-6	7-10	3.0	1.77	1,700	.78	-----	18,000
a	6-7	5-08	-3.1	2.12	1,460	.78	82.1	13,300
tc	A-1	6-01	-28.0	4.18	6,700	.90	81.0	13,800
tc	B-3	7-11	-29.4	"	7,000	"	105.1	11,600
tc	C-4	4-06	-40.8	"	9,800	"	60.0	15,250
tc	D-6	5-04	-43.7	"	10,450	"	71.1	14,550
tc	E-7	5-10	-46.9	"	11,200	"	77.8	14,130
bc	0-2	15-10	28.0	1.96	14,300	.77	-----	18,000
bc	0-5	3-4	31.6	"	16,100	"	-----	18,000
bc	0-7	8-2	33.0	"	17,000	"	-----	18,000
b	0-1	8-2	37.8	2.21	17,100	.95	-----	18,000
b	4-5	8-06	-5.3	2.26	2,340	.95	107.3	11,500
c	1-2	11-00	-25.3	3.38	7,500	1.28	103.0	11,950
d	2-3	9-08	2.0	2.98	660	.90	-----	18,000
e	3-4	11-00	-4.0	2.88	1,390	1.12	118.0	10,280

For truss T-3;

a - 2L - $2\frac{1}{2}$ x 2 x $\frac{1}{4}$

b - 2L - 3 x $2\frac{1}{2}$ x $\frac{1}{4}$

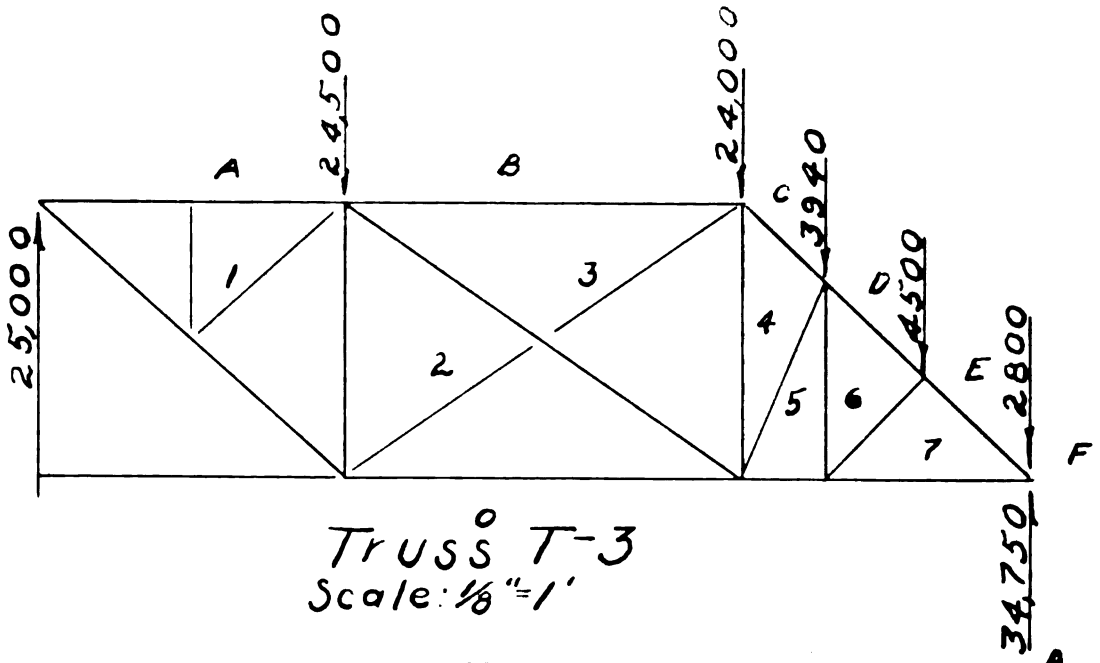
c - 2L - 4 x 3 x $\frac{1}{4}$

d - 2L - 4 x 3 x $\frac{1}{4}$ LLO

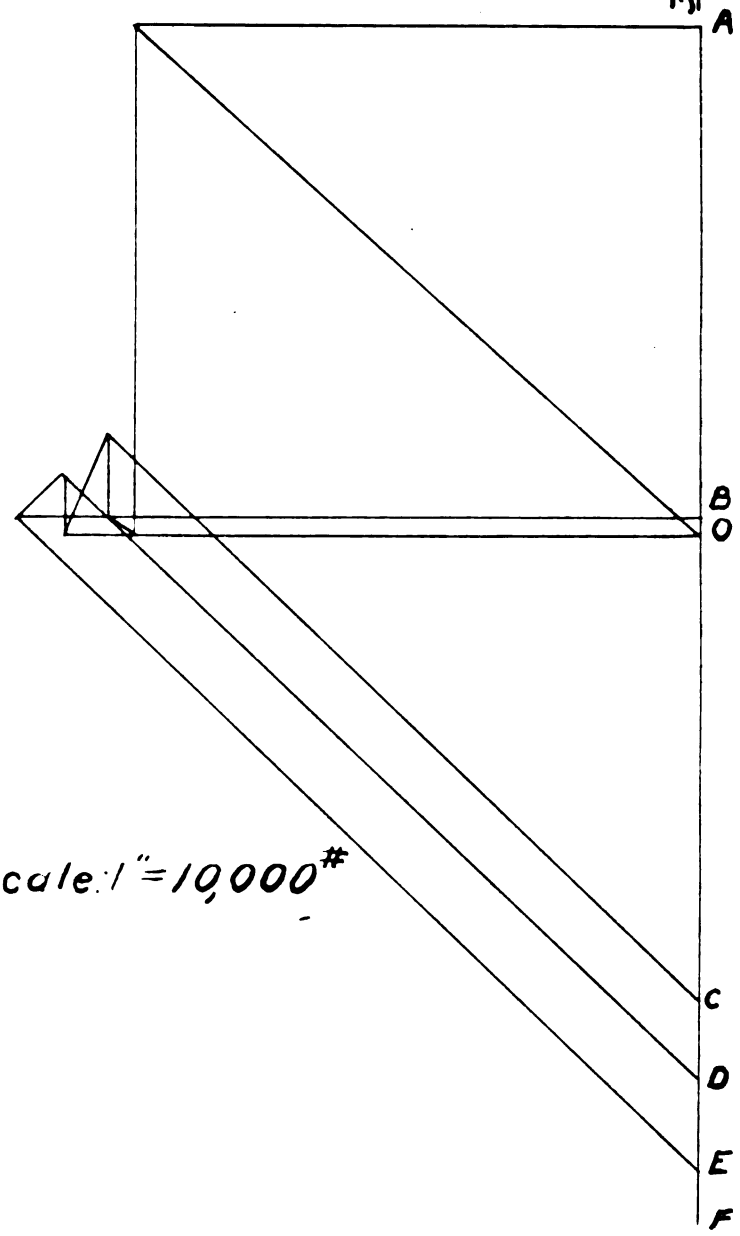
e - 2L - $3\frac{1}{2}$ x $2\frac{1}{2}$ x $\frac{1}{4}$

tc - 2L - 4 x 3 x 5/16

bc - 2L - $2\frac{1}{2}$ x $2\frac{1}{2}$ x $\frac{1}{4}$



Truss T-3
Scale: 1/8" = 1'



Scale: 1" = 10,000#

Plate III

Truss T-I

Plate IV

No.	Member	Length ft.-in.	Load kips	Area sq.in.	Stress #/sq.in.	r	l/r	Allowable #/sq.in.
tc	A-1	4-06	-38.0	6.50	5,850	.86	62.8	15,000
tc	B-2	4-06	-38.0	"	5,850	"	62.8	15,000
tc	C-5	4-04	-52.5	"	8,090	"	60.4	15,175
tc	D-6	4-10	-52.5	"	8,090	"	67.5	14,800
tc	E-9	4-08	-55.3	"	8,510	"	67.5	14,800
tc	F-10	4-10	-55.3	"	"	"	65.0	14,950
tc	G-13	4-10	-55.3	"	"	"	67.5	14,800
tc	H-14	5-09	-55.3	"	"	"	67.5	14,800
tc	I-16	5-04	-46.8	"	7,200	"	80.0	13,900
tc	J-17	5-10	-55.8	"	8,130	"	74.4	14,350
d	1-0	4-02	51.0	3.62	14,100	.91	----	18,000
d	3-0	4-06	45.0	3.62	12,400	.91	----	18,000
a	1-2	4-00	-8.0	2.12	3,700	.78	64.2	15,000
a	2-3	6-04	6.2	1.71	3,630	"	----	18,000
a	5-6	6-06	-5.0	2.12	2,360	"	61.5	15,160
a	6-7	4-00	4.0	1.71	2,340	"	----	18,000
a	8-9	6-04	6.5	1.71	3,800	"	----	18,000
a	9-10	4-02	-7.7	2.12	3,630	"	61.5	15,160
a	12-13	5-06	6.2	1.71	3,630	"	----	18,000
a	13-14	5-10	-7.7	2.12	3,630	"	64.2	15,000
a	16-17	6-08	6.2	2.12	2,830	"	84.6	13,500
b	4-5	8-06	-25.0	2.62	9,550	.95	73.7	14,400
b	4-7	6-08	-22.0	"	8,440	"	84.3	13,550
b	7-8	8-06	-14.6	"	5,560	"	109.5	11,200
b	11-12	8-02	-25.0	"	9,550	"	103.0	11,900

Truss T-1(con't.)

