STUDY OF THE STRESSES OF A MOVABLE DAM FOR THE NEW LOCK IN THE SAULT STE MARIE SHIP CANAL AT SAULT STE MARIE, MICHIGAN

Thesis for the Degree of B. S.
MICHIGAN STATE COLLEGE

Eugene H. Rook

1942

THESIS

Study of the Stresses of a Movable Dam

for the

New Lock in the Sault Ste Marie Ship Canal

at

Sault Ste Marie, Michigan

A Thesis Submitted to

The Faculty of

MICHIGAN STATE COLLEGE

of

AGRICULTURE AND APPLIED SCIENCE

by

Eugene H. Rook

Candidate for the Degree of

Bachelor of Science

1942

THESIS

Table of Contents

Study of the Stresses of a Movable Dam

for the

New Lock in the Sault Ste Marie Ship Canal

at

Sault Ste Marie, Michigan

Movable dams are structures that are built to partially or totally restrict the flow of water in a channel or canal for a period of time, usually short. They are entirely removable from the place of restriction of flow or are collapsible so that they are out of the way and permit the flow of water by or over them. Thus, they recieve the name of a "movable" dam. It should be thoroughly understood that they are not permanent dams such as are made of concrete or masonary, but are either wood or steel structures that are used only in emergencies or temporarily for the purpose of making repairs, adjustments, or to regulate a water level.

There are many types of movable dams such as curtain dams, wicket dams, needle dams, shutter dams, rolling dams, drum dams, and so forth. However, most of these are useable only in shallow water, up to about 15 feet or in places where the water is restricted before its full head is attained. These types would not fit the conditions (which are stated on succeeding pages) under which the dam I am analyzing must operate. This dam must operate easily, quickly, and efficiently under a full head of about 35 feet, and at the beginning of placement most of this head will be converted into velocity. Therefore, the design must be more stable and stronger than the types listed above.

The data for ship canal at Sault Ste Marie, Michigan is listed

below. This data was secured with the kind help of Mr. H. F. Rook*.

Upper pool

Low Water Datum --- 601.1 ft.

High Water Monthly Mean --- 603.64 ft.

Maximum hourly change --- $\pm$ 4.0 ft.

Lower pool

Low Water Datum --- 579.4 ft.

High Water Monthly Mean --- 583.65 ft.

High water hourly extreme --- 585.22 ft.

Lowest extreme --- 3 ft. below Low Water Datum

All of these elevations are based on the Low Water Datum at New York.

Elevation of canal Wall --- 121.00 ft.

This elevation is based on the Sault Ste Marie Canal Datum.

The relation between the two is:

Low Water Datum = 603.64 ft. at the Sault Ste Marie Canal Datum = 117.37 ft.

Depth of lock sill below Low Water Datum = 31 ft. or elev. = 570.1 ft.

Width of canal directly above upper lock gate = 80 ft. This is clear distance between bumpers which project one foot each from the sides of the canal wall.

Length of lock between inner service gates = 800 ft.

The data as it was obtained is of no use; so to put it into usable form, the elevations must be converted into differences of elevations or heights. The average difference of elevation of the upper pool and lower

---

* Mr. Rook was, at the time the information was obtained, Principle Engineer for the U. S. Engineer Office, and is now a Lt. Colonel in the U. S. Army Engineer Corp.

pool is 601.1 ft. less 579.4 ft. gives 21.7 feet. This is not the extreme difference; so the difference for the maximum hourly extremes of both pools will be used. This difference is found by adding both the maximum hourly change of the upper pool from the Low Water Datum and the extreme low water level from the Low Water Datum of the lower pool; thus giving 21.7 ≠ 4.0 ≠ 3.0 or 28.7 feet difference. This extreme is not likely to occur as the lower pool is usually at its highest level when the upper pool is at its highest level. The reason for this is that there is a power and water level control dam above the rapids connecting the two pools other than the locks. However, the extreme conditions are possible, so they will be used in the computations.

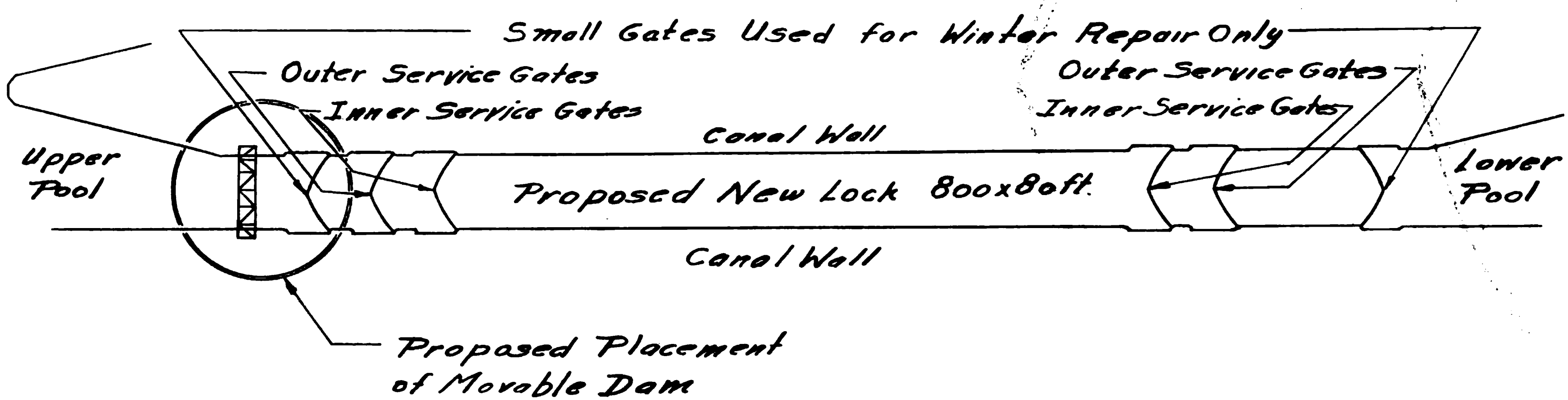
The truss supporting the wickets must be placed on the canal wall, so the elevation of that should be obtained. The elevation is known, but it is given on a different datum plane; so it must be converted to the Low Water Datum Plane of N. Y. The relation between the two as offically recorded is:

Low Water Datum = 603.64 at Canal Datum = 117.37.

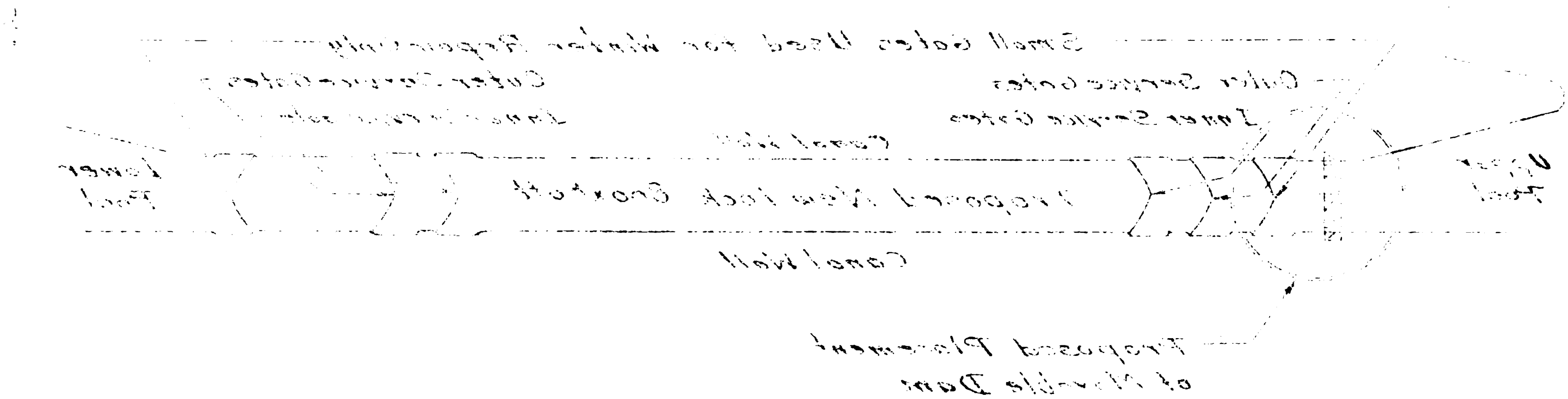
The elevation of the canal wall = 121.00 ft. The difference between the canal wall and the reference point is 121.00 - 117.37 or 3.63 feet. Therefore the elevation of the canal wall with reference to the Low Water Datum is 603.64 ≠ 3.63 or 607.27, and the difference between this and the extreme high water in the upper pool is 607.27 - (601.1 ≠ 4.0) or 2.17 ft.

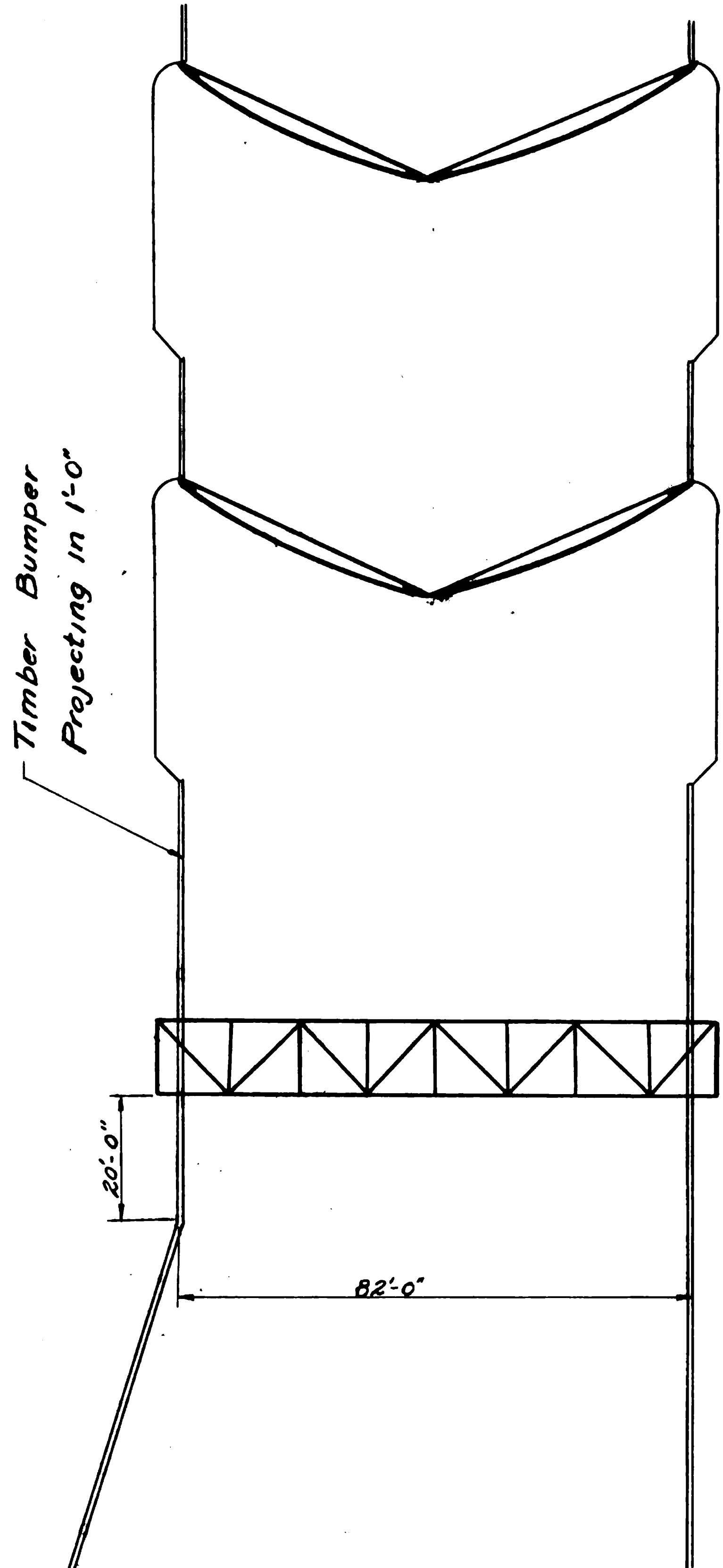
This I believe is all the conversion that needs an explaination, as the rest can readily be seen from a vertical section throught the lock. However, before the vertical section is shown, a general plan of the lock will be shown with the position of the dam indicated. This will give a better understanding of what the conditions are.

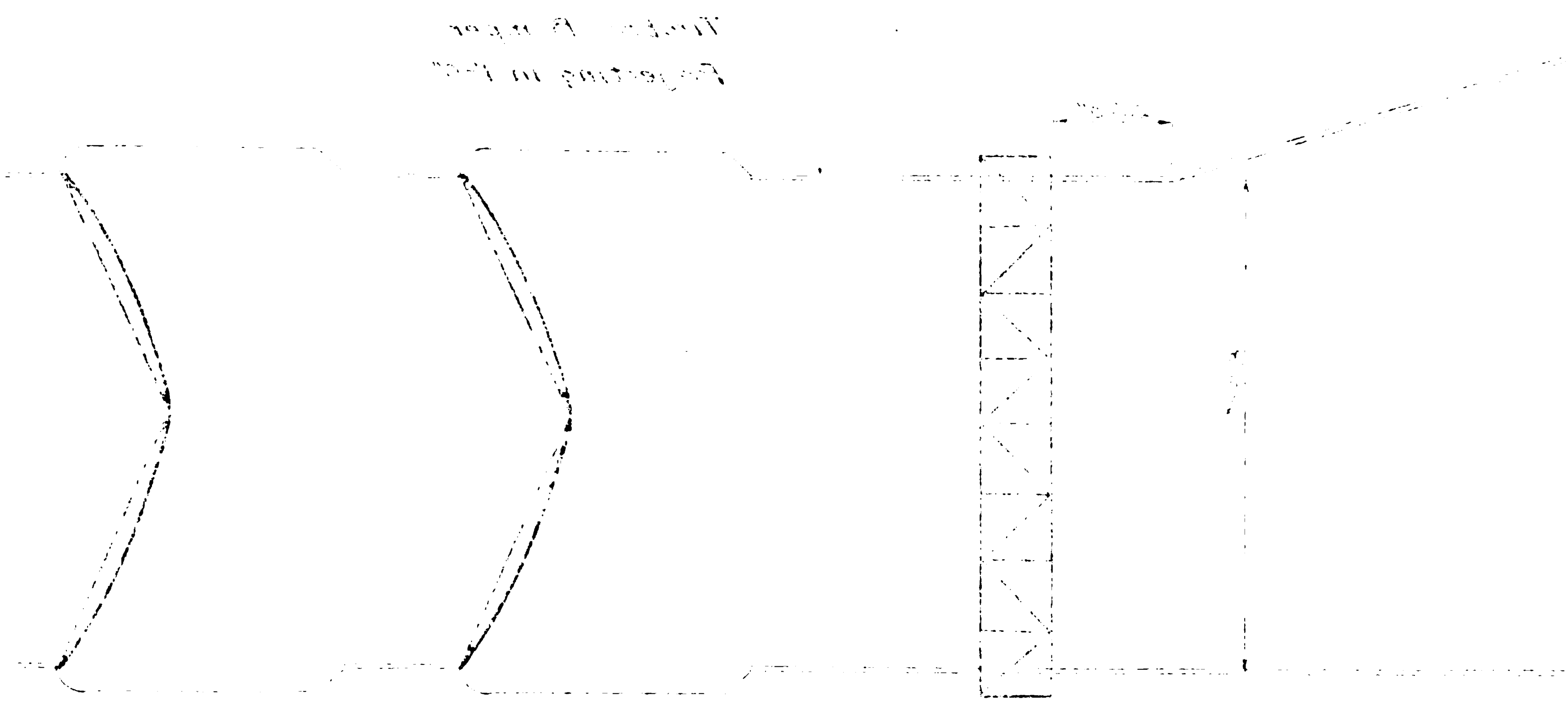
General Plan of New Lock

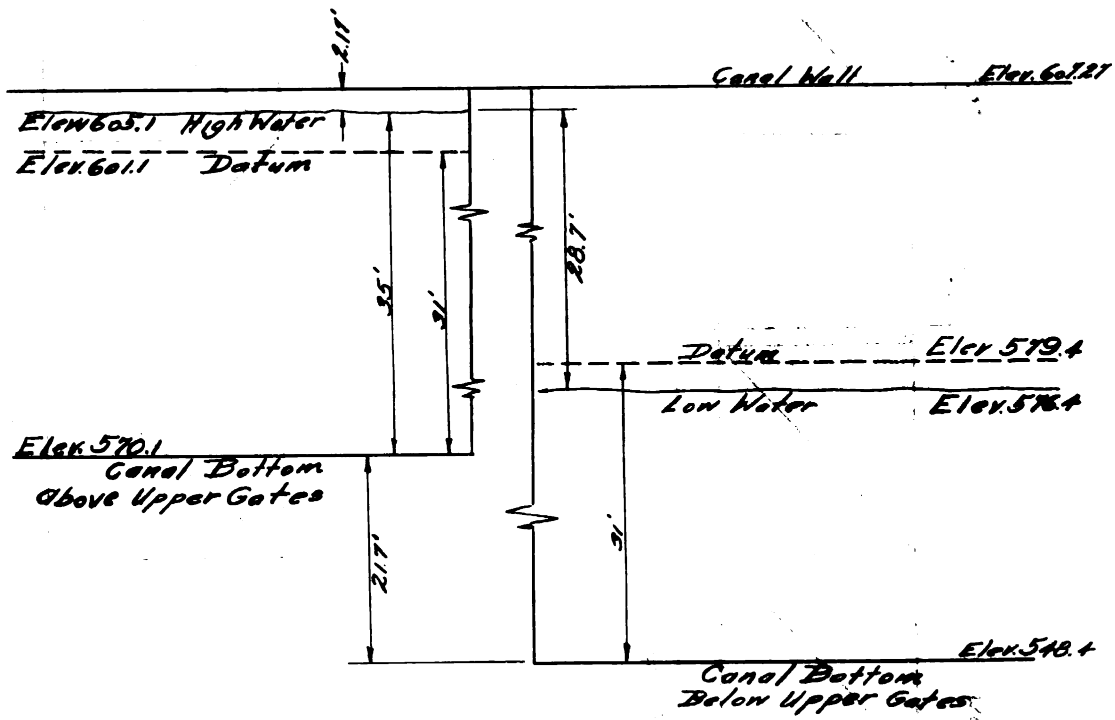


General Plan of New Lock



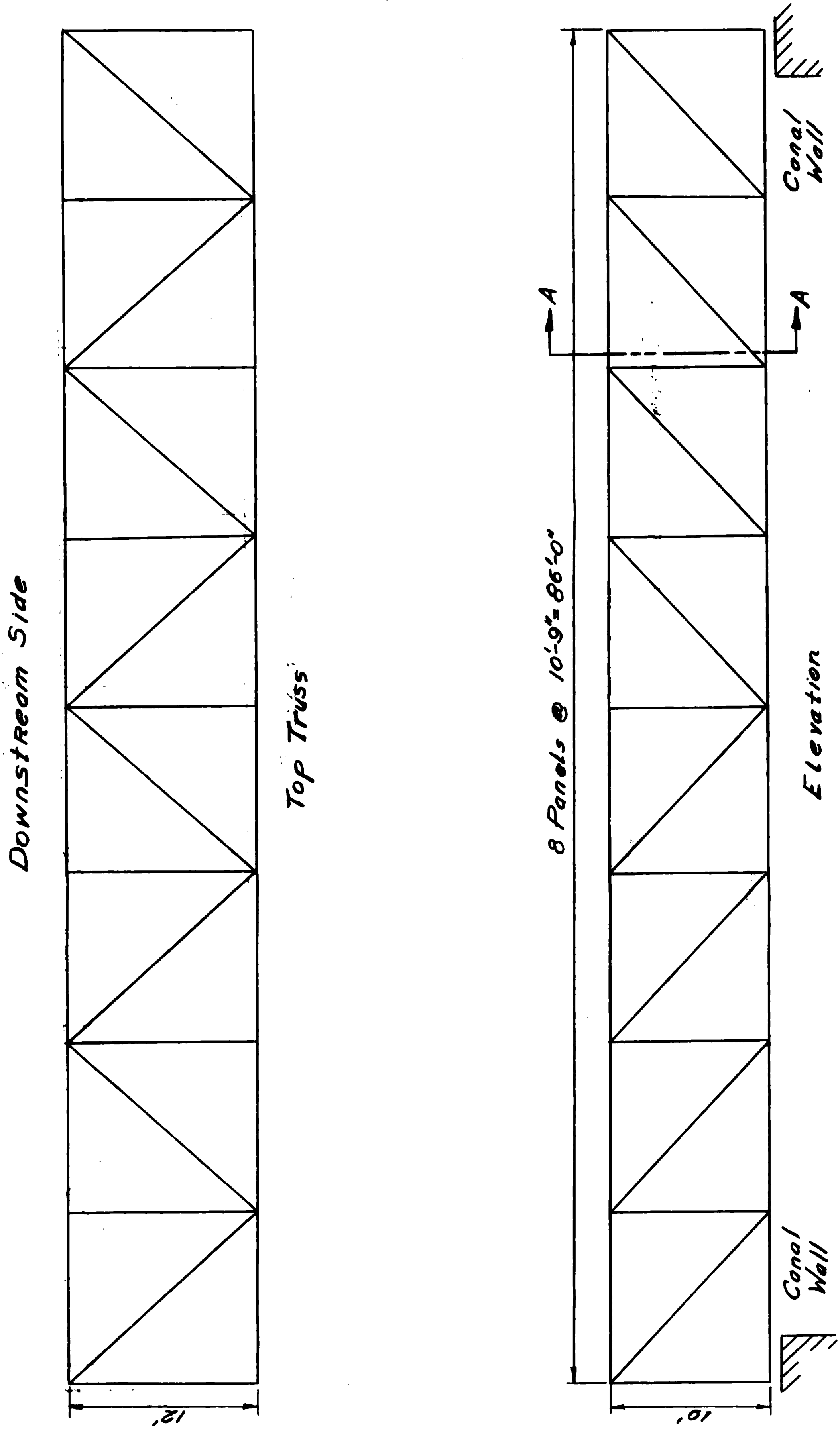

Timber Bumper
Projecting in 1'-0"
20'-0"
82'-0"



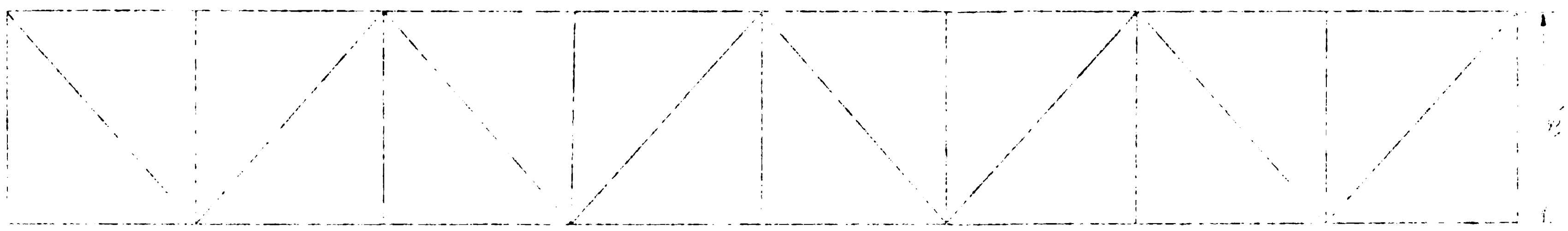


This gives the conditions that the dam must conform with or overcome. These conditions are much more severe than those for most of the movable dams designed. Therefore, this dam must be much stronger than most dams of this type. To overcome these severe conditions I have chosen the type of dam shown on the following two pages for my first trial.

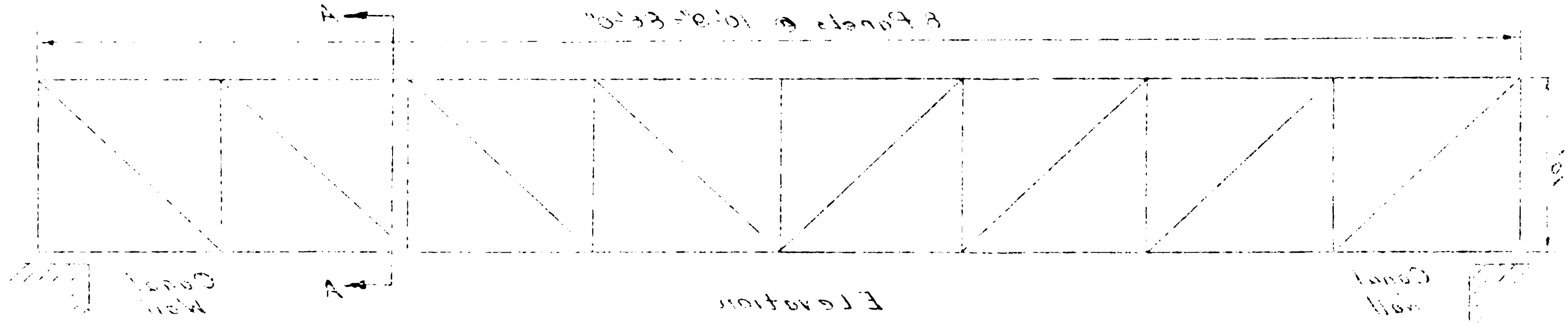
The dam consists of two horizontal trusses held togeather with two vertical trusses. Inside of the box formed by the trusses is placed X bracing at the panel points. Then upon the upstream side of this box is hung sheet piling. This piling cantilevers down into the water to the bottom of the canal. The dam is constructed so that the truss box can be put in place first, and then the piling put in piece by piece until the whole canal is shut off.


Downstream Side
Top Truss
12'
8 Panels @ 10'-9" = 86'-0"
A
A
Canal Wall
Canal Wall
Elevation
10'

Downstream Side

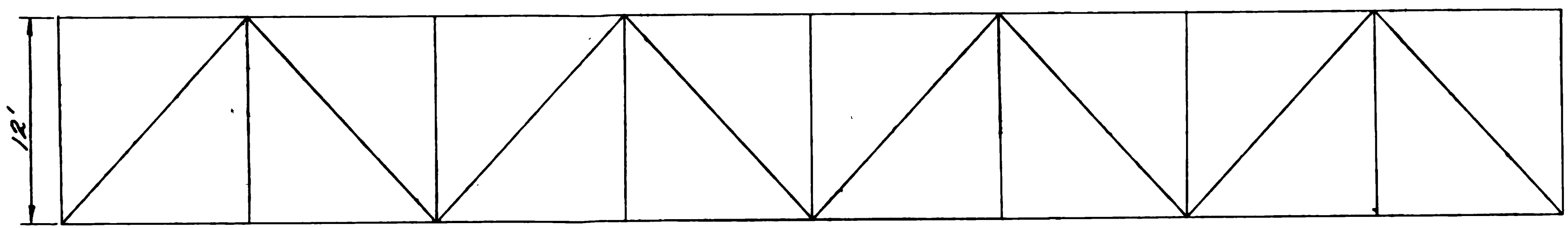


Top Truss



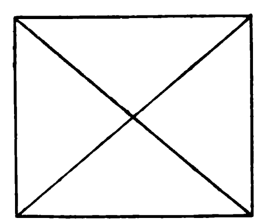
Elevation

Upstream Side



Bottom Truss

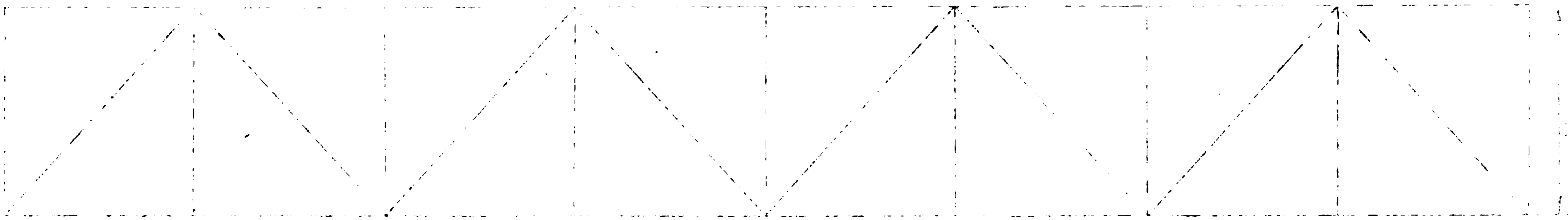
This is the side upon which the wickets are placed or rested



Downstream Side

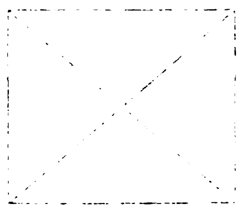
Internal Bracing
Section A-A

Upstream Side



Bottom Truss

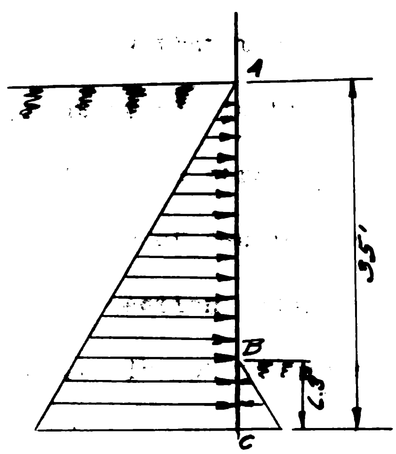
Downstream Side



This is the side upon which the wickets are placed or rested

Internal Bracing
Section A-A

The first thing to analyze is the wickets. However, before the stresses can be found, the pressure exerted by the water must be obtained. The forces of the water acting on the wickets is shown in the diagram below.



P_{ac} = height x width x average water pressure

$P_{ac} = 35 \times 1 \times \frac{(62.4 \times 35 + 0)}{2}$

P_{ac} = 38,220 # / lin. ft.

This force acts at $\frac{35}{3}$ or 11 2/3 ft. from the bottom.

$P_{bc} = 6.3 \times \frac{(62.4 \times 6.3 + 0)}{2} = 1,238$ #/lin. ft.

This force acts at $\frac{6.3}{3}$ or 2.1 ft. from the bottom.

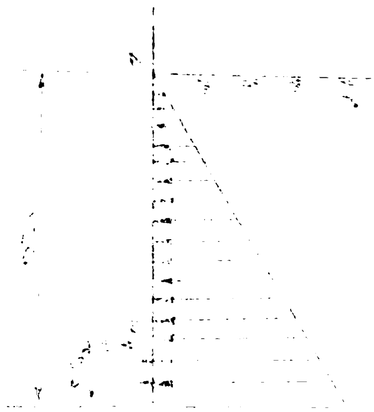
The resultant of these two forces is the difference of the two or 38,220 - 1,238 and is equal to 36,982 #/ lin. ft. The point of application of the resultant is found by Varignon's Theory which states that the moment of the resultant of two forces is equal to the sum of the moments of the two seperate components. Therefore:

$$36,982 X = 38,220 \times 11.67 - 1,238 \times 2.1$$

$$X = 11.987 \text{ ft. approximately from the bottom.}$$

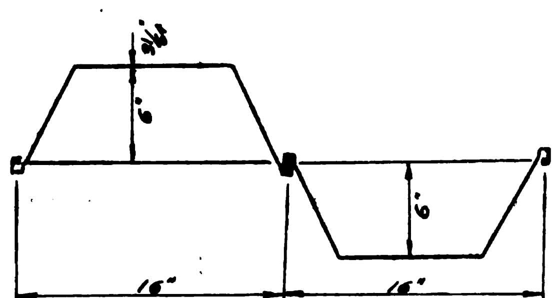
The point of application of the resultant from the top of the water surface equals 35 - 11.987 or 23.013 ft.

The forces acting on the wickets could possibly be worked out by the use of velocities. However, there has as yet been no failure of the gates on any of the American locks at Sault Ste Marie, so the velocities have



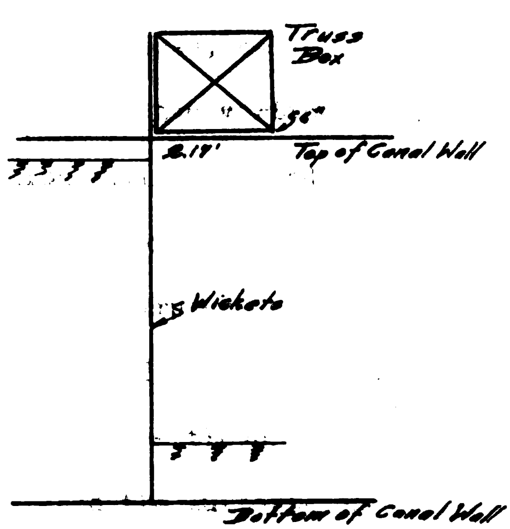
not been determined. The velocities could possibly be worked out by making use of the static head; and then from the velocities so gotten, figure the forces. This would just mean that the head was changed to velocity and velocity back to head, and as there are frictional losses the resulting head would be less. Therefore, there is no point in this type of an analysis.

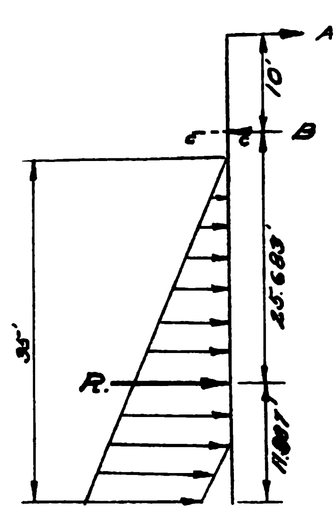
Now that the force created by the head of water is known and the point of application of the force is known, the next thing to be investigated is the stress in the wickets. These are usually made of sheet piling. The type of section that I will try is the Carnegie Section M 110 which has a section modulus of 20.54 in.^{3} per piling or 15.26 in.^{3} per lineal foot. The other properties are shown below.

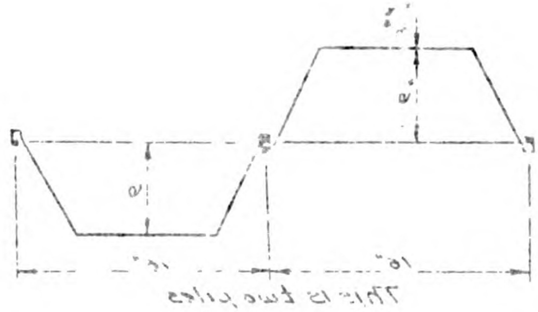


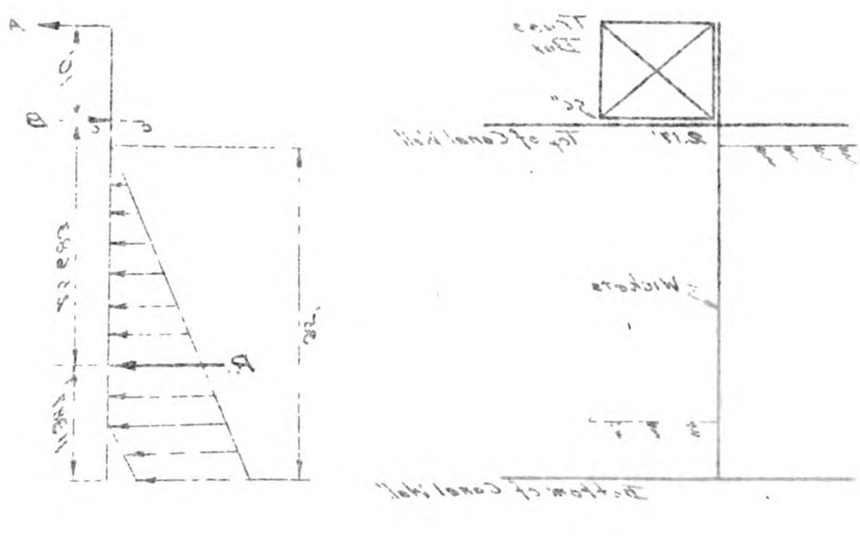
This is two piles

The forces acting on the piling are shown in the two diagrams below.









Taking moments about the section c-c or at the bottom of the truss box, the stress in the wickets can be found as follows:

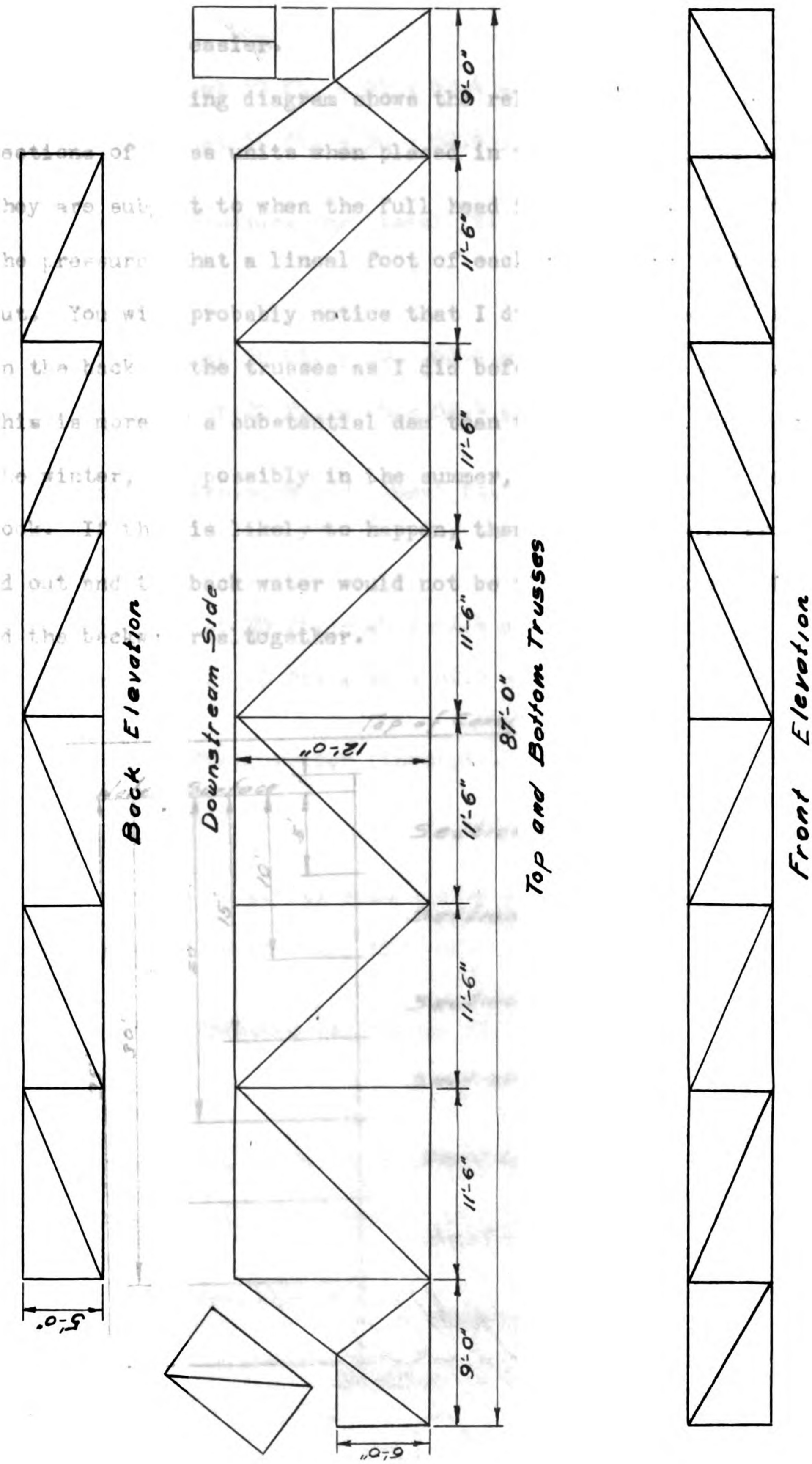
Moment $cc = 36,982 \times 25.683 = 949,808.706$ #-ft. / lin. ft.