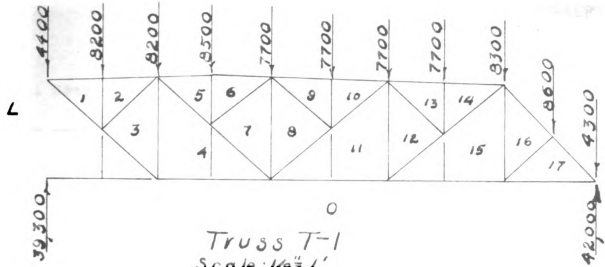
No.	Member	Length ft.-in.	Load kips	Area sq.in.	Stress #/sq.in.	r	l/r	Allowable #/sq.in.
b	15-16	8-00	4.3	2.21	1,950	----	-----	18,000
b*	8-11	6-08	0.0	2.21	0,000	.75	106.5	11,500
b*	10-11	6-00	6.0	2.21	2,720	.75	-----	18,000
c	12-15	6-00	23.7	2.48	9,550	1.12	-----	18,000
c	14-15	6-04	28.8	2.48	11,600	1.12	-----	18,000
c	3-4	8-06	-30.0	2.88	12,100	1.12	93.0	13,550
bc	4-0	9-04	33.5	3.62	9,250	.91	-----	18,000
bc	11-0	9-08	50.0	"	13,700	.91	-----	"
bc	15-0	9-08	32.2	"	8,900	.91	-----	"
bc	17-0	7-06	37.0	"	10,400	.91	-----	"

For truss T-1;

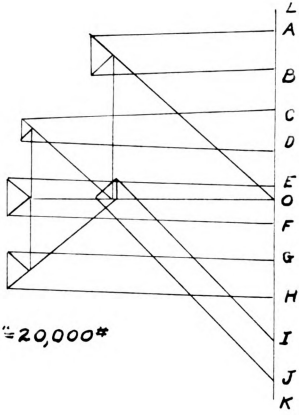
a - 2L - $2\frac{1}{2}$ x 2 x $\frac{1}{4}$ b - 2L - 3 x $2\frac{1}{2}$ x $\frac{1}{4}$

c - 2L - $3\frac{1}{2}$ x $2\frac{1}{2}$ x $\frac{1}{4}$ d - 2L - 4 x 3 x 3/8

tc - 2L - 4 x 3 x $\frac{1}{2}$ LLO bc - 2L - 3 x 3 x 3/8



Truss T-1
Scale: 1/8" = 1'



Scale: 1" = 20,000#

Plate IV

Truss T-2

Plate V

No.	Member	Length ft.-in.	Load kips	Area sq.in.	Stress #/sq.in.	r	l/r	Allowable #/sq.in.
tc	B-1	4-06	-50.0	7.00	7,150	1.23	43.9	16,065
tc	C-2	4-06	-50.0	"	7,150	"	43.9	16,065
tc	D-5	4-04	-68.2	"	9,750	"	42.2	16,320
tc	E-6	4-10	-69.2	"	9,750	"	47.1	15,960
tc	F-9	4-10	-69.6	"	9,950	"	47.1	15,960
tc	G-10	4-08	-69.6	"	9,950	"	45.6	16,000
tc	H-13	4-10	-70.5	"	10,000	"	47.1	15,960
tc	I-14	4-10	-70.5	"	10,000	"	47.1	15,960
tc	J-16	5-09	-55.8	"	7,980	"	56.0	15,470
tc	K-17	5-04	-62.0	"	8,860	"	52.0	15,700
bc	O-17	7-06	44.2	4.33	10,200	.88	-----	18,000
bc	O-15	9-08	39.5	"	9,120	"	-----	"
bc	O-11	9-08	64.0	"	14,800	"	-----	"
bc	O-4	9-04	43.9	"	10,130	"	-----	"
bc	O-3	5-10	57.8	"	13,350	"	-----	"
bc	O-1	5-10	66.0	"	15,250	"	-----	"
a	1-2	4-02	-11.0	2.12	5,200	.78	64.0	15,000
a	2-3	6-06	8.0	1.71	4,680	"	-----	18,000
a	5-6	4-00	-6.0	2.12	2,830	"	61.5	15,160
a	6-7	6-04	5.0	1.71	2,920	"	-----	18,000
a	8-9	6-06	7.8	1.71	4,560	"	-----	18,000
a	9-10	4-00	-10.0	2.12	4,710	"	61.5	15,160
a	12-13	6-04	8.8	1.71	5,150	"	-----	18,000
a	13-14	4-02	-10.0	2.12	4,170	"	64.0	15,000
a	16-17	5-06	-6.5	2.12	3,070	"	84.6	13,520

Truss T-2(con't.)

No.	Member	Length ft.-in.	Load kips	Area sq.in.	Stress #/sq.in.	r	l/r	Allowable #/sq.in.
b	4-5	5-10	32.5	2.21	15,300	.95	-----	18,000
b	4-7	6-08	27.5	2.21	12,950	"	-----	18,000
b	7-8	8-06	-18.0	2.62	6,900	"	110.5	11,090
b	11-12	8-02	-20.0	2.62	7,640	"	103.0	11,830
b	15-16	8-00	4.0	2.21	1,890	"	-----	18,000
b*	8-11	6-08	0.0	2.21	0,000	.75	-----	18,000
b*	10-11	6-00	7.2	2.21	3,390	.75	-----	18,000
c	3-4	8-06	-38.0	3.56	10,700	1.11	93.7	12,660
c	12-15	6-00	32.2	3.05	10,550	1.11	-----	18,000
c	14-15	6-04	40.5	3.05	13,300	1.11	-----	18,000

For truss T-2;

a - 2L - $2\frac{1}{2}$ x 2 x $\frac{1}{4}$ b - 2L - 3 x $2\frac{1}{2}$ x $\frac{1}{4}$

c - 2L - $3\frac{1}{2}$ x $2\frac{1}{2}$ x 5/16 b*- 2L - 3 x $2\frac{1}{2}$ x $\frac{1}{4}$ LLO

tc - 2L - 4 x $3\frac{1}{2}$ x $\frac{1}{2}$ bc - 2L - 4 x 3 x 3/8

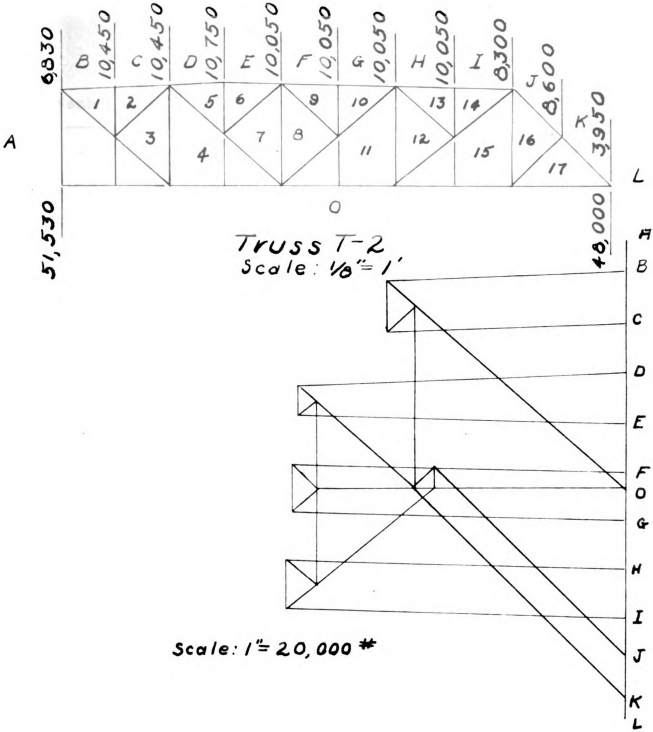

A
B
C
D
E
F
G
H
I
J
K
L
O
8,830
10,450
10,450
10,750
10,050
10,050
10,050
10,050
8,300
8,600
3,950
51,530
48,000
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
Truss T-2
Scale: 1/8" = 1'
Scale: 1" = 20,000 #

Plate V

Truss T-6

Plate VI

No.	Member	Length ft-in.	Load kips	Area sq.in.	Stress #/sq.in.	r	l/r	Allowable #/sq.in.
tc	A-1	5-04	-37.7	3.83	9,830	.74	86.5	13,380
tc	C-2	6-00	-30.0	"	7,820	"	97.5	12,400
tc	D-4	6-02	-23.9	"	6,230	"	100.0	12,150
tc	E-6	6-04	-19.2	"	5,000	"	102.8	11,900
a	1-2	5-10	-7.7	2.12	3,630	.78	90.0	13,150
a	2-3	6-06	3.9	1.71	2,280	"	100.0	18,000
a	4-5	10-02	6.3	1.71	3,680	"	-----	"
bc	1-0	9-04	30.8	2.21	13,900	.75	-----	"
bc	3-0	5-02	24.3	2.21	11,000	"	-----	"
bc	5-0	5-02	19.6	2.21	8,850	"	-----	"
b	3-4	8-04	-8.0	2.62	3,090	.95	105.0	11,600
c	5-6	11-04	-8.2	3.38	2,430	1.28	109.0	11,200
d	6-7	12-08	13.3	2.48	5,360	.93	-----	18,000

For truss T-6;

a - 2L - $2\frac{1}{2}$ x 2 x $\frac{1}{4}$

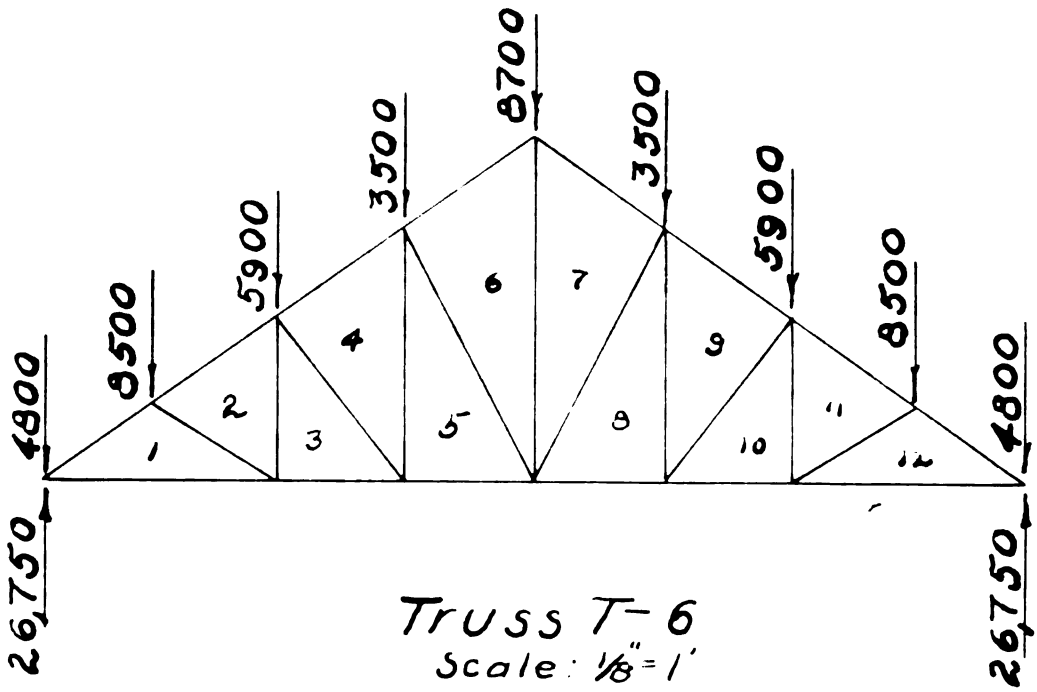
b - 2L - 3 x $2\frac{1}{2}$ x $\frac{1}{4}$

c - 2L - 4 x 3 x $\frac{1}{4}$

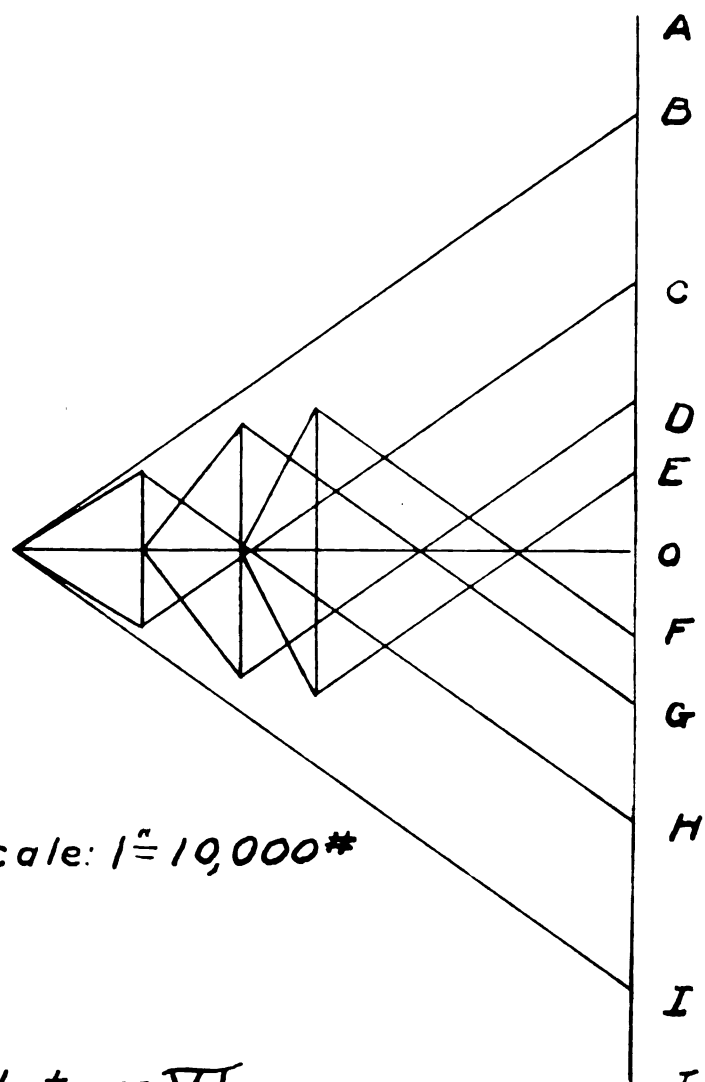
d - 2L - 3 x 3 x $\frac{1}{4}$

tc - 2L - 3 x $2\frac{1}{2}$ x 3/8

bc - 2L - 3 x $2\frac{1}{2}$ x $\frac{1}{4}$ LLO



Truss T-6
Scale: 1/8" = 1'



Scale: 1" = 10,000#

Plate - VI

Truss T-7

Plate VII

No.	Member	Length ft.-in.	Load kips	Area sq.in.	Stress #/sq.in.	r	l/r	Allowable #/sq.in.
tc1	A-1	6-03	-9,1	2.62	3,470	.95	79.0	14,000
tc1	A-2	5-08	-9.1	2.62	3,470	.95	71.5	14,500
tc2	B-5	4-01	-16.0	2.38	6,720	.77	63.6	15,080
tc2	C-7	5-02	-19.0	2.38	8,000	.77	80.6	13,850
tc2	D-8	5-08	-19.1	2.38	8,000	.77	88.3	13,220
bc	0-1	8-04	12.3	1.96	6,280	.77	----	18,000
bc	0-3	7-07	12.3	1.96	6,280	.77	----	"
bc	0-4	2-08	9.2	1.96	4,650	.77	----	"
bc	0-6	3-07	11.1	1.96	5,660	.77	----	"
bc	0-8	3-11	13.3	1.96	6,780	.77	----	"
a	1-2	5-08	0.0	1.77	0,000	.78	----	"
a	2-3	8-00	0.0	1.77	0,000	.78	----	"
a	4-5	11-00	7.5	1.77	4,340	.78	----	"
a	5-6	7-8	-7.4	2.12	3,500	.78	118.0	10,200
a	6-7	8-06	5.4	1.71	3,050	.78	----	18,000
a	7-8	4-00	-5.0	2.12	2,360	.78	61.5	15,160
b	3-4	10-08	-8.3	2.88	2,900	1.12	114.3	10,680

For truss T-7;

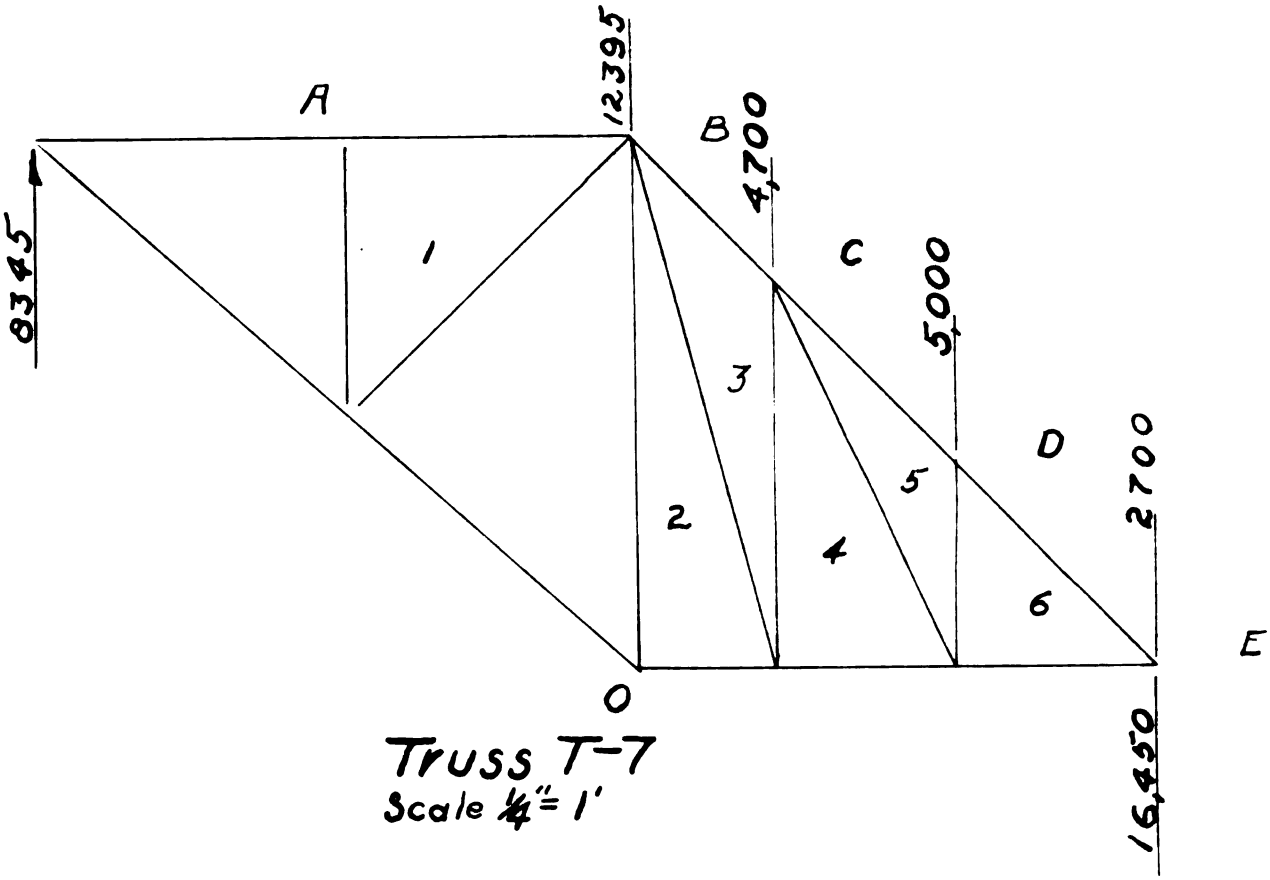
a - 2L - $2\frac{1}{2}$ x 2 x $\frac{1}{4}$