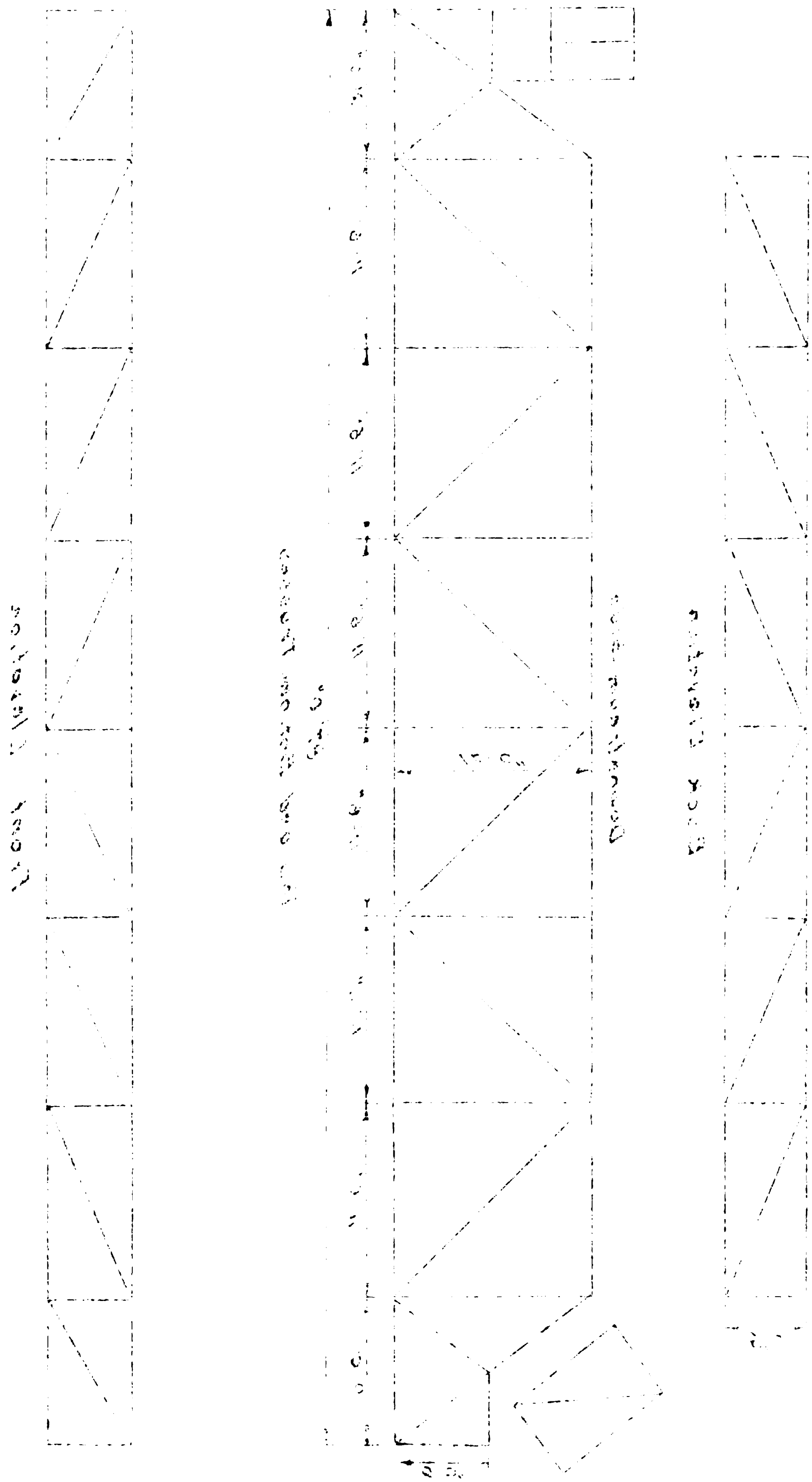
or $949,808.706 \times 12 = 11,397,704$ #-in. / lin. ft.

$$\text{Stress} = \frac{\text{Moment}}{\text{Section Modulus}} = \frac{11,397,704}{15.26} = 746,900 \text{ \# / sq. in.}$$

This shows that with this type of arrangement, the wickets acting as pure cantilevers, the stresses in the wickets would be so large that it would be uneconomical to reinforce them enough to make them safe. Therefore, this type of movable dam should not be used under these conditions. Also with a little bit of computation it can be shown that if the wickets rest on a support on the bottom of the canal, the stresses will still be too large. Then too, if a little more computation is done, which I haven't shown here, it can be shown that if the truss is made so that it will project down into the canal about 13.5 feet, the stresses will still be too large to be economically overcome.

With the same idea of using trusses to overcome the force of the water, I am going to try to drop sets of two horizontal trusses each into the canal. The outline of the trusses are shown on the following page. These two trusses, as can be seen from the drawing, are to be held five feet apart by two vertical trusses. Upon the truss shown as the front elevation will be placed sheet steel to hold back the water. There will have to be six of these five foot sections and one of six foot depth. And as these trusses need no center supports, they can be placed over the canal with a derrick and lowered down into the canal with cables from the sides. Because these sections would have to be slid into the water I have placed the supports on the back side of the trusses. Also I have made provision so that rollers can be used as the supports so that they


Back Elevation
Downstream Side
Top and Bottom Trusses
Front Elevation
9'-0"
11'-6"
11'-6"
11'-6"
11'-6"
11'-6"
11'-6"
9'-0"
87'-0"
12'-0"
5'-0"
6'-0"



can b

secti

they

the

out.

on t

this

t'e w

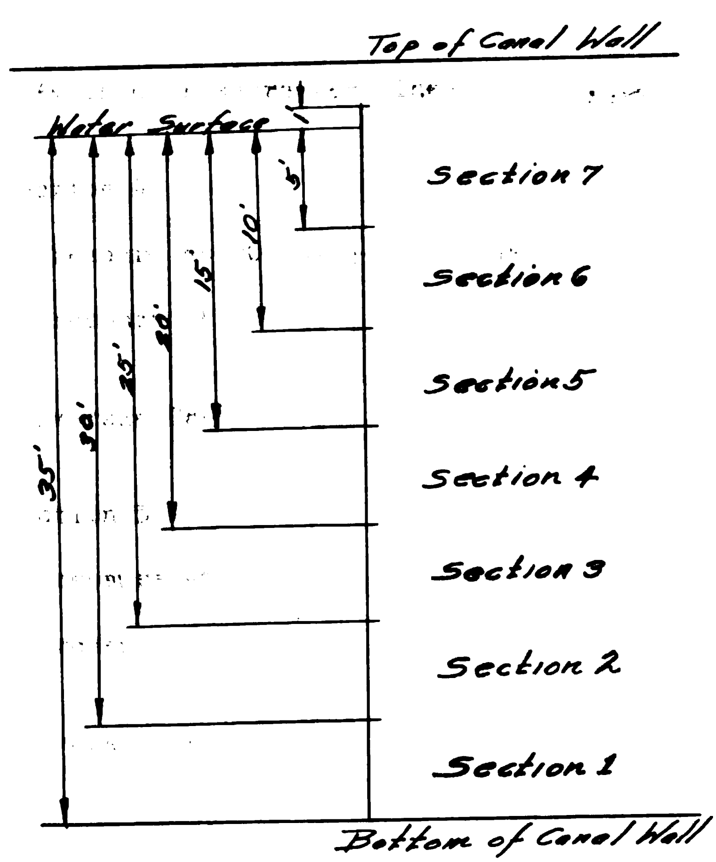
lo k.

ed o

ed th

can be lowered easier.

The following diagram shows the relative positions of the different sections of truss units when placed in the canal and the depths to which they are subject to when the full head is obtained. From this diagram the pressures that a lineal foot of each section is to take was worked out. You will probably notice that I did not use the 6.3 feet of water on the back of the trusses as I did before. The reason for this is; this is more of a substantial dam than the other and may be used more in the winter, and possibly in the summer, for making minor repairs on the lock. If this is likely to happen, then the lock would probably be drained out and the back water would not be there. Therefore, I have disregarded the backwater altogether.



Sect

Fr

Fr

Av

Sect

Pr

Fr

Av

Sec

F

Section 1

Pressure at 35 ft. = 35 x 62.4 = 2184 #

Pressure at 30 ft. = 30 x 62.4 = 1872 #

$$\underline{}\ 4056\ \#$$

Average Pressure per lineal ft. = $\frac{4056}{2}$ x 5 = 10,140 # / lin. ft.

Section 2

Pressure at 30 ft. = 30 x 62.4 = 1872 #

Pressure at 25 ft. = 25 x 62.4 = 1560 #

$$\underline{}\ 3432\ \#$$

Average Pressure per lineal ft. = $\frac{3432}{2}$ x 5 = 8580 # / lin. ft.

Section 3

Pressure at 25 ft. = 25 x 62.4 = 1560 #

Pressure at 20 ft. = 20 x 62.4 = 1248 #

$$\underline{}\ 2808\ \#$$

Average Pressure per lineal ft. = $\frac{2808}{2}$ x 5 = 7020 # / lin. ft.

Section 4

Pressure at 20 ft. = 20 x 62.4 = 1248 #

Pressure at 15 ft. = 15 x 62.4 = 936 #

$$\underline{}\ 2184\ \#$$

Average Pressure per lineal ft. = $\frac{2184}{2}$ x 5 = 5460 # / lin. ft.

Section 5

Pressure at 15 ft. = 15 x 62.4 = 936 #

Pressure at 10 ft. = 10 x 62.4 = 624 #

$$\underline{}\ 1560\ \#$$

Average Pressure per lineal ft. = $\frac{1560}{2}$ x 5 = 3900 # / lin. ft.

Panel

Io

Rc

Section 6

Pressure at 10 ft. = 10 x 62.4 = 624 #

Pressure at 5 ft. = 5 x 62.4 = $\underline{312}$ #
$\quad$ 936 #

Average Pressure per lineal ft. = $\frac{936}{2} \times 5 = 2340$ # / lin. ft.

Section 7

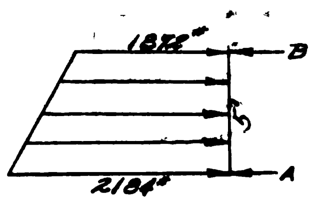
Pressure at 5 ft. = 5 x 62.4 = 312 #

Pressure at top = 0 x 62.4 = $\underline{000}$ #
$\quad$ 312 #

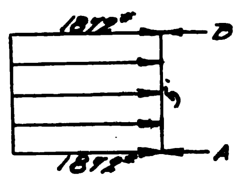
Average Pressure per lineal ft. = $\frac{312}{2} \times 5 = 780$ # / lin. ft.

From these loads per lineal ft. for each section are found the panel loads for each section, that is for each truss in each section.

Section 1

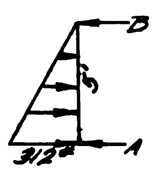


This diagram show how the load is distributed over the 5 ft. section. To obtain A and B, the load was broken up as shown.



A + B = 5 x 1872 # also A = B

2A = 9360 or A = 4680 # also B = 4680 #



Sum $M_B = 0$

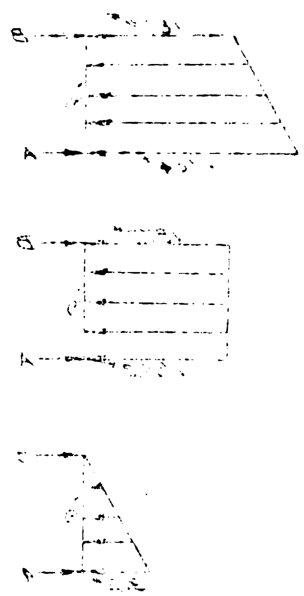
$5A = \frac{312}{2} \times 5 \times \frac{2}{3} \times 5$ or A = 520 #

Sum $M_A = 0$

$5B = \frac{312}{2} \times 5 \times \frac{1}{3} \times 5$ or B = 260 #

Load on top truss -- per long panel

Reaction B = 4680 + 260 = 4940 # / lin. ft.



Panel load = 4940 x 11.5 = 56,310 #

Load on top truss -- per short panel

Panel load = 4940 x 9 = 44,460 #

Load on lower truss -- per long panel

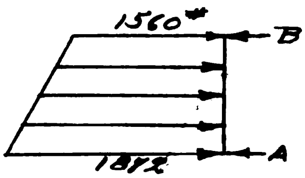
Reaction A = 4680 + 520 = 5,200 # / lin. ft.

Panel load = 5,200 x 11.5 = 59,800 #

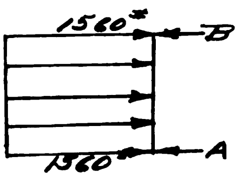
Load on lower truss -- per short panel

Panel load = 5,200 x 9 = 91,260 #

Section 2

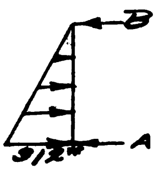


To obtain the reaction A and B, the load was broken up the same as in Section 1.



A + B = 5 x 1650 #   also A = B

2A = 7800 or A = 3900 # also B = 3900 #



Sum. $M_B = 0$

$5A = \frac{312}{2} \times 5 \times \frac{2}{3} \times 5$ or A = 520 #

Sum. $M_A = 0$

$5B = \frac{312}{2} \times 5 \times \frac{1}{3} \times 5$ or B = 260 #

Load on top truss -- per long panel

Reaction B = 3900 + 260 = 4,160 # / lin. ft.

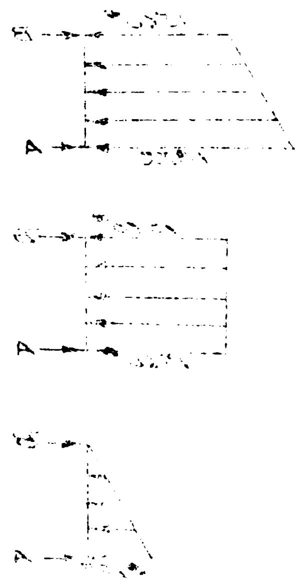
Panel load = 4,160 x 11.5 = 47,840 #

Load on top truss -- per short panel

Panel load = 4,160 x 9 = 37,440 #

Load on lower truss -- per long panel

Reaction A = 3900 + 520 = 4,420 # / lin. ft.



To

up. Fro

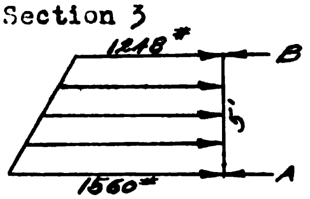
ft. eac

Panel load = 4,420 x 11.5 = 50,830 #

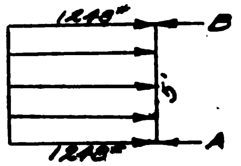
Load on lower truss -- per short panel

Panel load = 4,420 x 9 = 39,780 #

Section 3



To obtain the reactions A and B, the load was broken up the same as in Sections 1 and 2.



$A + B = 5 \times 1,248$ # also $A = B$

$2A = 6,240$ #

$A = 3,120$ # and $B = 3,120$ #

Sum. $M_B = 0$

$5\ A = \frac{312}{2} \times 5 \times \frac{2}{3} \times 5$ or $A = 520$ #/lin. ft.

Sum. $M_A = 0$

$5\ B = \frac{312}{2} \times 5 \times \frac{1}{3} \times 5$ or $B = 260$ #/lin. ft.

Load on top truss -- per long panel

Reaction B = 3120 + 260 = 3380 # / lin. ft.

Panel load = 3,380 x 11.5 = 38,870 #

Load on top truss -- per short panel

Panel load = 3,380 x 9 = 30,420 #

Load on lower truss -- per long panel

Reaction A = 3,120 - 520 = 3,640 # / lin. ft.

Panel load = 3,640 x 11.5 = 41,860 #

Load on lower truss -- per short panel

Panel load = 3,640 x 9 = 32,760 #

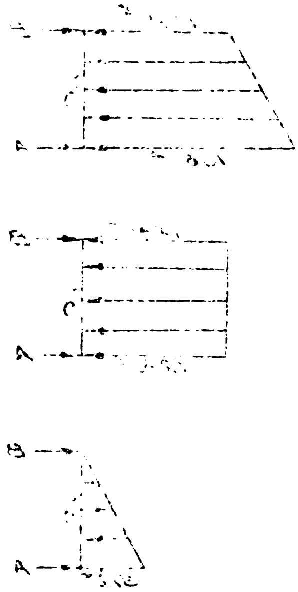
To simplify the work of figuring the panel loads, a ratio was set up. From this ratio we find that the reaction B reduces by 780 # / lin. ft. each time and the reaction A reduces by the same amount, 780 # / lin.

ft.

panel

the

secti



ft. Also the long panel loads reduce by 8,970 # each time and the short panel loads reduce by 7,020 # each time. With the use of these ratios the rest of the truss panel loads can be worked out. That is, all but section 7, which is a special case as it is six feet deep.

Section 4

Reaction B = 3,380 - 780 = 3600 # / lin. ft.

Reaction A = 3,640 - 780 = 2,860 # / lin. ft.

Load on top truss -- per long panel

Panel load = 38,870 - 8,970 = 29,900 #

Load on top truss -- per short panel

Panel load = 30,420 - 7,020 = 23,400 #

Load on lower truss -- per long panel

Panel load = 41,860 - 8,970 = 32,890 #

Load on lower truss -- per short panel

Panel load = 32,760 - 7,020 = 25,740 #

Section 5

Reaction A = 2,860 - 780 = 2,080 # / lin. ft.

Reaction B = 2,600 - 780 = 1,820 # / lin. ft.

Load on top truss -- per long panel

Panel load = 29,900 - 8,970 = 20,930 #

Load on top truss -- per short panel

Panel load = 23,400 - 7,020 = 16,380 #

Load on lower truss -- per long panel

Panel load = 32,890 - 8,970 = 23,920 #

Load on lower truss -- per short panel

Panel load = 25,740 - 7,020 = 18,720 #

Section 6

Reaction A = 2,080 - 780 = 1,300 # / lin. ft.

Reaction B = 1,820 - 780 = 1,040 # / lin. ft.

Load on top truss -- per long panel

Panel load = 20,930 - 8,970 = 11,960 #

Load on top truss -- per short panel

Panel load = 16,380 - 7,020 = 9,360 #

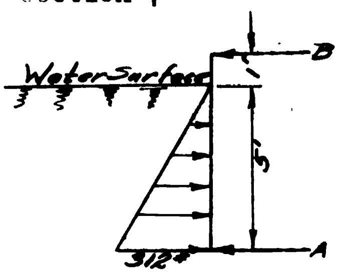
Load on lower truss -- per long panel

Panel load = 23,920 - 8,970 = 14,950 #

Load on lower truss -- per short panel

Panle load = 18,720 - 7,020 = 11,700 #

Section 7



This section must be worked seperate from the rest of the others because it is six feet in depth, and is only holding back five feet of water.