b - 2L - $3\frac{1}{2}$ x $2\frac{1}{2}$ x $\frac{1}{4}$

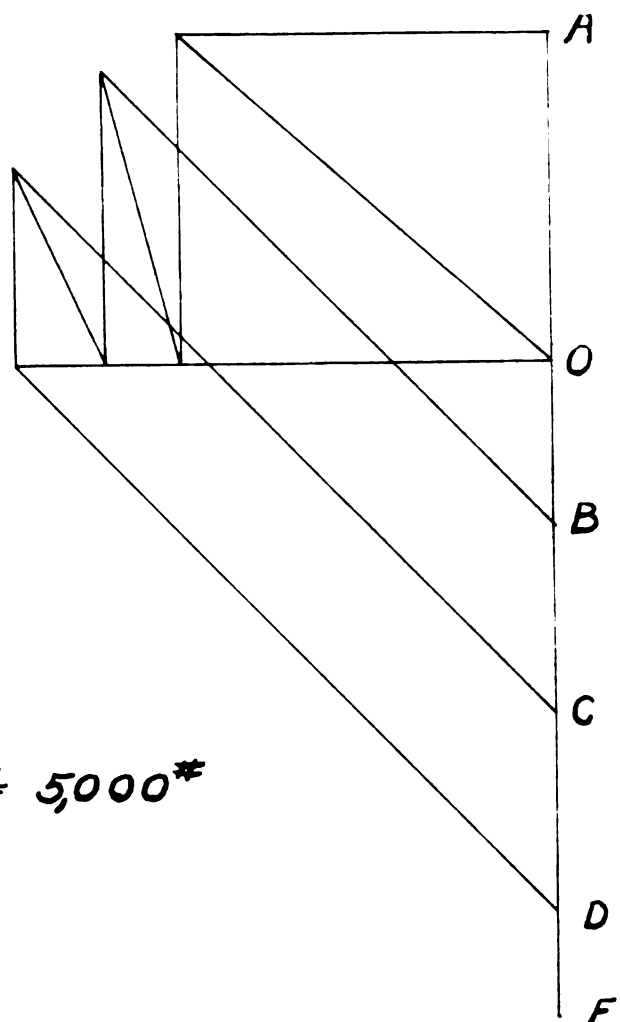
bc - 2L - $2\frac{1}{2}$ x $2\frac{1}{2}$ x $\frac{1}{4}$

tc1 - 2L - 3 x $2\frac{1}{2}$ x $\frac{1}{4}$

tc2 - 2L - $2\frac{1}{2}$ x $2\frac{1}{2}$ x $\frac{1}{4}$



Truss T-7
Scale ¼" = 1'



Scale 1" = 5,000#

Plate-VII

Truss T-8

Plate VIII

No.	Member &	length ft-in.	Load kips	Area sq. in.	Stress #/sq.in.	r	l/r	Allowable #/sq.in.
tc	A-1	4-10	-5.0	6.13	815	1.19	48.8	15,840
tc	B-2	7-09	-7.0	6.13	4,430	1.19	78.0	14,100
tc	D-3	7-10	-12.0	6.13	7,960	1.19	79.0	13,980
c	3-0	11-01	8.5	1.96	4,080	.77	----	18,000
d	1-2	11-00	-7.8	2.62	2,980	.95	123.0	9,560
b	2-3	7-09	-5.0	2.62	1,900	.96	98.0	12,350
a	1-0	12-00	12.5	1.71	7,350	.78	----	18,000

For truss T-8;

a - 2L - $2\frac{1}{2}$ x 2 z $\frac{1}{4}$ b - 2L - 3 x $2\frac{1}{2}$ x $\frac{1}{4}$

c - 2L - $2\frac{1}{2}$ x $2\frac{1}{2}$ x $\frac{1}{4}$ d - 2L - 3 x $2\frac{1}{2}$ x $\frac{1}{4}$

tc - 2L - $2\frac{1}{2}$ x $2\frac{1}{2}$ x $\frac{1}{4}$
10 x 3/8 stem pl.

The upper chord of this truss was designed to take a bending stress, and therefore the purlin loads may be brought to the upper chord at points other than the panel points. This practice results in a heavier upper chord than would otherwise be necessary, but by requiring less members it may also result in a cheaper truss.

Sample computations:

Compressiom Member; T-8, 1-2

Unsupported length - 11'-00"

Load -7.8 kips, area - 2.62 sq. in.s, stress - 2,980

Allowable stress = 17,000 - .485 $\left(\frac{l}{r}\right)^2$ = 9,560#/sq.in.

Tension member; T-8, 1-0

Gross area - 2.12

Net area = Gross area - two rivet holes

= 2.12 - .406 = 1.71 sq. in.s

Load(scaled from Plate VIII)-12.5kips

Stress = $\frac{12,500}{1.71}$ = 7,350 #/sq.in.

Allowable stress - 18,000#/sq.in.

Member designed for bending; T-8, D-3,

2L - $2\frac{1}{2}$ x $2\frac{1}{2}$ x $\frac{1}{4}$, Area - 2.38 , I_c = 1.40 in.^{4}

1 - 10 x 3/8 stem plate , Area - 3.75 , I_c = 31.3 in.^{4}

	Area	Area x $\bar{y}$
2L	2.38	21.8
St.pl.	3.75	18.75
	6.13	40.55

$\bar{x} = \frac{40.55}{6.13} = 6.61$

I(2L) = 1.40 + 2.38(9.16 - 6.61) = 16.88

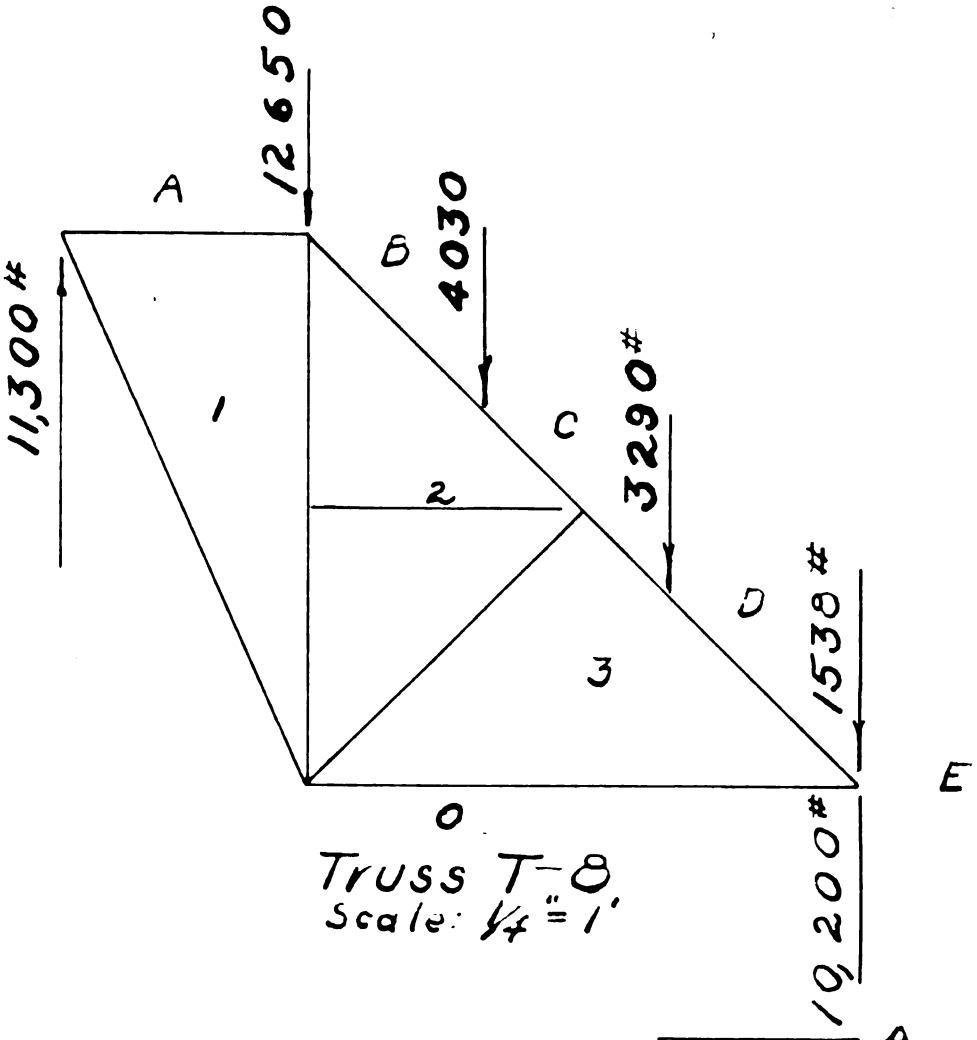
I(st.pl.)= 31.3 - 3.75(1.61) = 41.03

I(beam) = 57.91

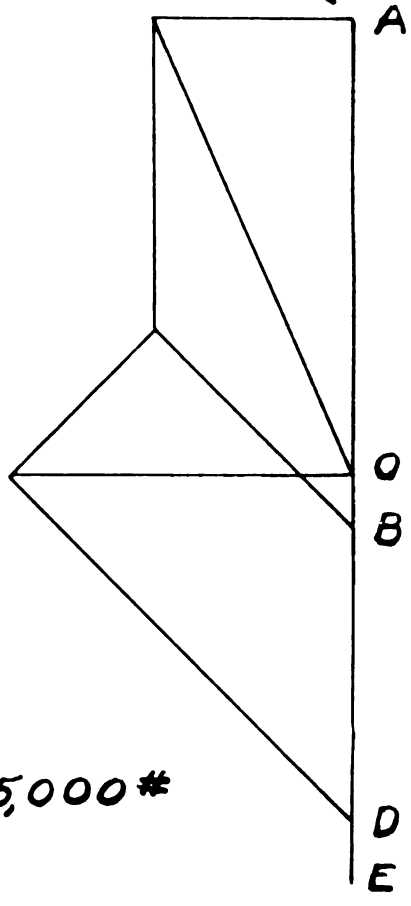
Stress = $\frac{P}{A} \pm \frac{M\ c}{I} = \frac{12.0}{6.13} + \frac{7500 \times 3.87 \times 12}{57.91}$

= 7,960#/sq.in.

Allowable in compression - 13,980#/sq. in.



Truss T-8
Scale: 1/4" = 1'



Scale: 1" = 5,000#

Plate VIII

Floor Slabs

Second Floor

Size & Type	Positive steel Need	Positive steel Supplied	Negative steel Need	Negative steel Supplied	Bond	Shear
8-2 S.T. @ 25"	.67	.78	.42	.44	105.0	57.8
12-2 S.T. @ 25"	1.32	1.78	.81	.78	97.5	78.0
6-2T.C.T.@ 17"	.44	.44	.28	.31	102.0	42.3
6-2T.C.T.@ 17"	.59	.60	.32	.31	102.0	49.0
10-2 S.T. @ 25"	.94	1.20	.58	.60	129.0	62.0
6-2T.C.T.@ 16"	.24	.31	.14	.11	87.0	36.2
6-2T.C.T.@ 16"	.31	.31	.19	.20	95.0	43.8
Slab B 5"solid	.25	.29	----	----	31.8	11.8
4"solid	.10	.17	----	----	58.6	8.1

First Floor

Size & Type	Positive steel Need	Positive steel Supplied	Negative steel Need	Negative steel Supplied	Bond	Shear
6-2T.C.T.@17"	.44	.44	.28	.31	102.0	42.3
Slab B 5"solid	.25	.29	----	----	31.8	11.8
6-2 S.T. @ 25"	.96	1.22	.96	1.04	120.0	82.5
8-2 S.T. @ 25"	1.00	1.38	1.00	1.04	115.0	59.0
4"solid	.27	.27	----	----	116.0	19.0
6-2T.C.T.@ 17"	.59	.60	.32	.31	102.0	49.0
6-2T.C.T.@ 16"	.24	.31	.14	.11	87.0	36.2
6-2T.C.T.@ 16"	.30	.31	.19	.20	95.0	43.8
12-2½ S.T.@ 25"	1.76	1.78	.59	.78	108.0	86.2
10-2½ S.T.@ 25"	1.71	1.78	.57	.78	128.0	103.0
10-2½ S.T.@ 25"	.96	1.20	.32	.60	105.0	58.0
6-2 S.T.. @ 25"	.56	.62	.56	.91	135.0	63.5

Floor Slabs

First Floor(con't)

Size & Type	Positive steel Need	Positive steel Supplied	Negative steel Need	Negative steel Supplied	Bond	Shear
8-2 S.T. @ 25"	1.09	1.22	.91	.91	93.8	88.7
6-2 S.T. @ 25"	.49	.61	.49	.91	56.5	58.4
6-2 S.T. @ 25"	.87	.88	.87	1.04	76.0	78.0
5" solid	.15	.19	----	----	84.0	11.0

The ribbed floor slabs are considered as beams of approximately equal spans built to act integrally with columns, walls, or other restraining supports and assumed to carry uniformily distributed loads, and designed for the following moments;

$$M_p = \frac{wL^2}{10}, \text{ and}$$

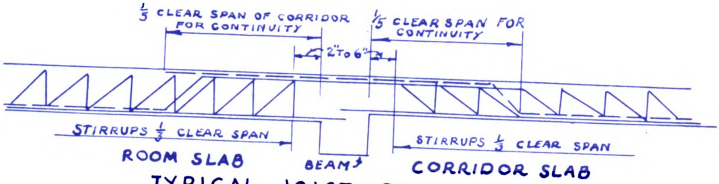
$$M_n = \frac{wL^2}{16}$$

The solid slabs are considered as freely supported simple beams with one-way steel and having a bending moment of;

$$M = \frac{wL^2}{8}.$$

All steel areas given for solid slabs are given in square inches per foot of slab, while the values given for the ribbed slabs are given in square inches.

All slabs were found to very closely designed, and at points where some of the beams appear to be over - designed in negative bending, the large areas of negative steel are necessary because of the fact that the bars are bent for double continuitity and must supply some steel to take care of the negative moment in the adjacent beam.



TYPICAL JOIST DETAIL

Floor Beams

Second Floor

No.	Size	Positive steel Need	Positive steel Supplied	Negative steel Need	Negative steel Supplied	Bond	Shear
201	12"x22"	1.77	1.78	1.11	1.20	116.0	68.8
202	12"x16"	.94	1.50	1.14	1.24	97.0	109.0
203	12"x16"	1.77	1.78	1.11	1.20	116.0	68.0
204	12"x20"	1.76	2.00	.625	.78	102.0	74.0
205	12"x14¼"	.50	.88	.30	.31	58.0	18.5
206	12"x14¼"	.93	.93	.58	.62	86.1	30.0
207	12"x16"	1.48	1.48	.51	.60	120.0	75.0
208	12"x10"	.80	.88	.22	.20	124.0	34.2
209	12"x15"	.94	1.93	.55	.78	51.0	48.5
210	12"x18"	1.63	1.77	1.05	1.36	49.4	62.1
211	12"x22"	1.95	208	1.95	2.08	59.1	87.5
212	12"x16"	1.29	1.63	.79	.78	120.0	74.5
213	12"x20"	2.36	2.36	.80	.78	78.0	40.5
214	12"x12"	.83	.81	.83	.81	86.0.	78.6
215	12"x18"	1.65	1.88	1.03	1.00	104.0	66.6
216	10"x14"	.98	1.05	.32	.44	124.0	43.0
217	12"x16"	1.60	1.66	.70	.78	148.0	72.0
218	12"x16"	1.20	1.19	.40	.44	141.0	55.5
219	12"x12"	.39	.75	.24	1.00	136.0	55.0
220	10"x32"	1.68	1.80	.56	.60	122.0	67.0
221	10"x20"	2.02	2.01	.39	.44	139.0	66.6
222	12"x18"	.60	.62	.25	.44	92.0	30.0
223	12"x18"	.45	.61	.19	.20	92.0	30.0

Floor Beams

Second Floor (con't)

No.	Size	Positive steel Need	Positive steel Supplied	Negative steel Need	Negative steel Supplied	Bond	Shear
224	10¼"x19"	2.10	2.36	.75	.78	99.0	109.0
225	12"x18"	1.80	2.00	1.50	1.62	101.0	37.0
226	12"x18"	.99	1.19	.32	.31	129.0	55.0
227	12¼"x19"	.33	.40	.18	.20	64.0	16.5
228	12¼"x19"	.77	.82	.25	.20	84.5	27.0
229	12¼"x19"	.13	.40	.06	.11	49.0	12.5
230	10"x10"	.34	.40	.12	.31	165.0	32.2

Steel Beams on Second Floor

Type Beam	Length	Sect. Mod. required	Sect. Mod. supplied
16"WF 40#	17'-00"	24.3	64.4
16"WF 45#	21'-00"	37.1	72.3
16 WF 64#	21'-00"	61.2	102.4
12"WF 40#	13'-06"	25.0	51.9
12"WF 32#	13'-00"	17.6	40.7
12"WF 40#	16'-00"	27.8	51.9

The steel beams listed here are all over designed as far as their tensile and compressive stresses are concerned. However, these beams carry plastered walls and are therefore designed to a maximum deflection.

Floor Beams

First Floor

No.	Size	Positive steel Need	Positive steel Supplied	Negative steel Need	Negative steel Supplied	Bond	Shear
101	12"x22"	2.81	2.98	2.10	2.08	127.0	108.0
102	12"x16"	1.39	1.49	1.39	1.77	127.5	100.0
103	12"x16"	1.73	1.77	1.08	1.13	130.0	100.0
104	12"x16"	.42	.50	.42	1.13	100.0	50.0
105	12"x14½"	.98	1.19	.29	.31	114.0	44.0
106	12"x14½"	.43	.62	.54	.62	101.0	29.0
107	12"x16"	1.63	1.80	.56	.60	132.0	77.0
108	12"x10"	.83	.88	.83	.80	126.0	44.5
109	12"x15"	1.33	1.39	1.33	1.69	129.0	84.4
110	12"x18"	1.75	1.77	1.75	1.66	135.0	88.0
111	12"x22"	1.80	2.08	1.80	2.08	108.5	81.0
112	12"x16"	1.50	1.82	1.50	1.98	126.0	79.0
113	12"x20"	2.44	2.57	.81	1.00	154.0	74.5
114	12"x12"	1.13	1.19	1.35	1.44	188.0	101.0
115	12"x18"	2.15	2.34	1.35	1.44	94.8	91.5
116	12"x18"	1.44	1.56	.60	.78	230.0	54.0
117	12"x20"	1.35	1.56	1.35	1.39	166.0	71.5
118	12"x20"	1.11	1.20	1.11	1.56	144.0	58.0
119	12"x20"	1.46	1.56	1.46	1.56	160.0	65.6
120	12"x22"	1.89	1.98	.98	1.20	149.0	77.5
121	12"x22"	2.02	2.17	1.18	1.20	201.0	83.0
122	12"x22"	1.89	1.98	1.31	1.57	150.0	78.0

Floor Beams

First Floor(con't)

No.	Size	Positive steel Need	Positive steel Supplied	Negative steel Need	Negative steel Supplied	Bond	Shear
123	12"x22"	1.81	1.98	1.48	1.57	150.0	78.0
124	12"x20"	1.92	2.08	1.62	1.77	158.0	87.5
125	12"x20"	1.98	2.08	1.76	1.77	163.0	93.0
128	*12x27"	Length = 16'-00" W/s = 132.1				I/c = 180.0	
130	10"x12"	.21	.39	----	----	90.0	23.0
131	12"x14"	1.53	1.64	.43	.60	87.5	55.0
132	12"x18"	1.34	1.49	.53	.61	98.7	62.0
133	12"x14"	1.23	1.49	.54	.61	98.0	34.0
134	12"x14"	1.09	1.22	.51	.61	109.0	38.6
135	12"x22"	2.08	2.36	.69	.78	109.0	84.0
136	12"x22"	1.78	1.90	.90	1.38	127.0	63.0
137	12"x22"	1.67	2.17	1.38	1.38	141.0	60.0
138	12"x22"	2.14	2.78	1.58	1.64	165.0	89.0
139	22"x26"	2.43	2.78	1.58	1.64	165.0	89.0
140	12"x20"	1.71	1.80	1.58	1.60	122.0	67.0
141	12"x12"	.61	.75	----	----	110.2	34.4
142	12"x16"	1.44	1.64	.48	.44	89.2	63.4
143	10"x20"	1.62	1.80	.53	.60	102.0	78.0
144	8"x 8"	.44	.59	.14	.20	89.0	22.0

*224" C B 87#/ft. Encased

The floor beams were considered as beams freely supported or built to act integrally with beams, girders, or restraining supports, or beams built into masonry walls in such that only partial end restraint is developed, and carrying uniformly

distributed loads , and having the following moments at the critical sections:

Positive moment near the center and negative moment at the support of interior spans,

$$M = \frac{w\,l^2}{12}$$

Positive moment near centers of end spans and negative moment at the first interior support,

$$M = \frac{w\,l^2}{10}$$

Negative moment at end supports for all cases to be not less than

$$M = \frac{w\,l^2}{24}$$

All beams were found to be sufficient with the exception of one or two which had a stress of 21,000 pounds per sq. in. in the main steel. However, recent specifications have been more liberal in the tensile stress of steel and allow 22,000 pounds per sq. in. and 21,000 pounds per sq. in. is undoubtedly all right.