Sum. $M_B = 0$

$$6\,A = \frac{312}{2} \times 5 \times \left(\frac{2 \times 5}{3} + 1\right)$$

$$A = 563\ 1/3\ \#\ /\ \text{lin. ft.}$$

Sum. $M_A = 0$

$$6\,B = \frac{312}{2} \times 5 \times \frac{1}{3} \times 5$$

$$B = 216\ 2/3\ \#\ /\ \text{lin. ft.}$$

Load on top truss -- per long panel

Panel load $= 216\ 2/3 \times 11\ 1/2 = \frac{14,950}{6} = 2,491\ 2/3$ #

Load on top truss -- per short panel

Panel load = 216 2/3 x 9 = 1,950 #

Load on lower truss -- per long panel

Panel load = 563 1/3 x 11 1/2 = $\frac{38,870}{6}$ = 6,478 1/3 #

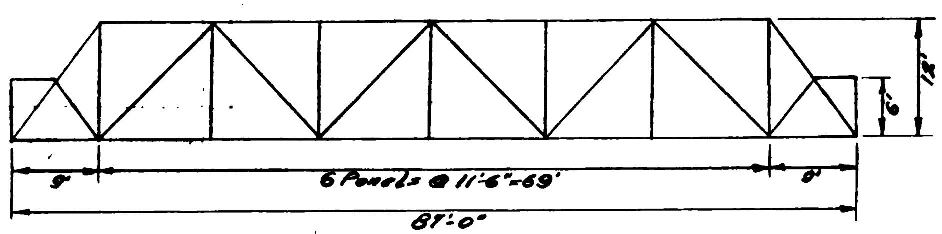
Load on lower truss -- per short panel

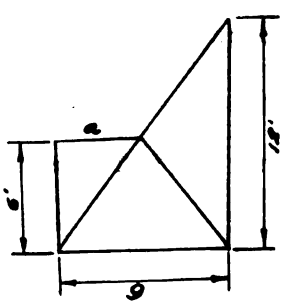
Panel load = 563 1/3 x 9 = 5,070 #

Now that the panel loads have been obtained, the exact dimensions of the diagonals of the truss must be worked out so that the stresses or loads in the truss members can be computed. These loads will be computed algebrically and also graphically. The graphic solution is used only as a check upon the algebric work.

The graphic solutions are drawn to a fairly small scale. This was done because the stresses in the members are so large that in the larger stressed ones the difference of 500 pounds will not mean much in computing the area of steel needed. Therefore, I believe that the small scale is justified.

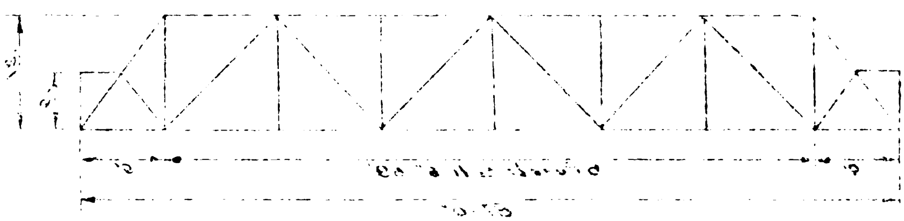
Below are shown the computations for the lengths of the members.

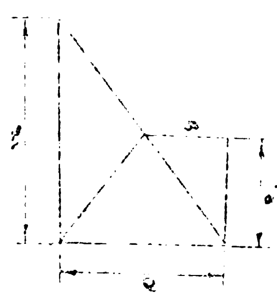




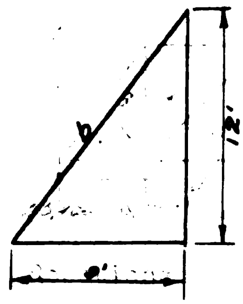
This diagram shows the end diagonals that must be figured. This is most easily done by breaking it down into simple triangles.

The side "a" can be shown to equal 4.5 feet by simple geometry, so it will not be



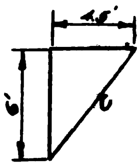


proven here.



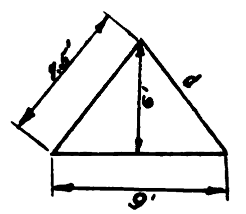
$$b^2 = 9^2 + 12^2$$

$$b = \sqrt{81 + 144} = 15 \text{ ft.}$$



$$c^2 = 4.5^2 + 6^2$$

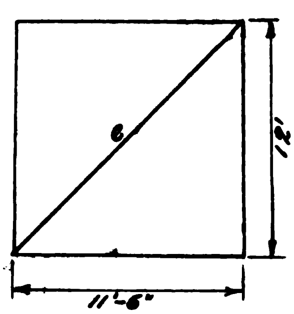
$$c = \sqrt{20.25 + 36} = 7.5 \text{ ft.}$$



The diagonal "d" really is the same as the diagonal "c", but turned around in a slightly different position. Therefore:

$d = 7.5$ ft.

The computations for the main diagonal is the same for all of them so only one is shown here.

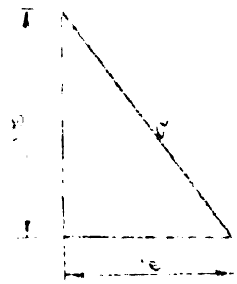


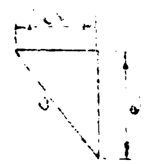
$$e^2 = 12^2 + 11.5^2$$

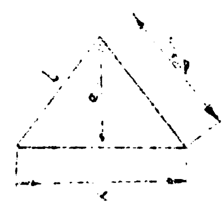
$$e = \sqrt{144 + 132.25} = 16.62074 \text{ ft.}$$

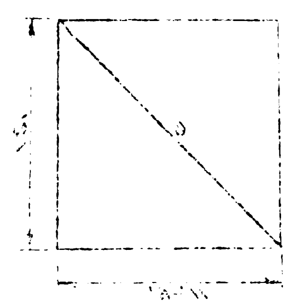
This is a very awkward number to use, so the last three figures will be dropped off. This can be done as it will only cause an error of less than one tenth of one per cent.

The algebric solution of one truss is show below. This same method was run through for each truss, but as it is exactly the same, except for the answers and figures, only the answers are given.

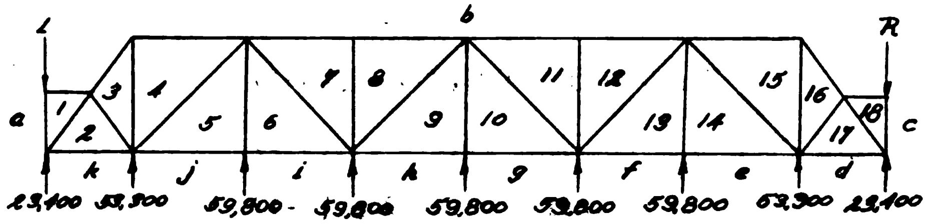








Section 1 -- Bottom truss



Reactions

Sum. $M_L = 0$

$87\,R - (23{,}400 \times 0 + 53{,}300 \times 9 + 59{,}800 \times 20.5 + 59{,}800 \times 30 + 59{,}800 \times 43.5 + 59{,}800 \times 55 + 59{,}800 \times 66.5 + 53{,}300 \times 78 + 23{,}400 \times 87) = 0$

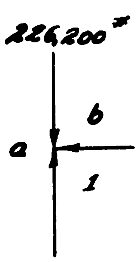
$R = 226{,}200$ #

Sum. $M_R = 0$

$87\,L - (23{,}400 \times 0 + 53{,}300 \times 9 + 59{,}800 \times 20.5 + 59{,}800 \times 30 + 59{,}800 \times 43.5 + 59{,}800 \times 55 + 59{,}800 \times 66.5 + 53{,}300 \times 78 + 23{,}400 \times 87) = 0$

$L = 226{,}200$ #

Joint a-b-1



Sum. $F_h = 0$

$(1-b) - 0 = 0$

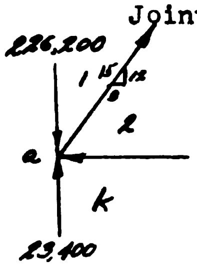
$(1-b) = 0$ #

Sum. $F_v = 0$

$(1-a) - 226{,}200 = 0$

$(1-a) = 226{,}200$ # comp.

Joint a-1-2-k



Sum. $F_v = 0$

$$(1-2)_v - 226{,}200 + 23{,}400 = 0$$

$$(1-2)_v = 202{,}800 \qquad (1-2) = 202{,}800\,\frac{15}{12}$$

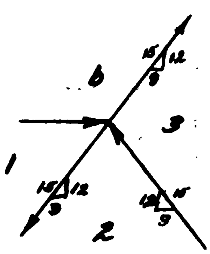
$(1-2) = 253{,}500$ # Ten.

Sum. $F_h = 0$

$$(1-2)_h - (2-k) = 0$$

$(2-k) = 253{,}500\,\dfrac{9}{15} = 152{,}100$ # Comp.

Joint b-3-2-1



Sum. $F_h = 0$

$$(1-b) - (1-2)_h + (3-b)_h - (2-3)_h = 0$$

$$0 - 152{,}100 + (3-b)\frac{9}{15} - (2-3)\frac{9}{15} = 0$$

$$(3-b) = \left[(2-3)\frac{9}{15} + 152{,}100\right]\frac{15}{9}$$

Sum. $F_v = 0$

$$-(1-2)_v + (3-b)_v + (2-3)_v = 0$$

$$-202{,}800 + (3-b)\,\frac{12}{15} + (2-3)\,\frac{12}{15} = 0$$

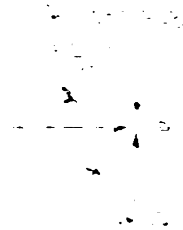
$$-202{,}800 + \left[(2-3)\frac{9}{15} + 152{,}100\right]\frac{15}{9} \times \frac{12}{15} + (2-3)\frac{12}{15} = 0$$

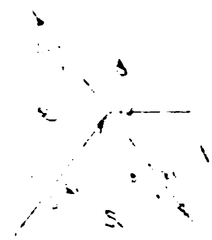
$$(2-3)\frac{12}{15} + (2-3)\,\frac{12}{15} = 0$$

$(2-3) = 0$ #

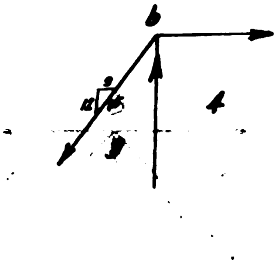
$$(3-b) = 0 + 152{,}100 \times \frac{15}{9}$$

$(3-b) = 253{,}500$ # Ten.





Joint b-4-3



Sum. $F_v = 0$

$-(2\text{-}b)_v + (3\text{-}4) = 0$

$(3\text{-}4) = (3\text{-}b)_v = 253{,}500 \ \frac{12}{15} = 2\text{-}2{,}800$ # Comp.

Sum. $F_H = 0$

$-(3\text{-}b)_h + (4\text{-}b) = 0$

$(4\text{-}b) = (3\text{-}b)_h = 253{,}500 \ \frac{9}{15} = 152{,}100$ # Ten.

Joint 2-3-4-5-j-k

Sum. $F_v = 0$

$(4\text{-}5)_v - (3\text{-}4) + 53{,}300 = 0$

$(4\text{-}5)_v = 2\text{-}2{,}800 - 53{,}300 = 149{,}500$

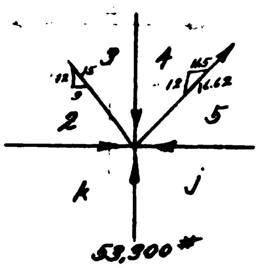
$(4\text{-}5) = 149{,}500 \ \frac{16.62}{12} = 207{,}058$ # Ten.

Sum. $F_h = 0$

$(2\text{-}k) + (4\text{-}5) - (5\text{-}j) = 0$

$(5\text{-}j) = 152{,}100 + 207{,}058 \ \frac{11.5}{16.62}$

$(5\text{-}j) = 295{,}371$ # Comp.



Joint 5-6-i-j

Sum. $F_v = 0$

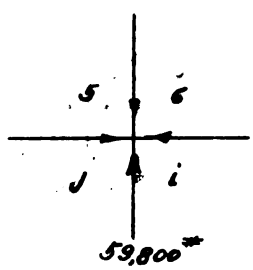
$(i\text{-}j) - (5\text{-}6) = 0$

$(5\text{-}6) = 59{,}800$ # Comp.

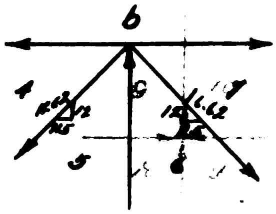
Sum. $F_h = 0$

$(5\text{-}j) - (6\text{-}i) = 0$

$(6\text{-}i) = 295{,}371$ # Comp.



Joint 4-5-6-7-b



Sum. $F_v = 0$

$(6\text{-}7)_v + (5\text{-}6) - (4\text{-}5)_v = 0$

$(6\text{-}7)_v = 149{,}500 - 59{,}800 = 89{,}700$

$(6\text{-}7) = 89{,}700 \dfrac{16.62}{12} = 124{,}235$ # Comp.

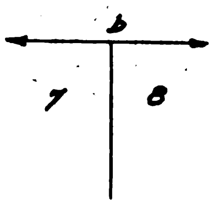
Sum. $F_h = 0$

$(7\text{-}b) - (6\text{-}7)_h - (4\text{-}5)_h = 0$

$(7\text{-}b) = 152{,}100 + 143{,}271 + 124{,}235 \dfrac{11.5}{16.62}$

$(7\text{-}b) = 381{,}333$ # Ten.

Joint 7-8-b



Sum. $F_v = 0$

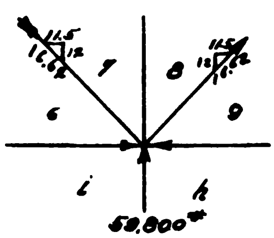
$(7\text{-}8) = 0$

Sum. $F_h = 0$

$(8\text{-}b) - (7\text{-}b) = 0$

$(8\text{-}b) = 381{,}333$ # Ten.

Joint 6-7-8-9-h-i



Sum. $F_v = 0$

$(8\text{-}9)_v - (6\text{-}7)_v + 59{,}800 = 0$

$(8\text{-}9)_v = 89{,}700 - 59{,}800 = 29{,}900$

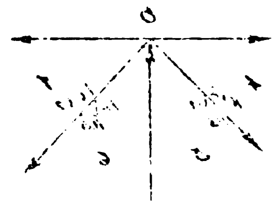
$(8\text{-}9) = 29{,}900 \dfrac{16.62}{12} = 41{,}412$ # Ten.

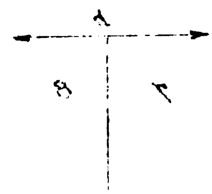
Sum. $F_h = 0$

$(6\text{-}i) + (6\text{-}7)_h + (8\text{-}9)_h - (9\text{-}h) = 0$

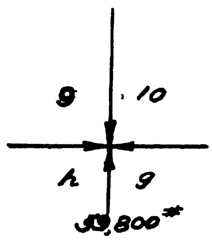
$(9\text{-}h) = 295{,}371 + 85{,}962 + 41{,}412 \dfrac{11.5}{16.62}$

$(9\text{-}h) = 409{,}988$ # Comp.





Joint 9-10-g-h



Sum. $F_h = 0$

$(9\text{-}h) - (10\text{-}g) = 0$

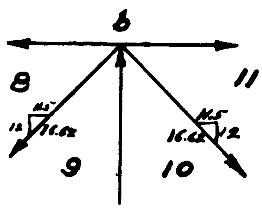
$(10\text{-}g) = 409{,}988$ # Comp.

Sum. $F_v = 0$

$(g\text{-}h) - (9\text{-}10) = 0$

$(9\text{-}10) = 59{,}800$ # Comp.

Joint 8-9-10-11-b



Sum. $F_v = 0$

$(9\text{-}10) - (8\text{-}9)_v - (10\text{-}11)_v = 0$

$(10\text{-}11)_v = (9\text{-}10) - (8\text{-}9)_v$

$(10\text{-}11)_v = 59{,}800 - 29{,}900 = 29{,}900$

$(10\text{-}11) = 29{,}900 \dfrac{16.62}{12} = 41{,}412$ # Ten.

Sum. $F_h = 0$

$(11\text{-}b) + (10\text{-}11)_h - (8\text{-}9)_h + (8\text{-}b) = 0$

$(11\text{-}b) = -41{,}412 \dfrac{11.5}{16.62} + 41{,}412 \dfrac{11.5}{16.62} + 381{,}333$

$(11\text{-}b) = 381{,}333$ # Ten.