Sample computations:

Beam 109 12"x15" $l = 11.5'$ $w = 2292\#/ft.$

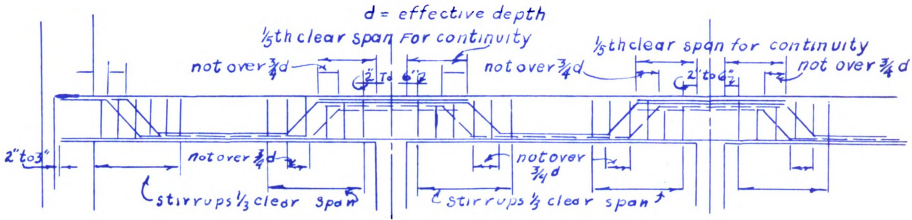
1-1"round B.D.C. 2-5/8" round st.

$$M = \frac{w\,l^2}{10} = \frac{2295(11.5)^2 12}{10} = 365{,}000"\#$$

$$A_s(p\&n) = \frac{M}{b\,j\,f_s} = \frac{356{,}000}{20{,}000 \times .875 \times 13} = 1.33\text{sq.in.}$$

$$A_s = \frac{365{,}000}{2.4 \times 20{,}000 \times .875 \times 13} = .55\text{sq.in.}$$
(end support)

1-1"round	.78	1-1"round	.78
2-5/8"round	.61	2-3/4"round(adj.beam)	.88
	1.39		1.66



TYPICAL BEAM DETAIL

Balcony

The beams in the balcony are designed as beams having cantilever ends projecting over the girder. This necessitates computing the bending moments in the beams with; 1 only the cantilever end loaded, 2 only the part between the columns and the girder loaded, and 3 with both parts loaded. The Maximum moment was found to be present when just the cantilever portion of the beams were loaded. Some of the balcony members, particularlly the girder, were over designed as far as the bending and shearing stresses are concerned, however, they are carrying plastered ceilings and therefore are designed to a maximum allowable deflection.

Joists

Concrete

No.	Size	Positive steel Need	Positive steel Supplied	Negative steel Need	Negative steel Supplied	Bond	Shear
JA	19"x 6"	.36	.44	.23	.31	84.0	30.0
JB	19"x 6"	.42	.44	.27	.31	99.0	35.0
JC	19"x 6"	.42	.44	.35	.31	99.0	35.0
JD	19"x 6"	.42	.44	.32	.31	97.0	34.0

Steel

Size	Max Moment	Sect. Mod. Needed M/s	Sect. Mod. Supplied I/c	Remarks
10"I 25.4	315,000"#	17.5	24.4	shear is O.K.
10"C 15.3	131,000"#	7.3	13.4	"
		Beams		
12"WF36#	676,000"#	37.5	45.9	"
2-7"Ch 9.8#	93,000"#	5.17	5.44	"
		Girder		
36"WF 230#	8,430,000"#	470.0	835.5	"

Columns

Supporting Second Floor

Steel

No.	Size	Plate	Load kips	Allowable load-kips	Bearing on plate
13	8"H 31	12"x10"x1"	54.0	132.0	450#/sq."
14	"	16"x12"x1"	89.0	132.0	463#/sq."
15	"	"	93.0	132.0	484#/sq."
16	8"H 48	20"x22"x2"	182.5	207.0	415#/sq."
17	8"H 31	12"x10"x1"	72.0	132.0	600#/sq."
18	"	12x 12"x1"	68.0	132.0	470#/sq."
19	"	12"x14"x1"	74.0	132.0	440#/sq."
20	"	12"x10"x1"	52.0	132.0	434#/sq."
23	8"H 48	18"x16"x1"	128.0	207.0	403#/sq."

Supporting Roof

Steel

1,4&9	6"H 15.5	8"x8"x3/4"	29.0	56.0	450#/sq."
2,5&10	"	8"x8"x3/4"	27.0	56.0	422#/sq."
3	"	"	30.0	56.0	470#/sq."
6&7	"	"	27.0	56.0	422#/sq."
8	"	10"x12"x1"	50.0	56.0	416#/sq."
11	"	8"x8"x3/4"	26.0	56.0	406#/sq."
12	"	10"x12"x1"	54.0	56.0	450#/sq."
13	8"H 31	10"x10"x3/4"	19.0	132.0	190#/sq."
14	"	"	53.0	132.0	440#/sq."
15	"	"	49.0	132.0	490#/sq."
16	"	"	53.0	132.0	530#/sq."

Columns (con't)

Supporting Roof

Steel

No.	Size	Plate	Load kips	Allowable load-kips	Bearing on plate
17	8"H 31	10"x10"x3/4"	25.0	132.0	250#/sq."
18	"	"	18.0	132.0	180#/sq."
19	"	"	21.0	132.0	210#/sq."
20	"	"	25.0	132.0	250#/sq."
21	6"H 15.5	8"x 8"x3/4"	22.0	56.0	344#/sq."
23	8"H 31	10"x10"x3/4"	53.0	132.0	530#/sq."

Sample Computations for steel columns(Supporting Roof No.12)

Load = 54.0kips 6"H 15.5#/ft. A = 4.59 sq. in/

length = 11'-00" r = 1.42

least radius of gyration = 1.42

$l/r = \frac{132}{1.42} = 93$(less than 120. O.K.)

Allowable stress $= 17{,}000 - .485 \left(\frac{l}{r}\right)^2$

$= 12{,}150$#/sq. in.

Allowable = 12,150 x 4.59 = 56,000#

Columns

Supporting Second Floor

Concrete

No.	Size	Main steel	Ties	Load kips	Stress #/sq."	Allowable #/sq. in.
1,4&9	12"x12"	4-5/8rd.	¼"rd.@10"	73.0	453	563
2,5&10	12"x12"	"	"	68.0	422	"
3	"	"	"	74.0	470	"
6	"	"	"	56.0	342	"
7	"	"	"	63.0	391	"
8	"	4-3/4"rd.	¼"rd.@12"	73.0	431	"
11	"	4-5/8"rd.	¼"rd.@10"	70.0	455	"
12	"	"	"	76.0	471	"
21	"	"	"	66.0	410	"
22	16"x36½"	8-3/4"rd.	¼"rd.@10"	23.0	540	"
		Supporting First Floor				
1,4&9	12"x12"	4-1"rd.	¼"rd.@12"	111.0	590	563
2,5&10	"	4-7/8"rd.	"	102.0	570	"
3	"	4-1"sq.	"	121.0	600	"
6	"	4-5/8"rd.	¼"rd.@10"	85.0	527	"
7	"	"	"	75.0	465	"
8	"	4-1"sq.	¼"rd.@12"	121.0	600	"
11	"	4-7/8rd.	"	101.0	567	"
12	"	4-1"sq.	"	123.0	615	"
13	"	4-5/8"rd.	¼"rd.@10"	84.0	520	"
15	12"x18"	6-1"rd.	¼"rd.@12"	156.0	400	"

Columns

Supporting First Floor (con't)

No.	Size	Main steel	Ties	Load kips	Stress #/sq."	Allowable #/sq. in.
16	20"x24"	8-7/8"rd.	½"rd.@12"	237.0	435	563
17	16"x12"	4-7/8"rd.	"	114.0	505	"
18	12"x12"	4-1"rd.	"	108.0	575	"
19	14"x12"	4-1"rd.	"	113.5	604	"
20	12"x12"	4-3/4"rd.	"	83.0	490	"
21	"	4-1"rd.	"	116.0	580	"
24&25	"	4-5/8"rd.	½"rd.@10"	49.0	304	"
26&27	"	"	"	32.0	200	"
28&29	"	"	"	49.0	304	"
30	"	"	"	39.0	243	"
31	"	"	"	26.0	162	"

Sample Computations for Concrete Columns:(Col. 18, First Floor)

12"x12", 4-1"rd. , ½"rd.@12"

Mom. Inerta = $bh^3/12$, Area = 12"x12" = 144 sq.in.

$p = \sqrt{\frac{I}{A}} = 3.46"$

Limiting height = 40p = 138"

Maximum height of Columns = 132", therefore all columns are short coulmns, and

$$\text{Stress} = \frac{P}{A_g} = \frac{108,000}{144 - 2.41(15-1)} = 575\#/\text{sq.in.}$$

where A_g is the gross area and P the total normal load.

Allowable stress = 563#/sq.in.

All ofthe columns were quite closely designed although some of them were slightly over stressed. A 12"x12" column with 4-5/8"rd. bars being the minimum size column allowed for buildings was used where ever it was possible.

Footings

No.	Size	Steel	Gross Sp	As Need	As Have	Bond #/sq. in.	Shear
1,4&9	6-8x6-8x16	12-1/2"sq.	2,600	2.60	3.00	103.0	206.0
2,5&10	6-4x6-4x15	11-1/2"sq.	2,750	2.47	2.75	140.0	272.0
3	5-8x5-8x16	12-1/2"sq.	3,490	2.31	3.00	97.0	223.0
6	5-8x5-8x14	9-1/2"sq.	2,810	1.66	2.25	70.0	160.0
7	6-0x6-0x15	10-1/2"sq.	2,270	1.74	2.50	94.0	158.0
11	"	10- "	3,000	2.30	2.50	124.0	209.0
12	6-9x6-9x16	12-1/2"sq.	2,920	3.10	3.00	129.0	232.0
13	6-4x6-4x15	11- "	2,280	2.05	2.75	116.0	226.0
14	8-0x8-0x18	14-5/8"rd.	2,600	4.15	4.30	130.0	130.0
15	7-8x7-8x18	13- "	3,040	4.50	4.10	131.0	224.0
16	4-9x4-9x14	20- "	5,550	5.72	6.12	68.5	95.0
17	6-4x6-4x15	11-1/2"sq.	3,000	2.67	2.75	152.0	296.0
18	6-8x6-8x16	12- "	2,620	2.62	3.00	104.0	210.0
19	5-8x5-8x16	11- "	3,750	2.25	2.75	93.0	213.0
20	6-4x6-4x15	11- "	2,260	2.03	2.75	115.0	224.0
21	6-8x6-8x16	12 "	2,800	2.80	3.00	111.0	224.0
24&25	4-4x4-4x13	11-3/8"rd.	2,750	.84	1.22	117.0	118.0
26&27	3-6x3-6x12	10-¼"rd.	2,780	.42	.50	156.0	82.0
28&29	4-4x4-4x13	11-3/8rd.	2,750	.84	1.22	117.0	118.0
30	4-0x4-0x13	15-½"rd.	3,500	.74	.75	106.0	125.0
31	3-4x3-4x12	9-½"rd.	2,400	.32	.44	150.0	64.0
8	7-0x7-0x16	12-5/8rd.	2,680	3.18	3.68	117.0	238.0

Sample computations for Footings: (Footing No. 8)

Column load - 121,000#
Footing - 9,800
130,800#

7-0x7-0x16, 12-5/8"rd.

$$\text{Gross Ep} = \frac{130,800}{49} = \underline{2,680\#/\text{sq. ft.}}$$

As by Bending,

$$As = \frac{M}{fsxjxd} \quad \frac{((36)^2 \times 2 + 3x1.5)2,680 \times 12}{(12)^2 x\ 20,000 \times 7/8 \times 13}$$

$$As = 3.18 \text{ sq. in.}$$

12-5/8" rd. bars = 3.68 sq. in.

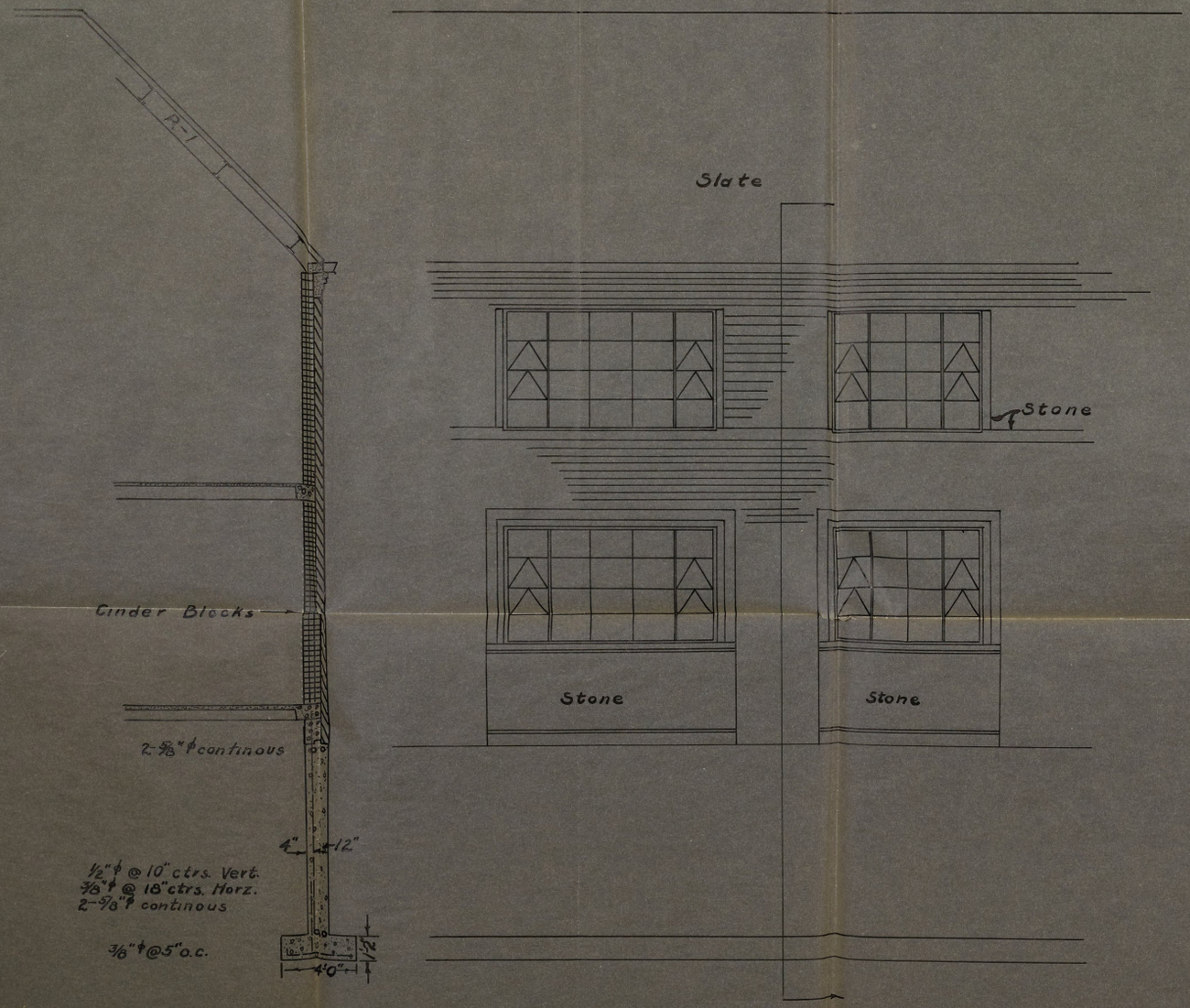
Shear,

$$s = \frac{(7x7-1)2,680}{48x13x7/8} = 238\#/\text{sq.in.}$$

Allowable with special anchorage - 225#/sq.in.

Bond,

$$u = \frac{(3x3 + 3)2,470}{13x7/8 \times 23.56} = 121\#/\text{sq.in.}$$



Section Thru South Wall

Scale: 1/4" = 1'

Walls

The walls of this building are load bearing walls replacing all exterior columns. However, due to a lack of time, only one section thru the south wall opposite column No. 11 will be analyzed. The reinforced concrete wall from the grade line down to the footing has horizontal steel both top and bottom giving it a beam action causing the load to be more evenly distributed along the footing than it ordinarily is by plain concrete or masonry walls.

Rafter Reaction	8,600 #
Lintel over 2nd floor windows	7,000#
Masonry	6,000
	13,000
Beam Reaction (222)	7,100
Beam Reaction (223)	6,500
Reinforced concrete wall	5,700
Footing	2,800
	56,700#

Gross Earth Pressure = $\frac{56,700}{16}$ = 3,550#/□'

This is OK as all footings are designed for 3500#/□' Earth Pressure

FOOTING

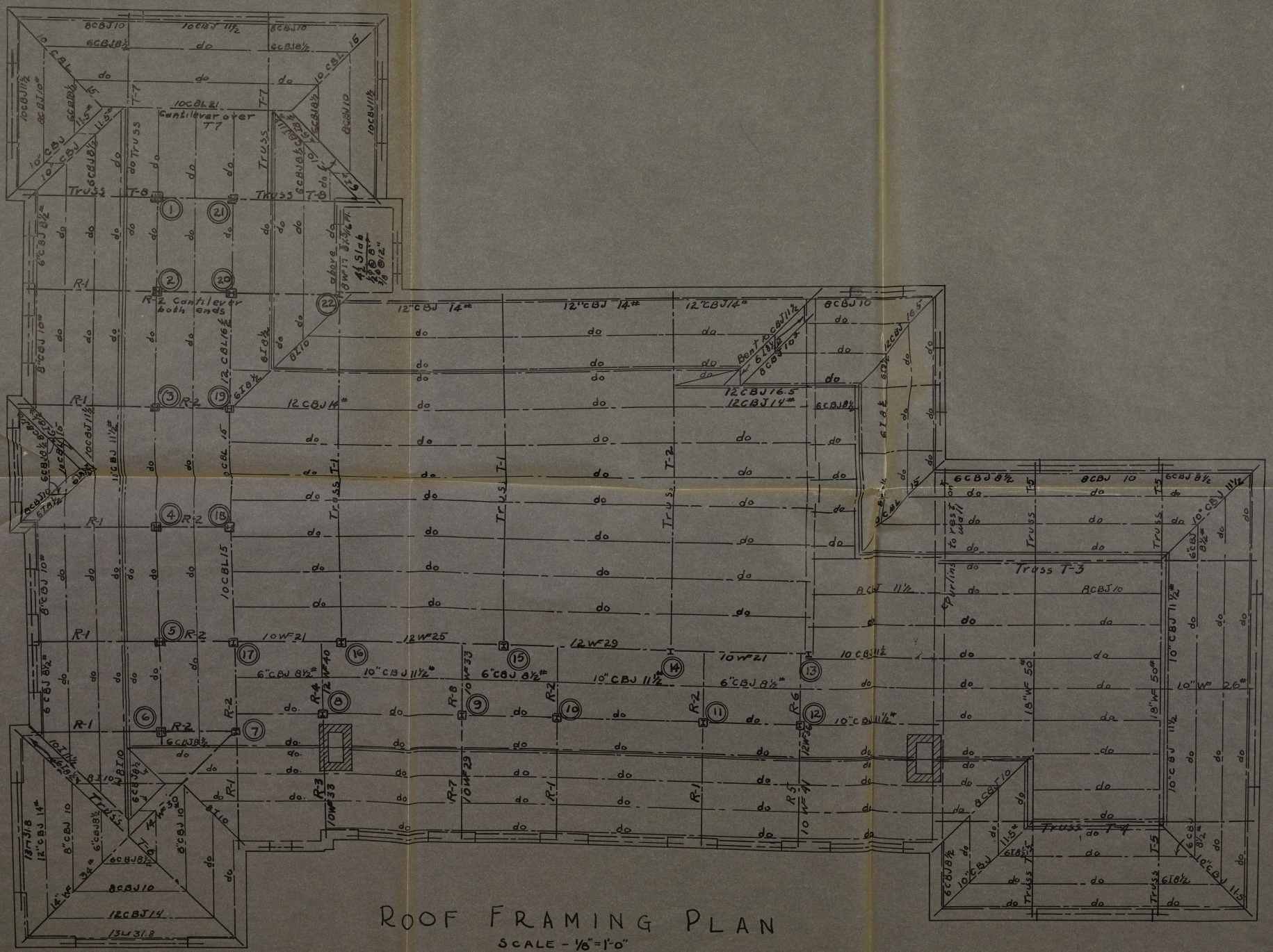
B.M. = $3,550 \times (1\frac{6}{12})^2 \times \frac{1}{2} \times 12$ = $3,140 \times 12$ "#