	Type of Stress	Section 1 Top Truss Stress	Section 2 Bottom Truss Stress	Section 2 Top Truss Stress	Section 3 Bottom Truss Stress	Section 3 Top Truss Stress
(1-b)		0	0	0	0	0
(1-a)	Comp.	214,890	192,270	180,680	158,340	147,030
(1-2)	Ten.	240,825	215,475	202,800	177,450	164,775
(2-k)	Comp.	144,495	129,285	121,680	106,095	89,865
(2-3)		0	0	0	0	0
(3-b)	Ten.	240,825	215,475	202,800	177,450	164,775
(3-4)	Comp.	192,660	172,380	162,240	141,960	131,820
(4-b)	Ten.	144,495	129,285	121,680	106,095	89,865
(4-5)	Ten.	196,621	175,999	165,646	144,857	134,587
(5-j)	Comp.	280,602	251,065	236,297	206,385	191,991
(5-6)	Comp.	56,810	50,830	47,840	41,860	38,870
(6-i)	Comp.	280,602	251,065	236,297	206,385	191,991
(6-7)	Comp.	118,023	105,599	99,508	86,964	80,752
(7-b)	Ten.	362,266	324,762	305,067	266,888	247,867
(7-8)		0	0	0	0	0
(8-b)	Ten.	362,266	324,762	305,067	266,888	247,867
(8-9)	Ten.	39,341	35,200	33,129	38,983	26,917
(9-h)	Comp.	389,487	349,072	327,990	286,619	266,492
(9-10)	Comp.	56,810	50,830	47,840	41,860	38,870
(10-g)	Comp.	389,487	349,072	327,990	286,619	266,492
(10-11)	Ten.	39,341	35,200	33,129	28,983	26,917
(11-b)	Ten.	362,266	324,762	305,067	266,888	247,867

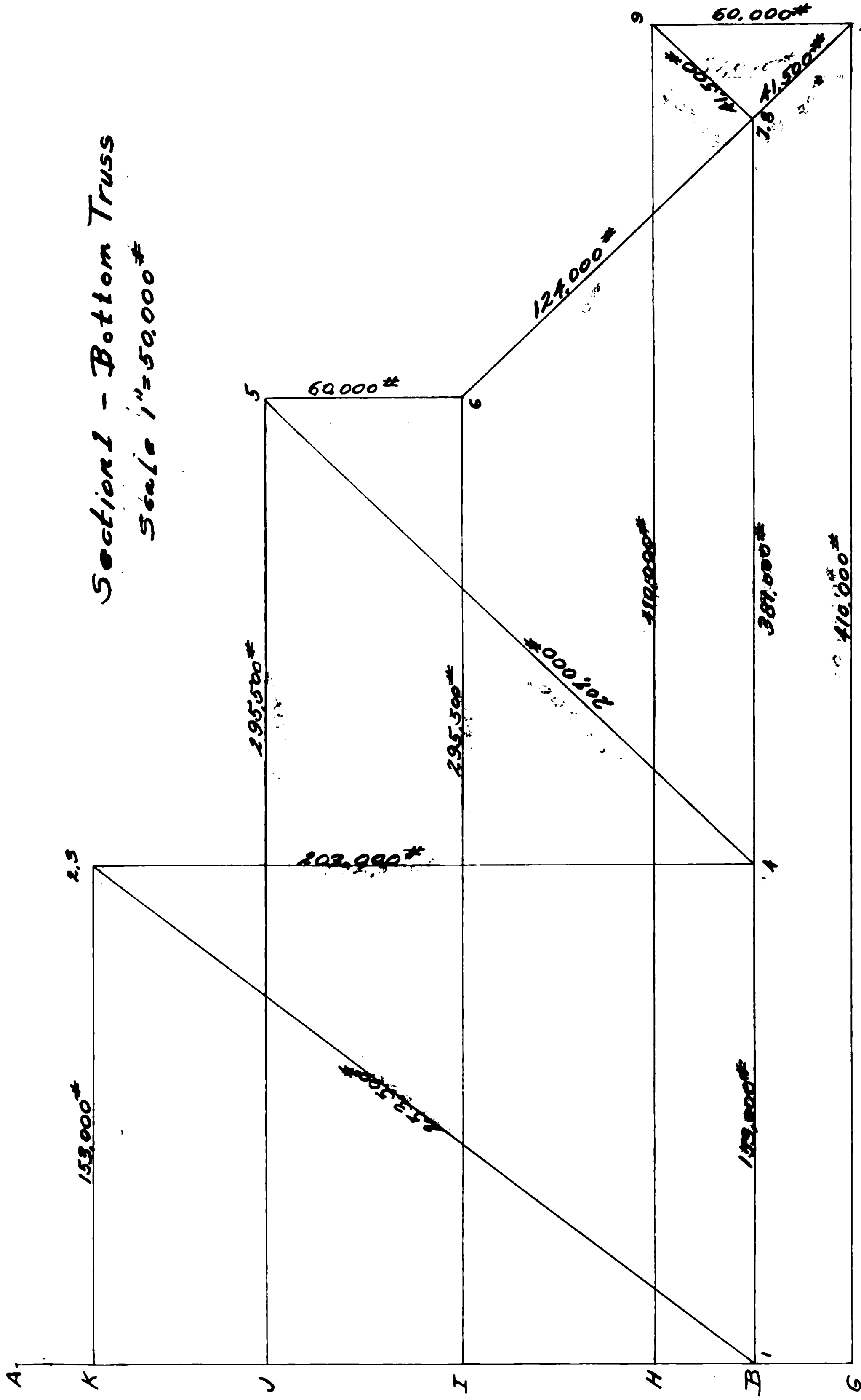
Member	Type of Stress	Section 4 Bottom Truss Stress	Section 4 Top Truss Stress	Section 5 Bottom Truss Stress	Section 5 Top Truss Stress
(1-b)		0	0	0	0
(1-a)	Comp.	124,410	113,100	90,480	79,170
(1-2)	Ten.	111,540	126,750	101,400	88,725
(2-k)	Comp.	83,605	76,050	60,840	53,235
(2-3)		0	0	0	0
(3-b)	Ten.	139,425	126,750	101,400	88,725
(3-4)	Comp.	111,540	101,400	81,120	70,930
(4-b)	Ten.	83,605	76,050	60,840	53,235
(4-5)	Ten.	113,882	103,529	82,823	72,470
(5-j)	Comp.	162,405	147,685	118,148	103,380
(5-6)	Comp.	32,890	29,900	23,920	20,930
(6-i)	Comp.	162,405	147,635	118,148	103,380
(6-7)	Comp.	68,327	62,117	49,794	43,482
(7-b)	Ten.	209,703	190,600	152,533	133,457
(7-8)		0	0	0	0
(8-b)	Ten.	209,703	190,600	152,533	133,457
(8-9)	Ten.	22,776	20,706	16,565	14,482
(9-h)	Comp.	225,464	204,927	163,995	143,479
(9-10)	Comp.	32,890	29,900	23,920	20,930
(10-g)	Comp.	225,464	204,927	163,995	143,479
(10-11)	Ten.	22,776	20,706	16,565	14,482
(11-b)	Ten.	209,703	190,600	152,533	133,457

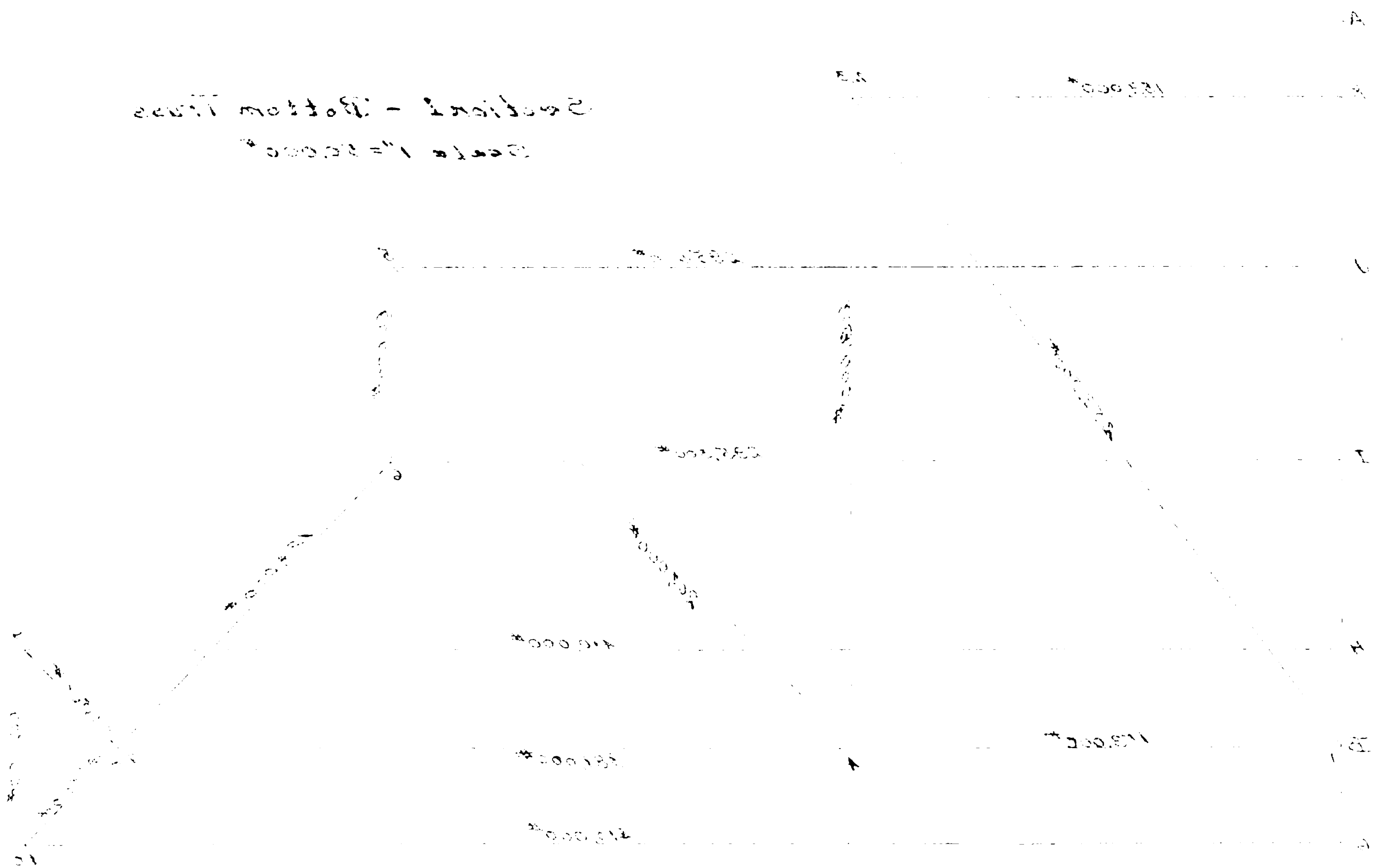
Member	Type of Stress	Section 6 Bottom Truss Stress	Section 6 Top Truss Stress	Section 7 Bottom Truss Stress	Section 7 Top Truss Stress
(1-b)		0	0	0	0
(1-a)	Comp.	56,550	45,240	24,504	9,426
(1-2)	Ten.	63,375	50,700	27,461	10,464
(2-k)	Comp.	38,025	30,420	16,477	6,338
(2-3)		0	0	0	0
(3-b)	Ten.	63,375	50,700	27,461	10,464
(3-4)	Comp.	50,700	40,560	21,969	8,541
(4-b)	Ten.	38,025	30,420	16,477	6,338
(4-5)	Ten.	51,764	41,412	22,430	8,627
(5-j)	Comp.	73,843	59,074	31,997	12,308
(5-6)	Comp.	14,950	11,960	6,478	2,492
(6-i)	Comp.	73,843	59,074	31,997	12,308
(6-7)	Comp.	31,059	24,847	13,458	5,177
(7-b)	Ten.	95,334	76,266	41,309	15,890
(7-8)		0	0	0	0
(8-b)	Ten.	95,334	76,266	41,309	15,890
(8-9)	Ten.	10,353	8,282	4,486	1,726
(9-h)	Comp.	102,501	81,997	44,413	17,084
(9-10)	Comp.	14,950	11,960	6,478	2,492
(10-g)	Comp.	102,501	81,997	44,413	17,084
(10-11)	Ten.	10,353	8,282	4,486	1,726
(11-b)	Ten.	95,334	76,266	41,309	15,890

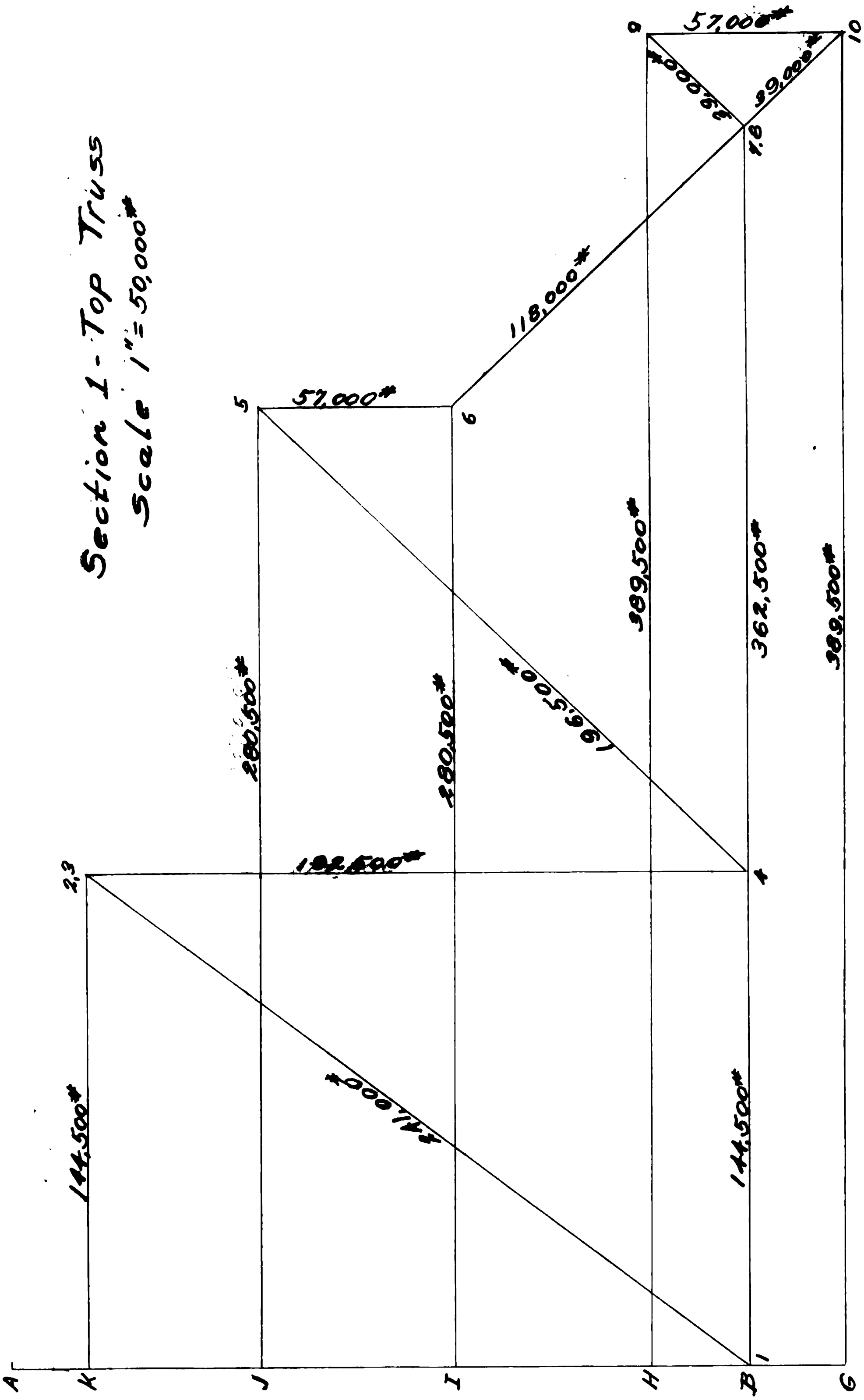
In section No. 7 the panel loads on the longer panels came out to fractions of a pound. These fractions were droped off if they were less than a half of a pound and added if over a half. This was done as the small fractions would not effect the stresses enough to make them worth while to carry in the computations.

On the following fourteen pages is shown the graphical solution of the trusses, as was explained previously. Along with the stress diagrams is shown one diagram of the truss. This diagram was used to make all of the stress diagrams.

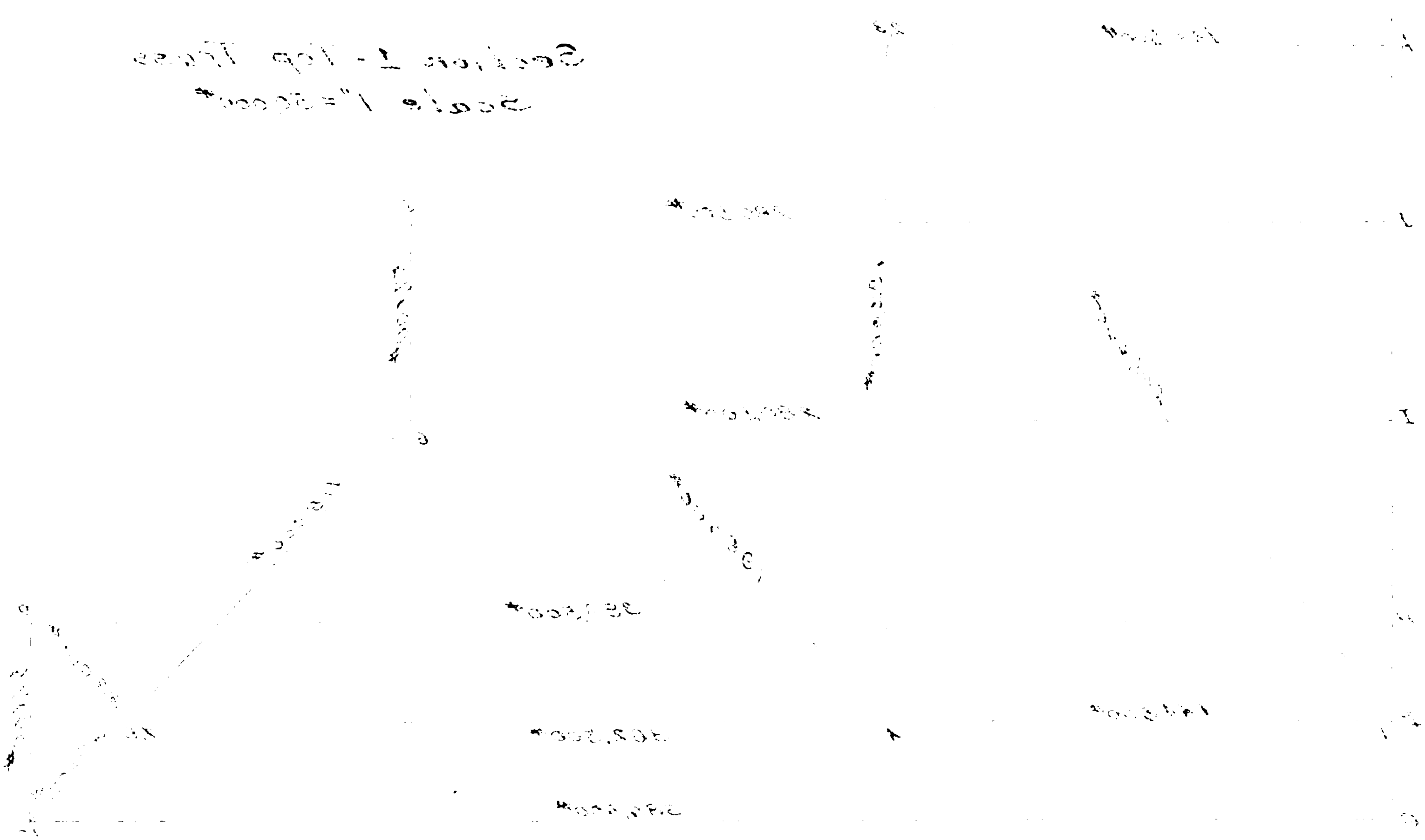
Section 1 - Bottom Truss
Scale 1" = 50,000#




Section I - Bottom Truss

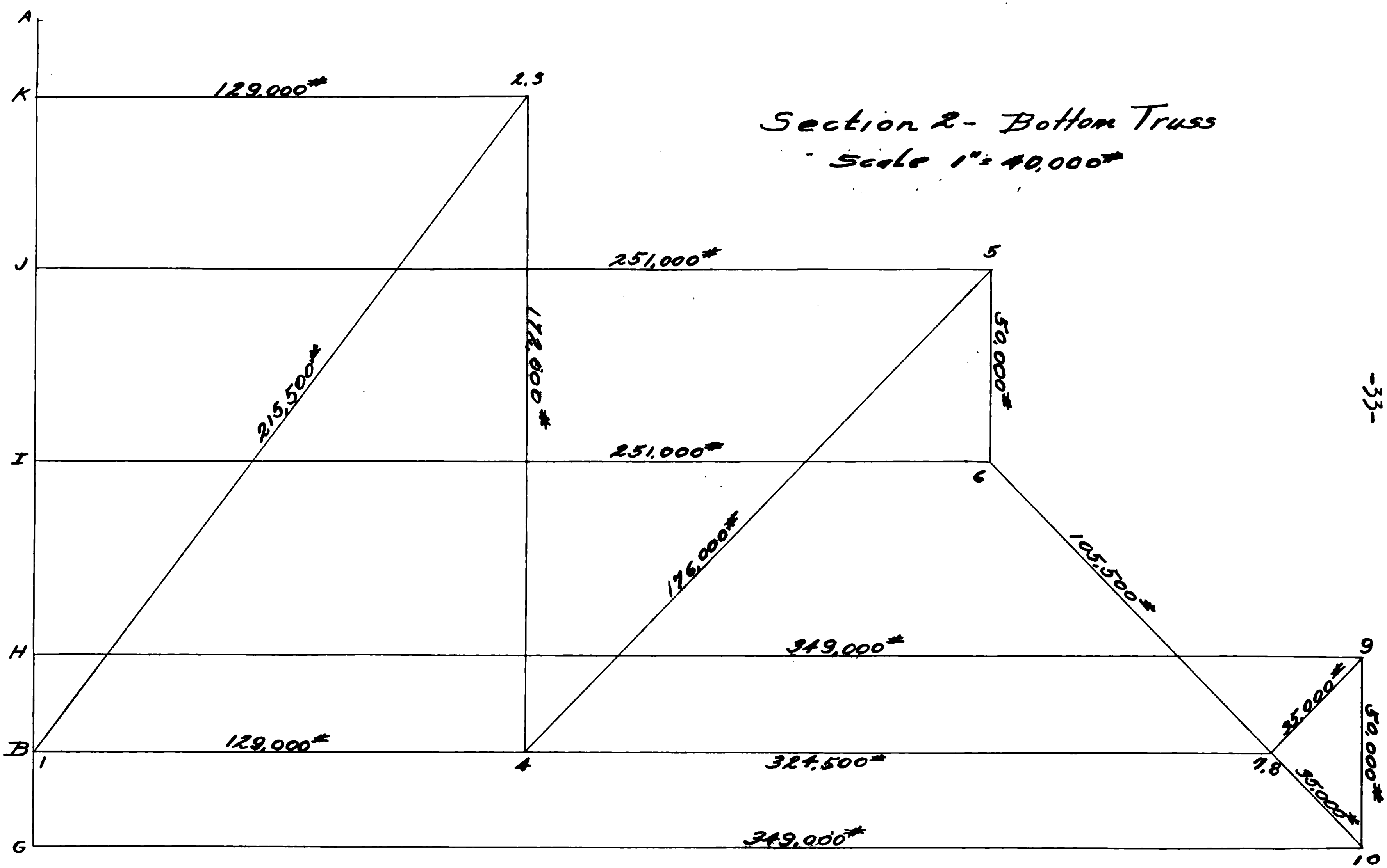

Section 1 - Top Truss
Scale 1" = 50,000#
57,000#
118,000#
57,000#
39,000#
39,000#
389,500#
362,500#
389,500#
196,500#
280,500#
280,500#
192,500#
144,500#
211,000#
144,500#
1
2,3
4
5
6
7,8
9
10
A
K
J
I
H
B
G