$d = \sqrt{\frac{3,140}{120}}$ = 5" + 3" = 8" ; 14" OK in bending

$A_s = \frac{M}{fJd} = \frac{3,140 \times 12}{20,000 \times \frac{7}{8} \times 11}$ = .196□"/ft

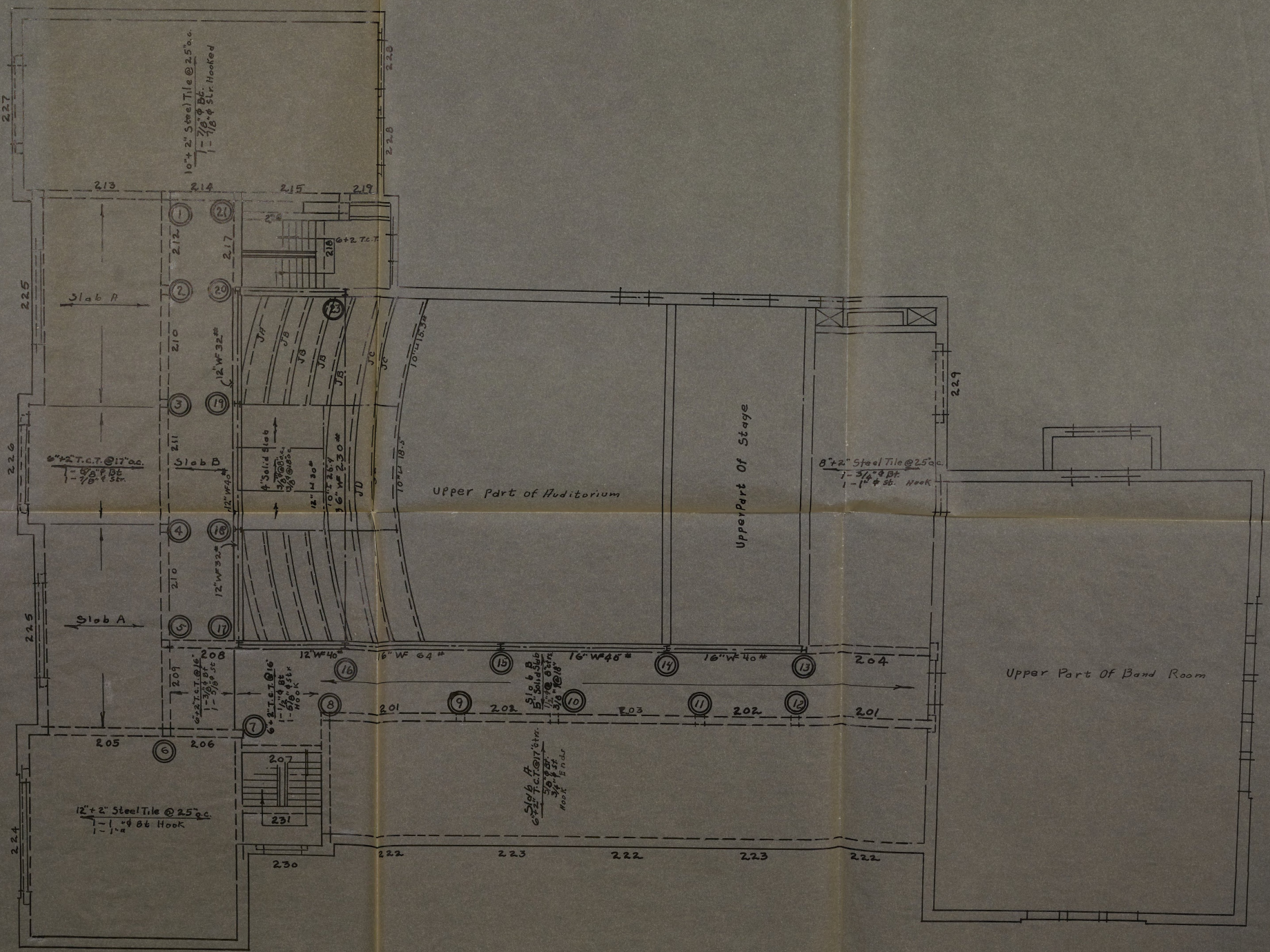
3/8"ϕ @ 5" o.c. = 26□"/ft OK.

Shear = $3,550 \times 1\frac{6}{12} \div 12 \times 11 \times \frac{7}{8}$ = 38#/□" OK.



ROOF FRAMING PLAN

SCALE - 1/8" = 1'-0"


10"+2" Steel Tile @ 25" o.c.
1 - 7/8" ϕ Bt.
1 - 7/8" ϕ Str. Hooked
Slab A
Slab B
6"+2" T.C.T. @ 17" o.c.
1 - 5/8" ϕ Bt
1 - 7/8" ϕ Str.
12" WF 32#
4" Solid Slab
Upper Part of Auditorium
Upper Part Of Stage
8"+2" Steel Tile @ 25" o.c.
1 - 3/4" ϕ Bt.
1 - 1" ϕ St. Hook
Upper Part Of Band Room
12" WF 40#
16" WF 64#
16" WF 45#
16" WF 40#
6"+2" T.C.T. @ 16"
1 - 3/8" ϕ Bt
1 - 5/8" ϕ Sc
6"+2" T.C.T. @ 16"
1 - 1/2" ϕ Bt
1 - 5/8" ϕ St.
Hook
Slab B
5" Solid Slab
Slab A
6"+2" T.C.T. @ 17" Str.
1 - 5/8" ϕ Bt
1 - 3/4" ϕ St.
Hook
End
12"+2" Steel Tile @ 25" o.c.
1 - 1" ϕ Bt Hook
1 - 1" ϕ
6+2 T.C.T.
2" ϕ
213
214
215
219
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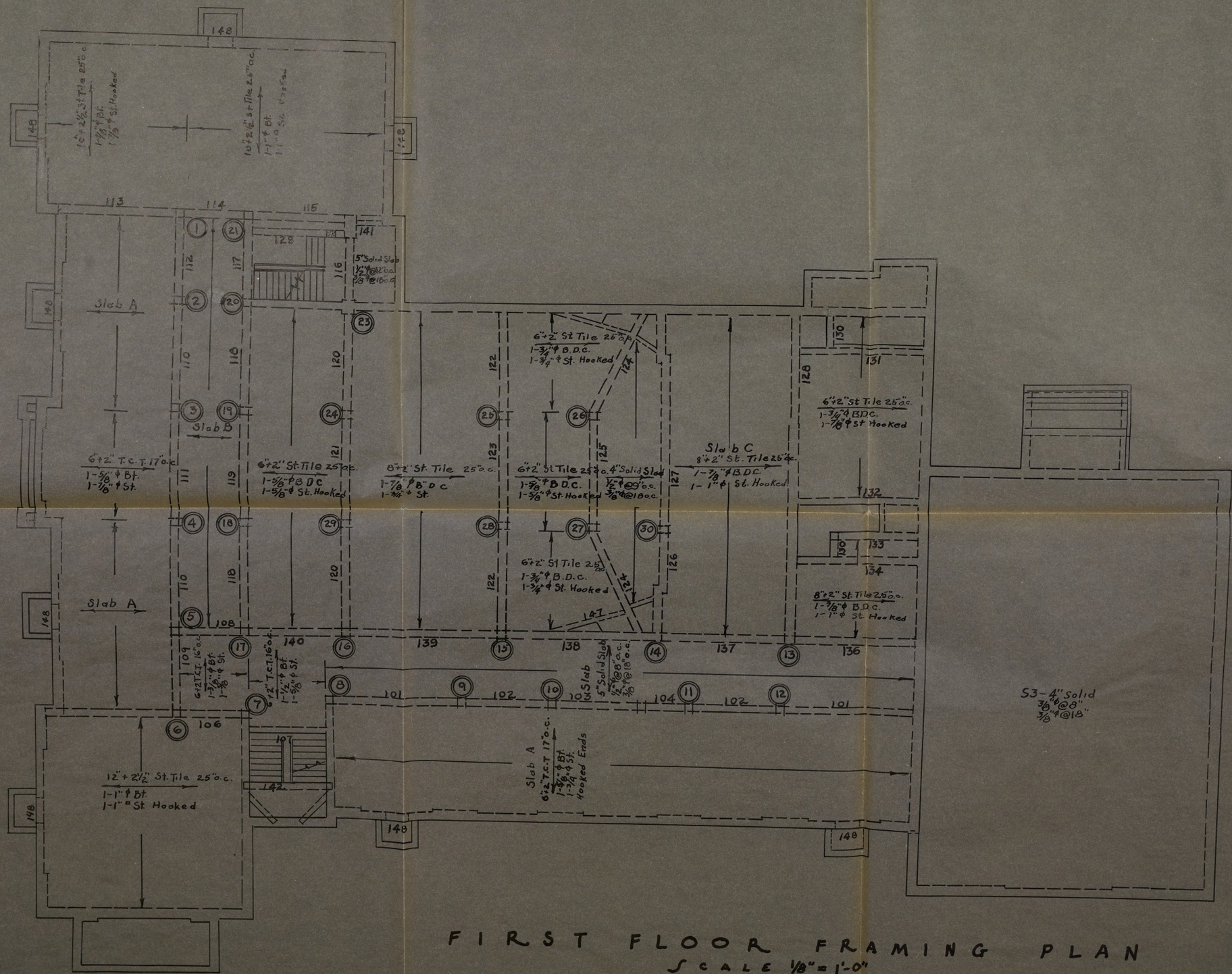

FIRST FLOOR FRAMING PLAN

SCALE 1/8" = 1'-0"