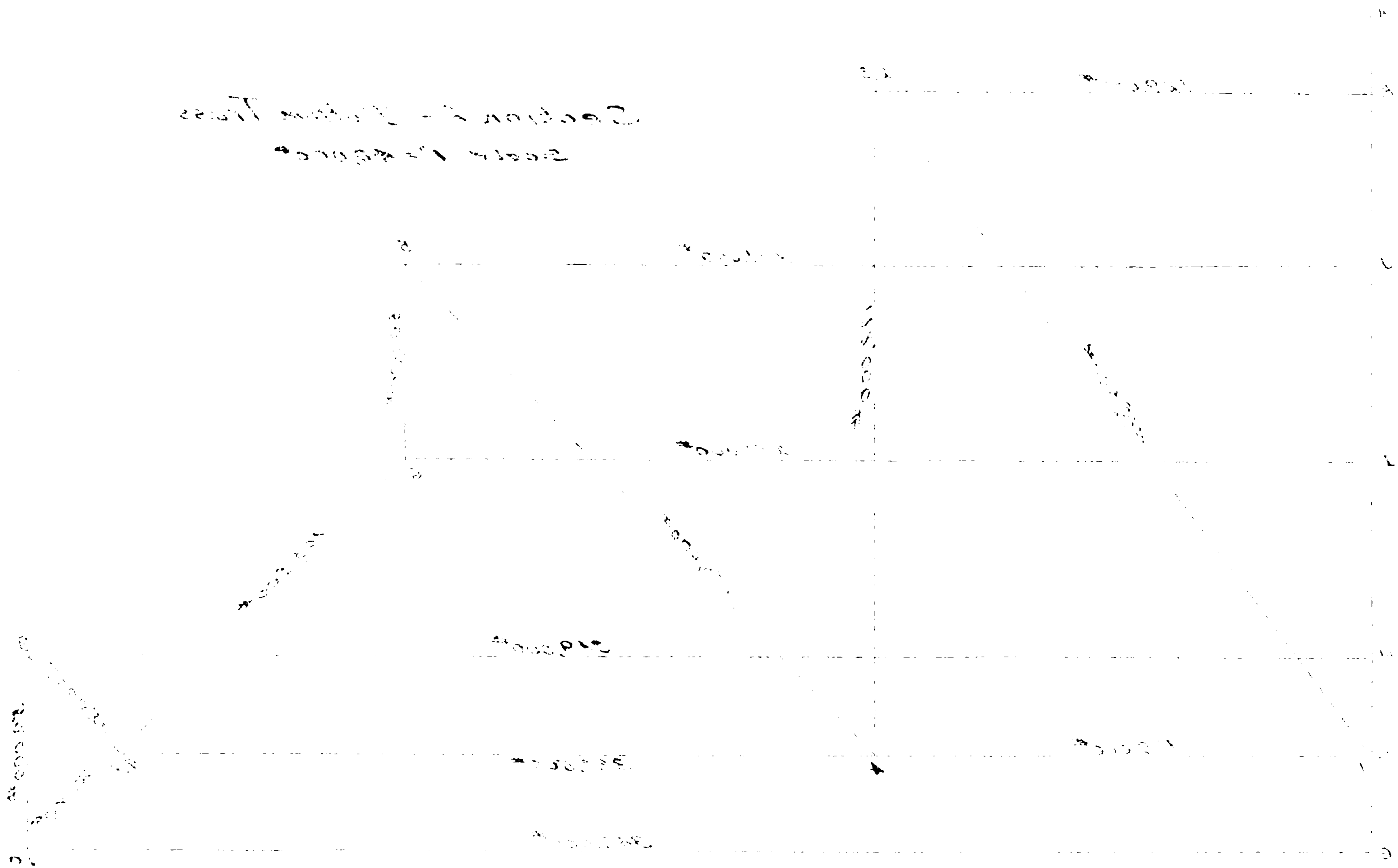
Section I - Top Truss
Scale 1" = 50000#

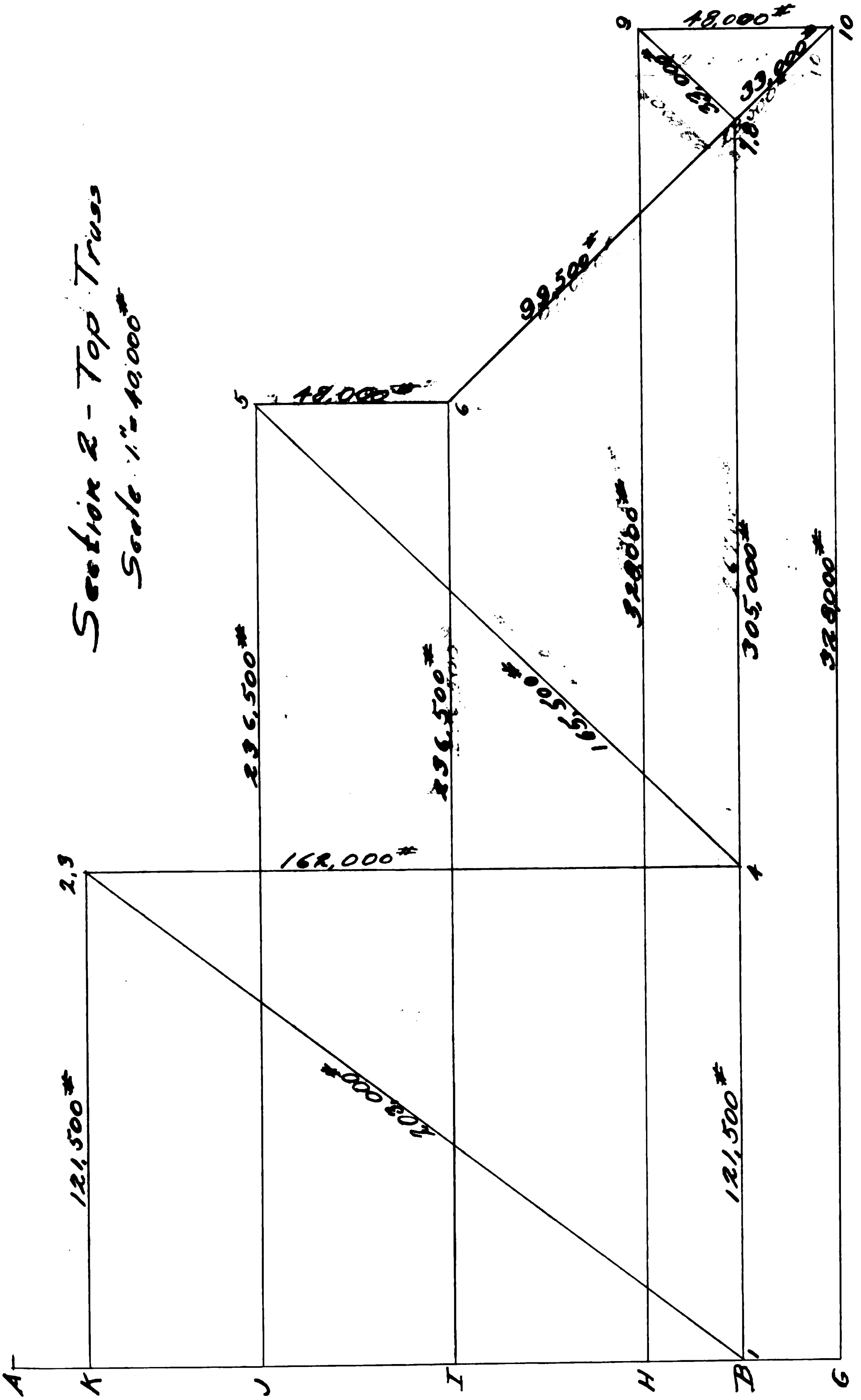


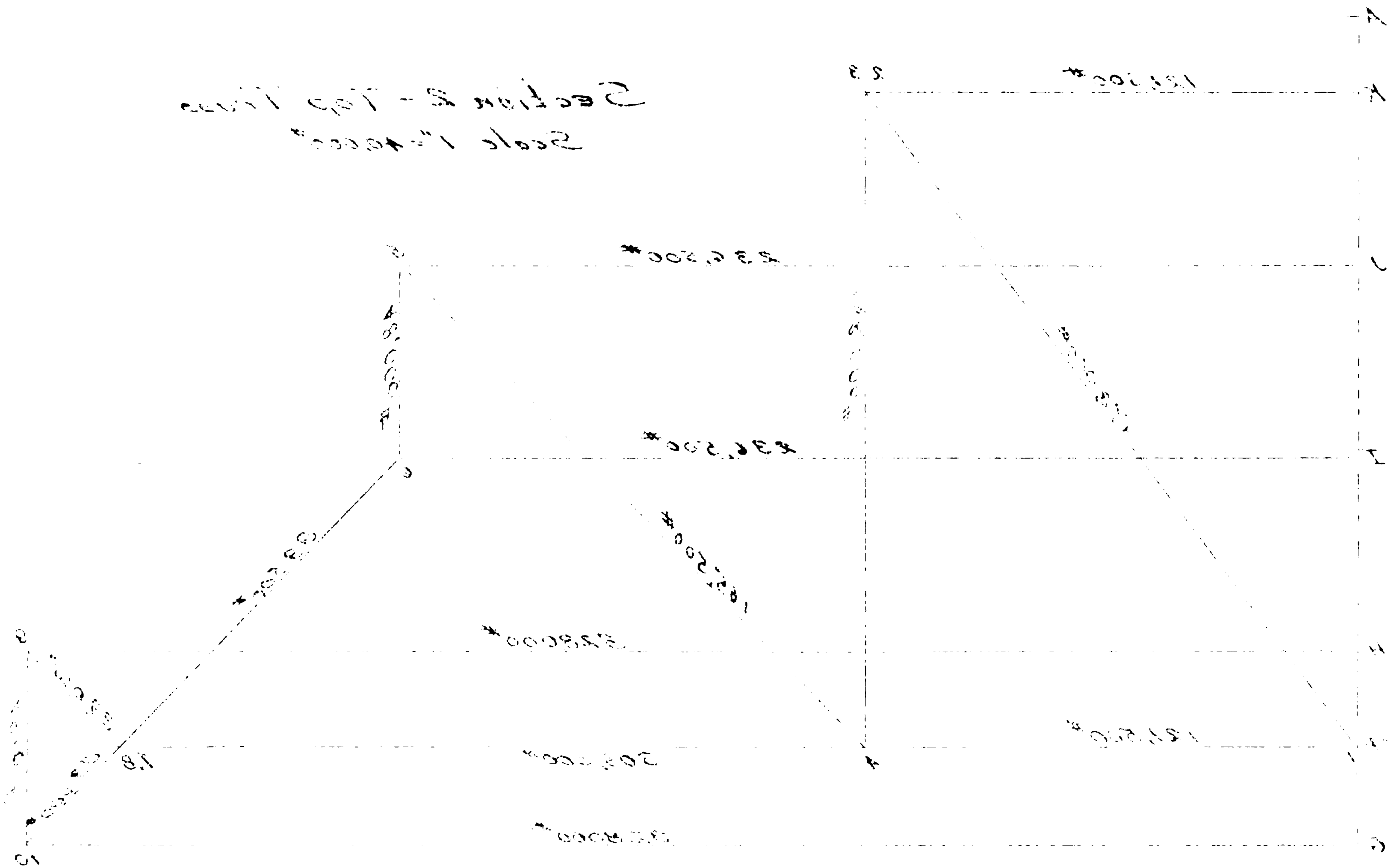
Section 2 - Bottom Truss

Scale 1" = 40,000#





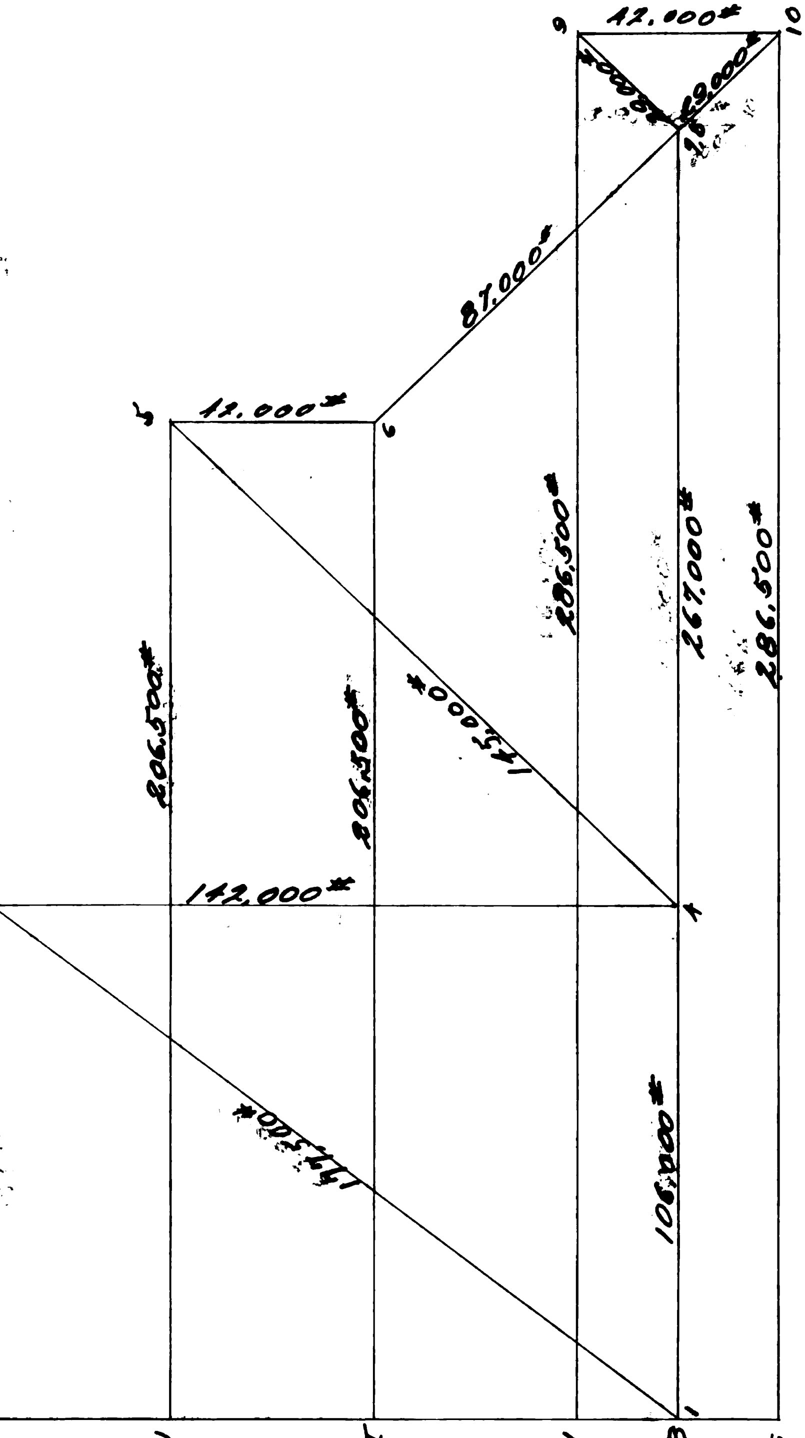

Section 2 - Top Truss
Scale 1" = 40,000#
121,500#
203,000#
162,000#
236,500#
236,500#
165,500#
48,000#
99,500#
305,000#
48,000#
33,000#
121,500#
A
K
J
I
H
B
G
2,3
4
5
6
9
10

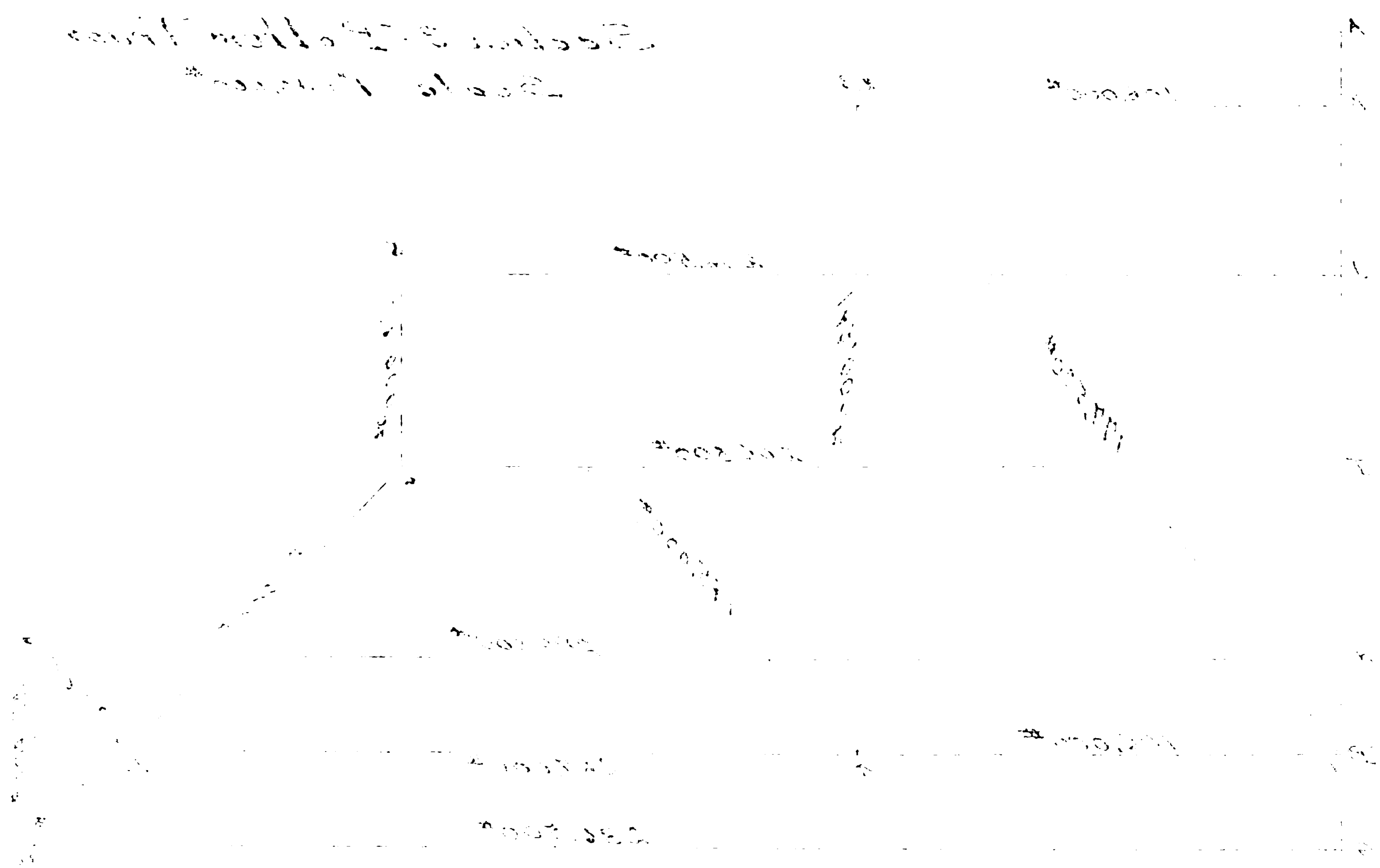


Section 2 - Top Truss
Scale 1" = 40000#

Section 3-Bottom Truss

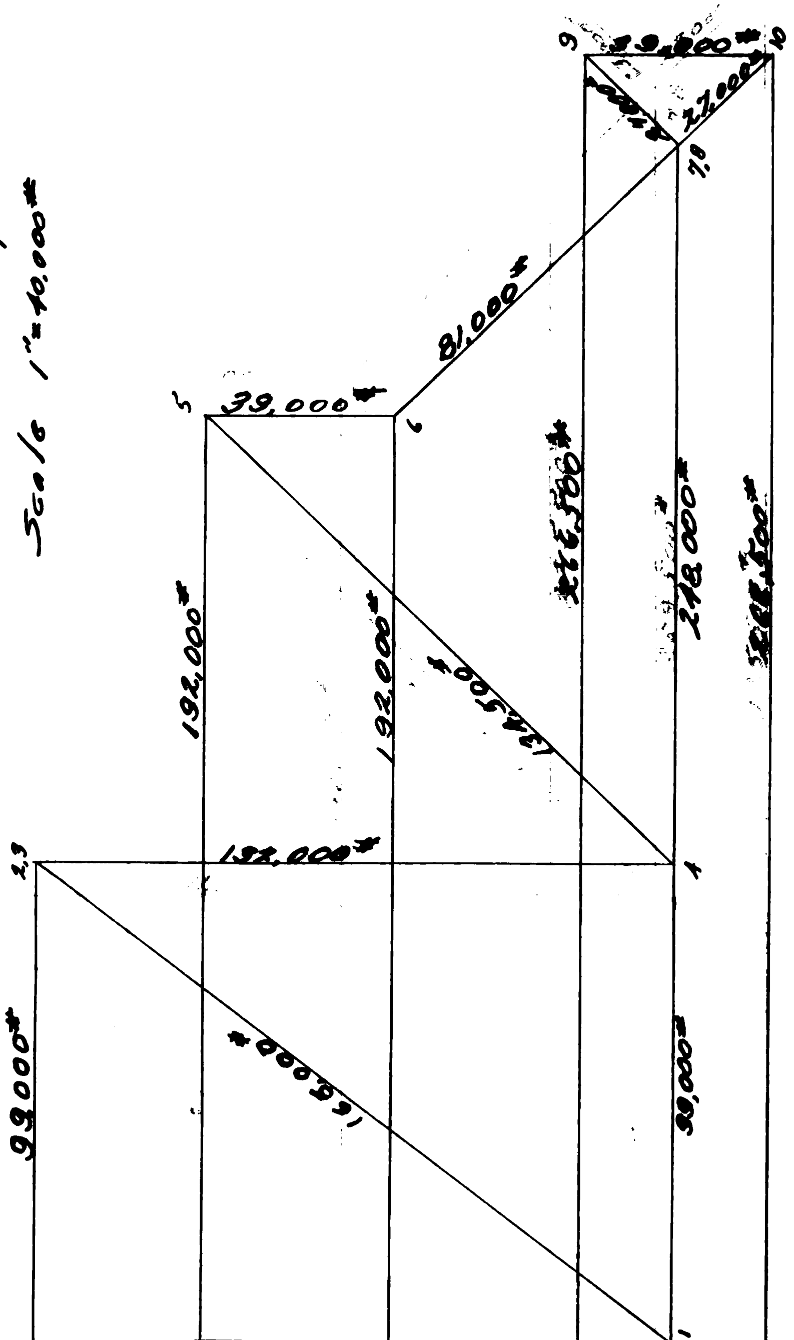
Scale 1" = 10,000#





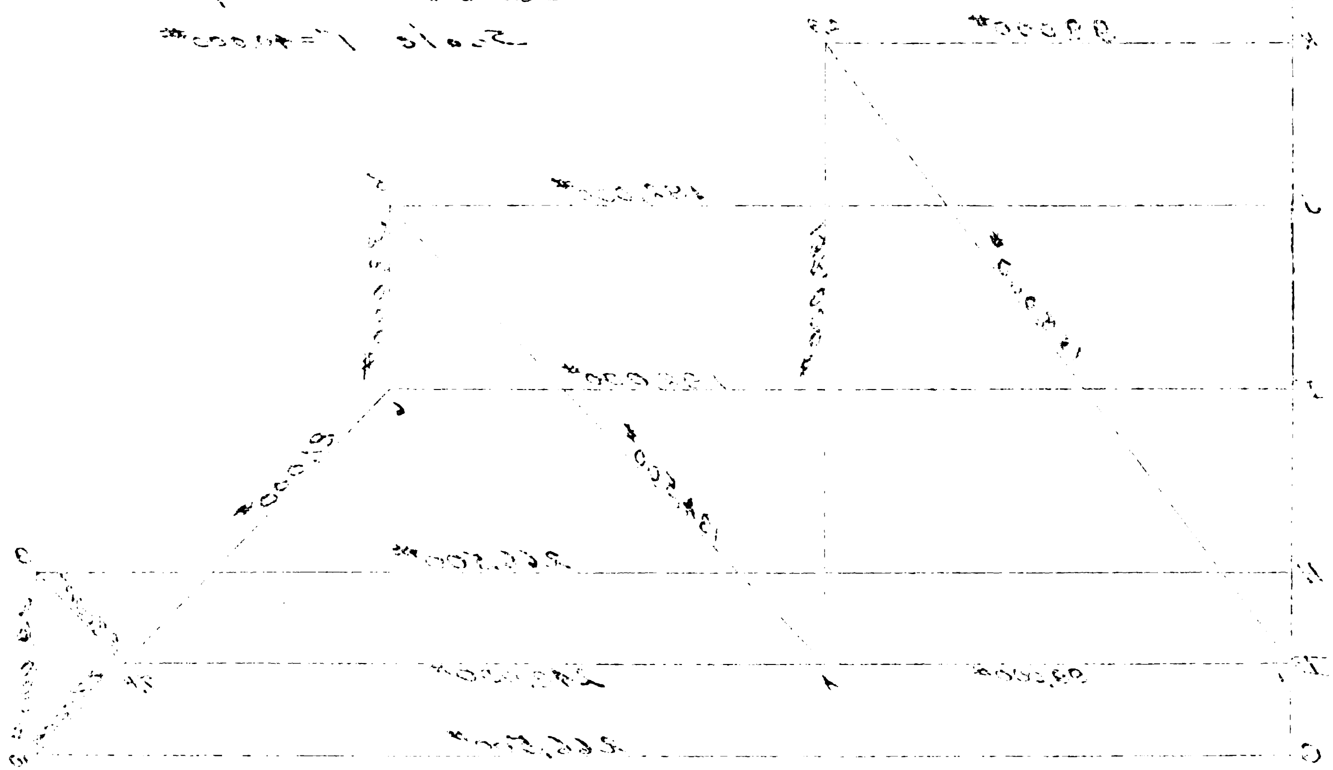
Section 3 - Top Truss

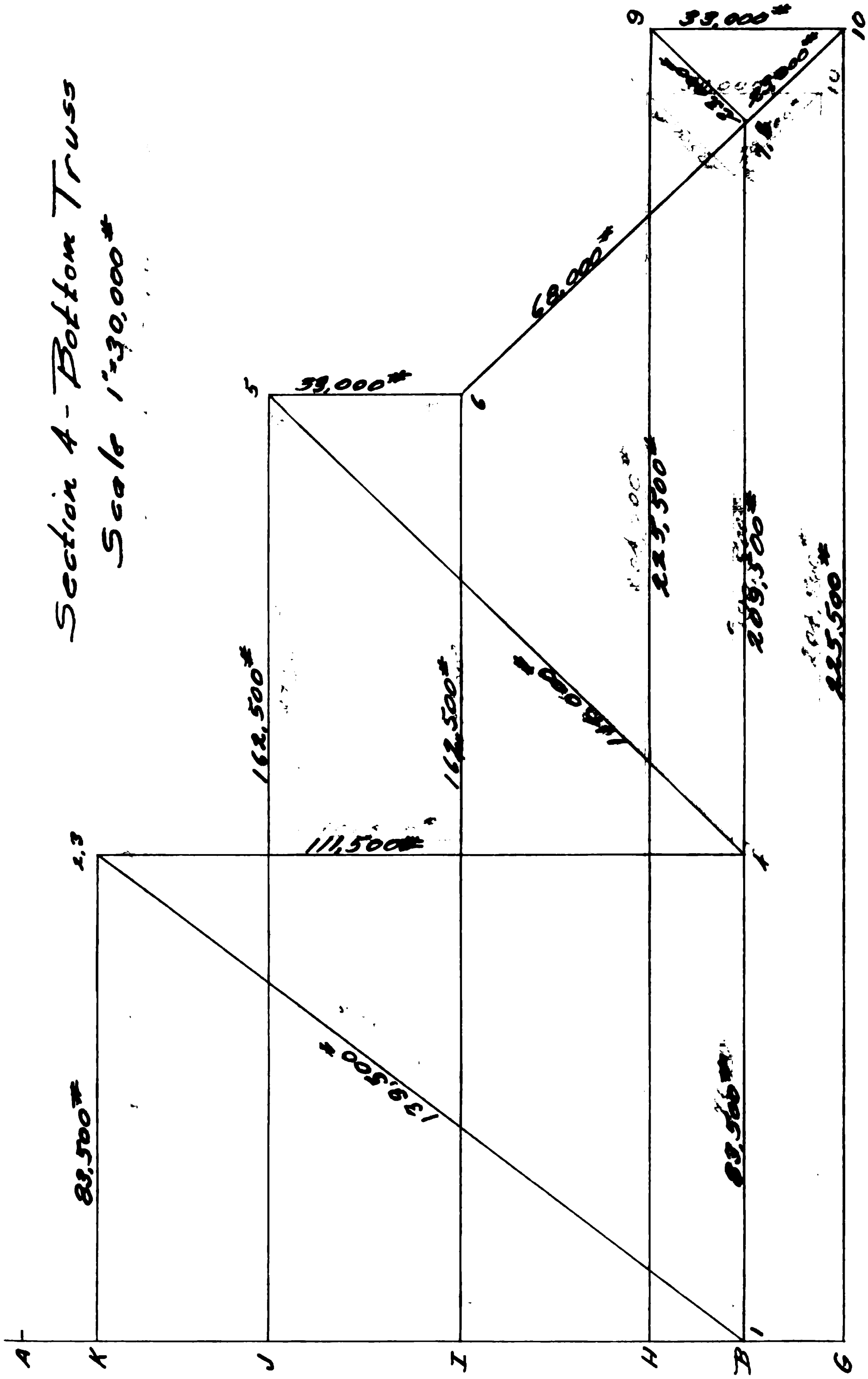
Scale 1" = 40,000#



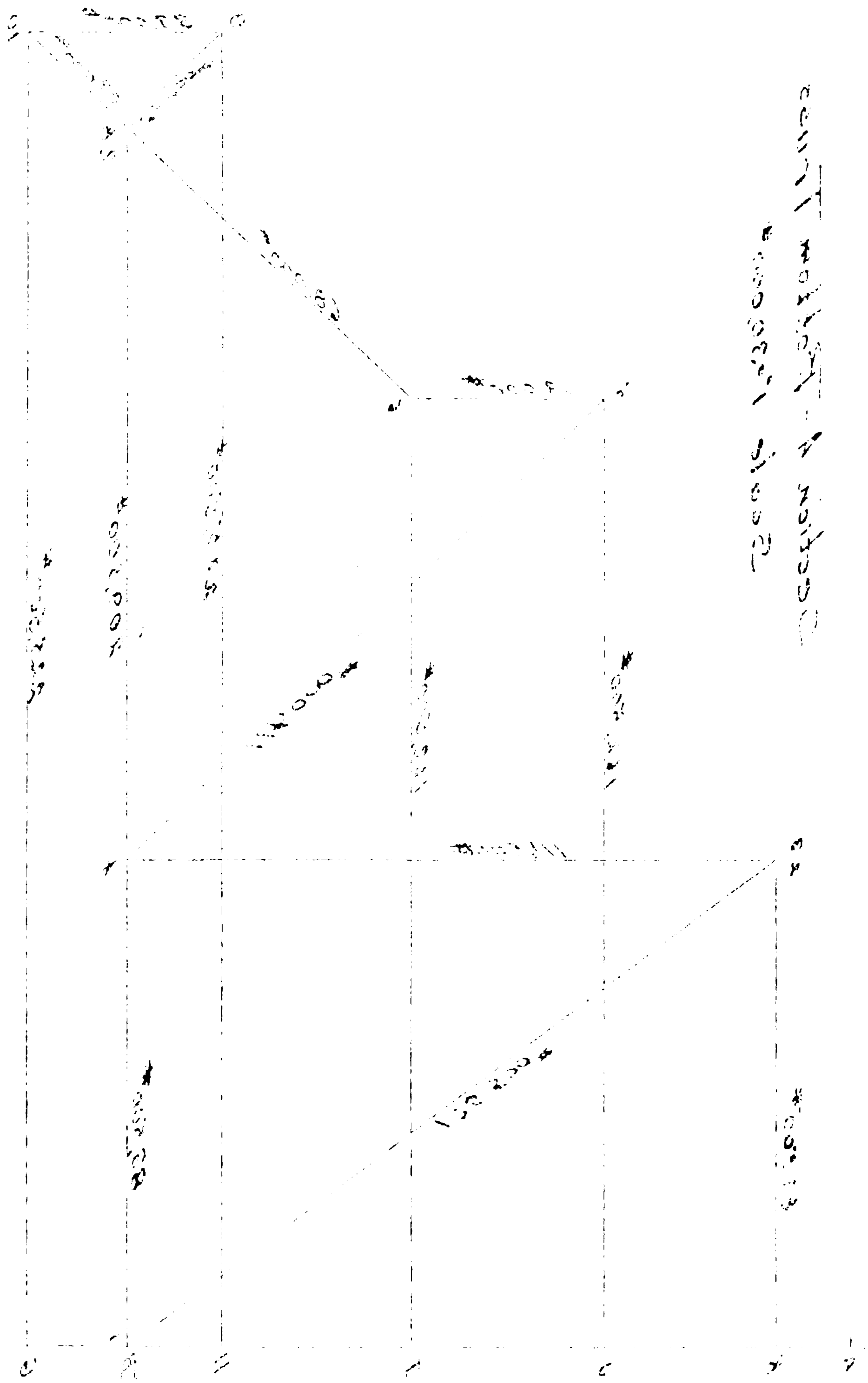
Section 3 - Top Truss

Scale 1"=4000#

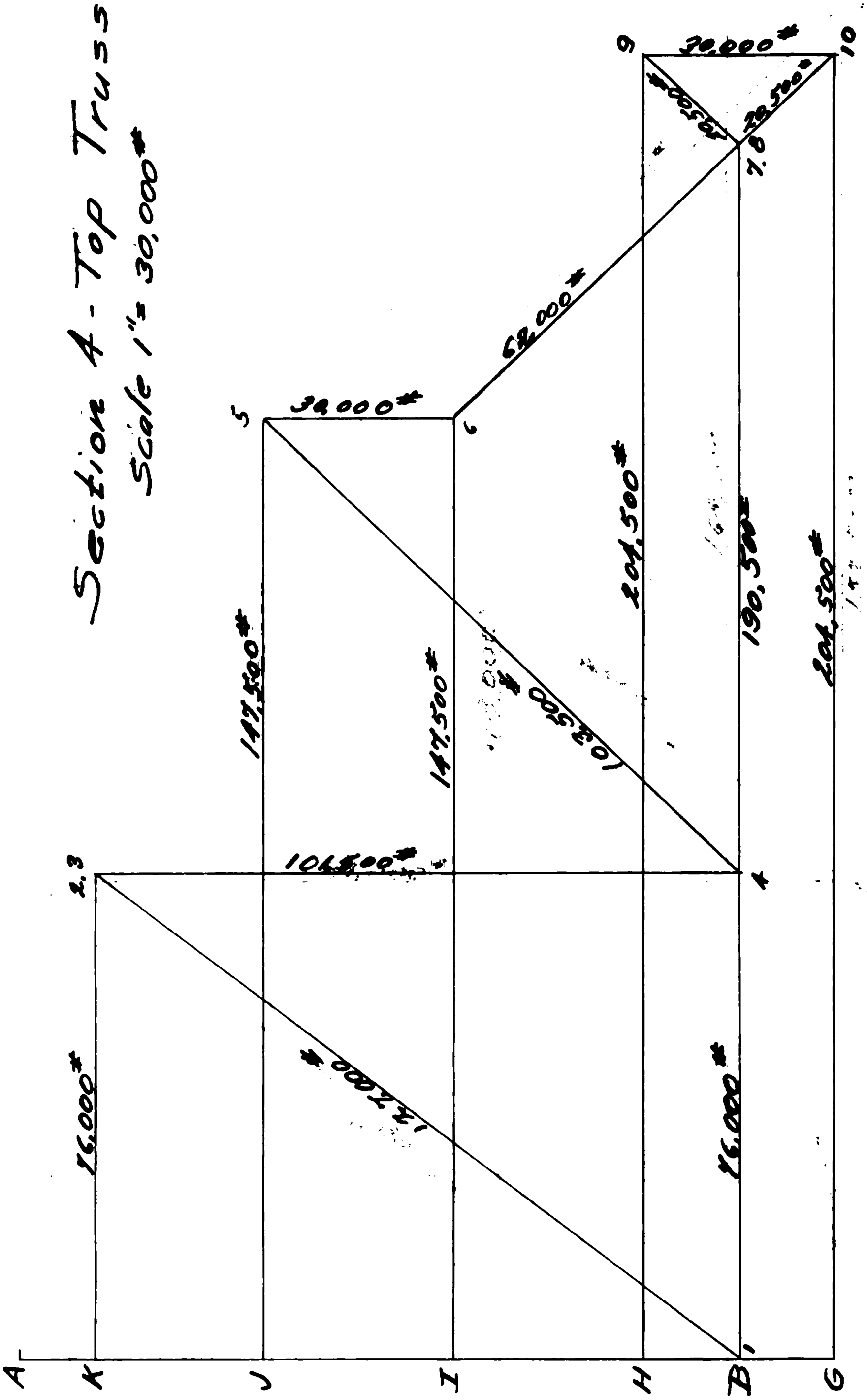


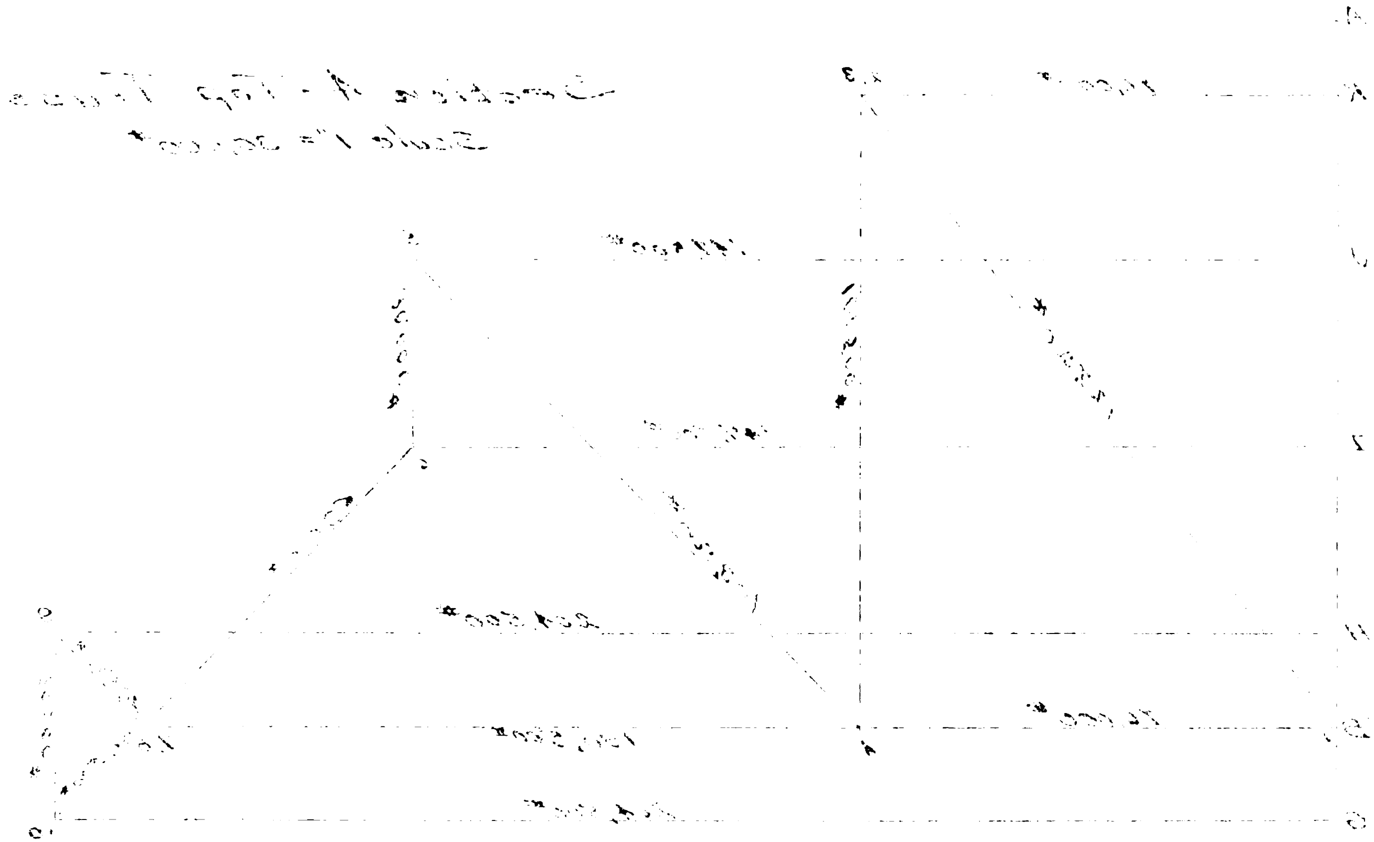


Section A - Bottom Truss
Scale 1"=30,000#
83,500#
139,500#
162,500#
162,500#
111,500#
33,000#
33,000#
68,000#
223,500#
209,500#
225,500
83,500#
A
K
J
I
H
B
G
1
2,3
4
5
6
9
10



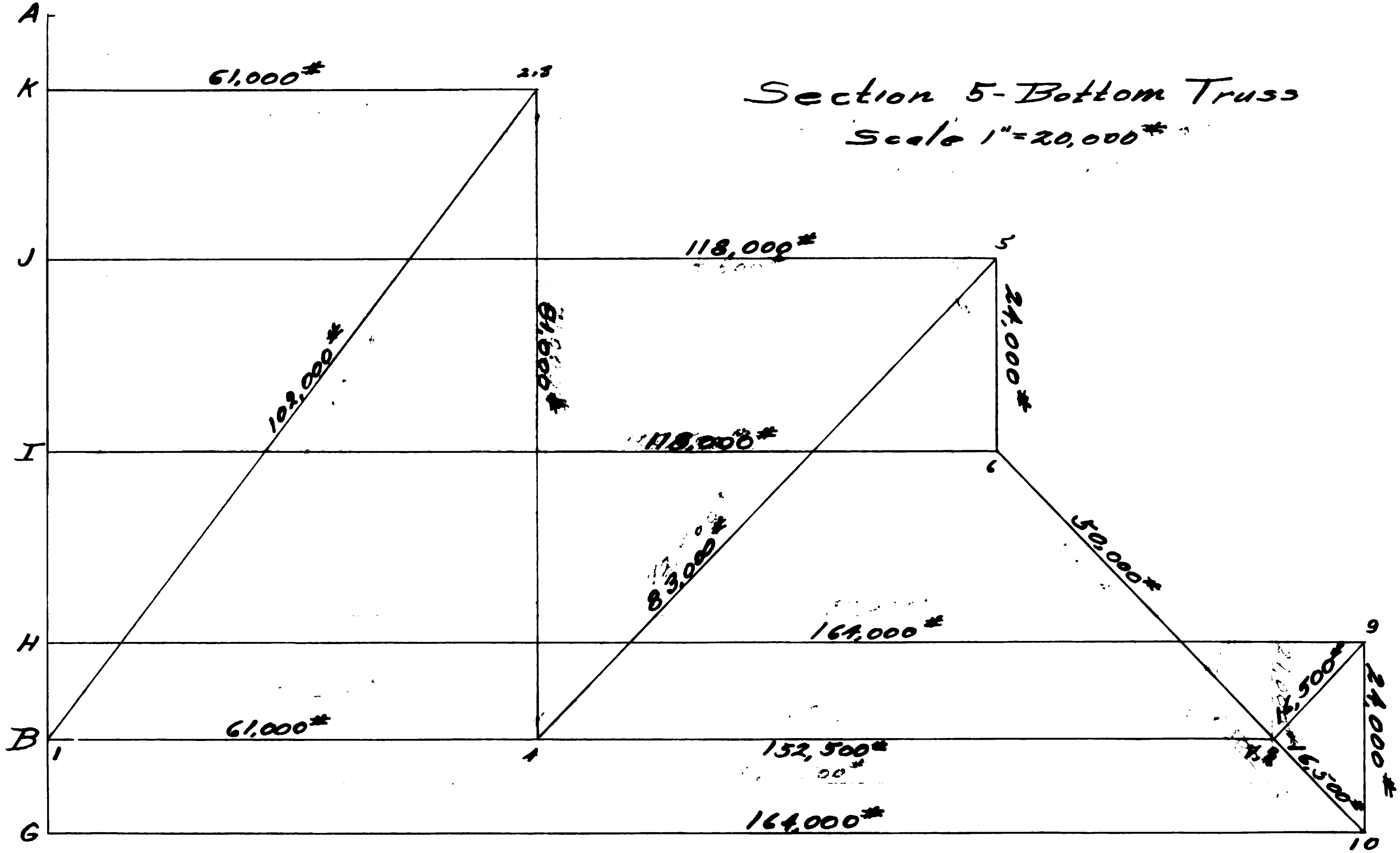
Section A - Top Truss
Scale 1" = 30,000#

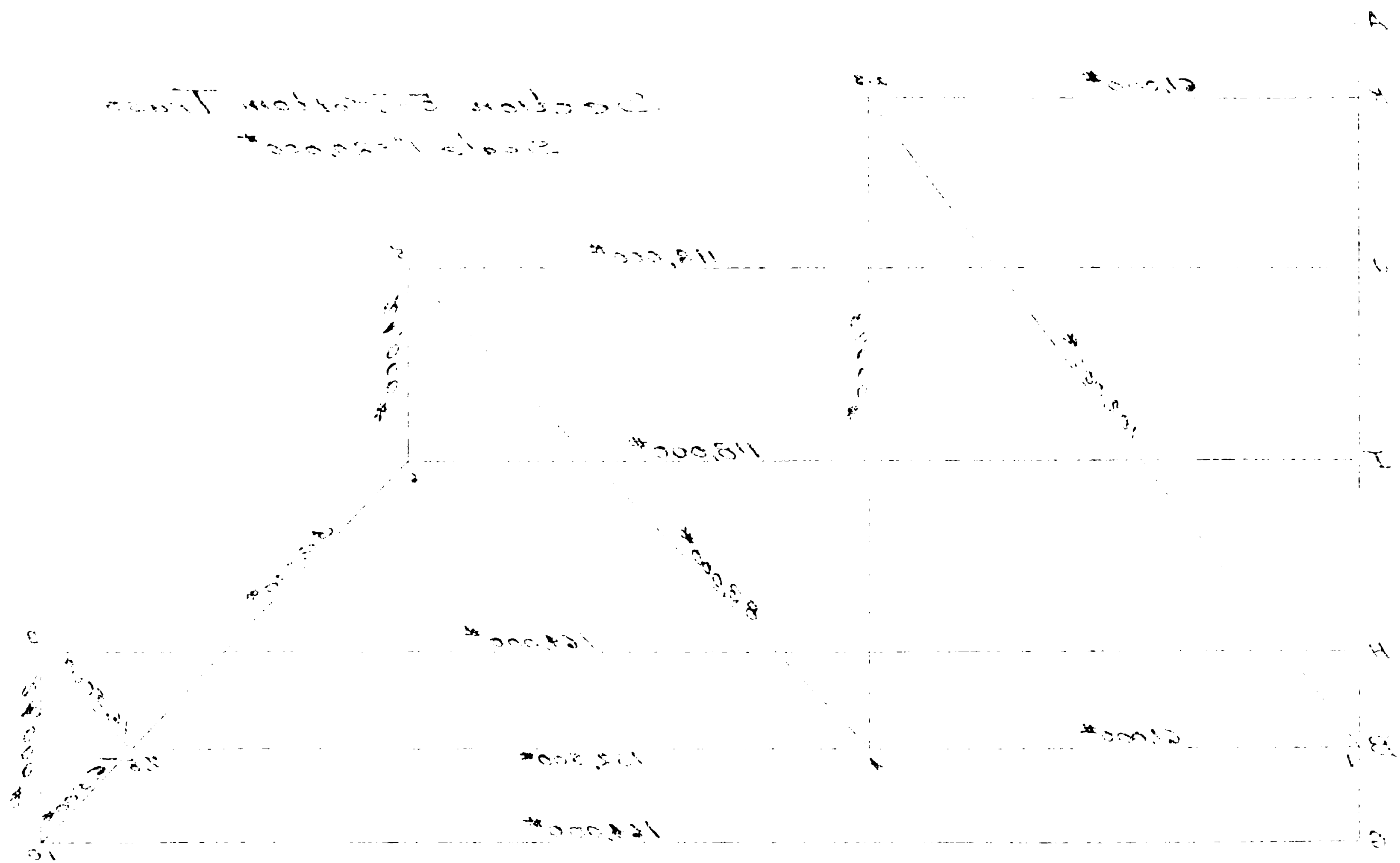


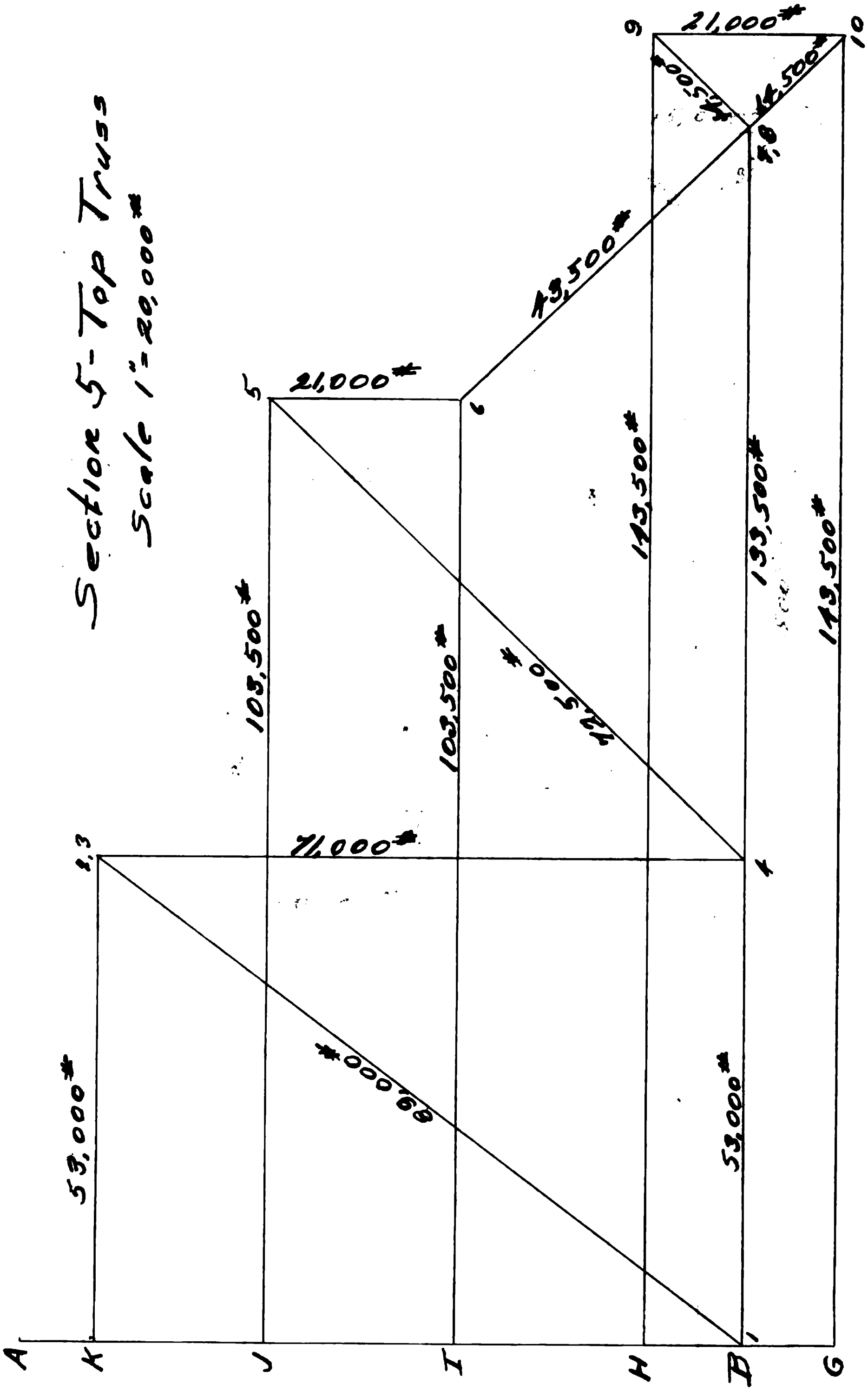


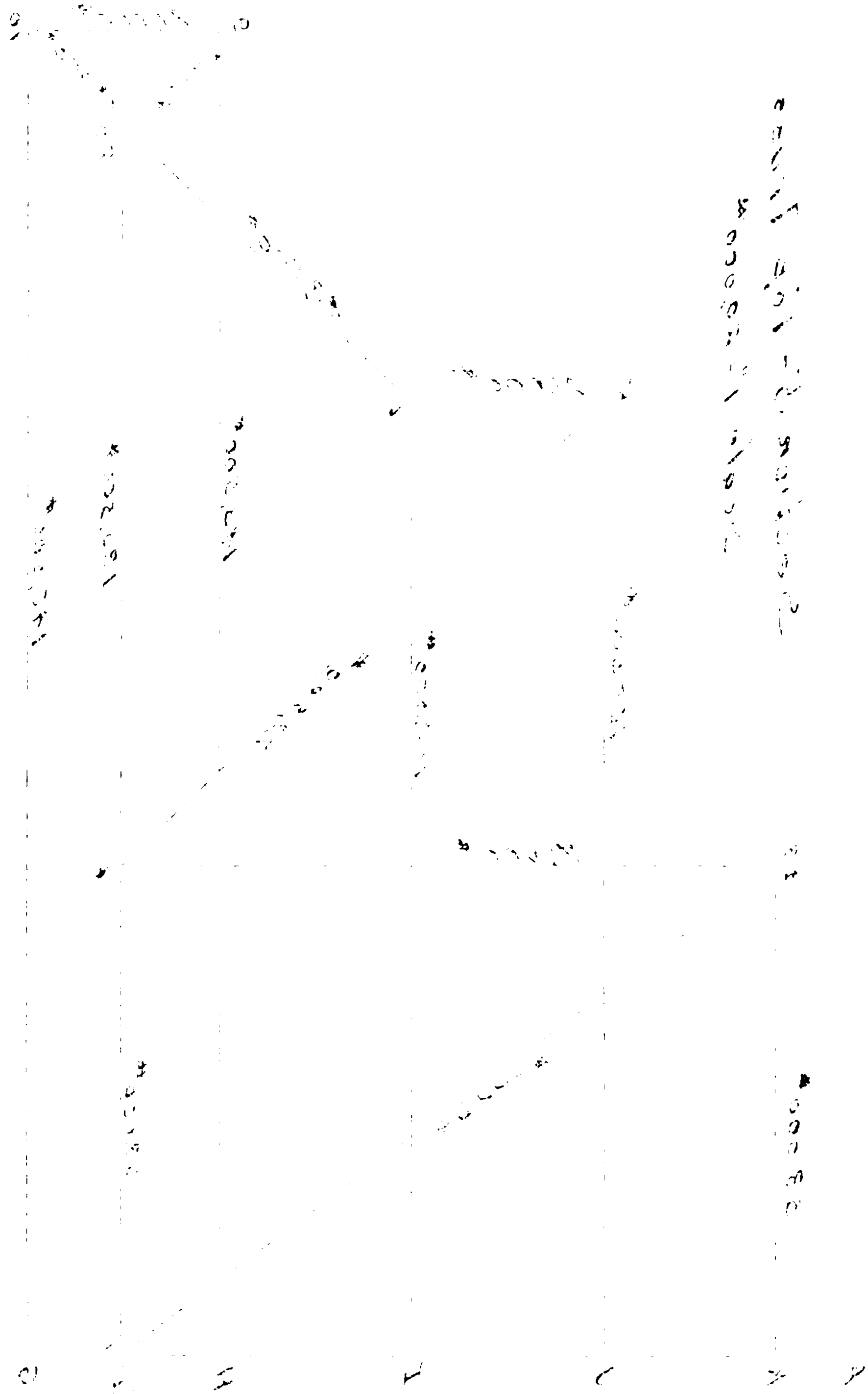
Section 5- Bottom Truss

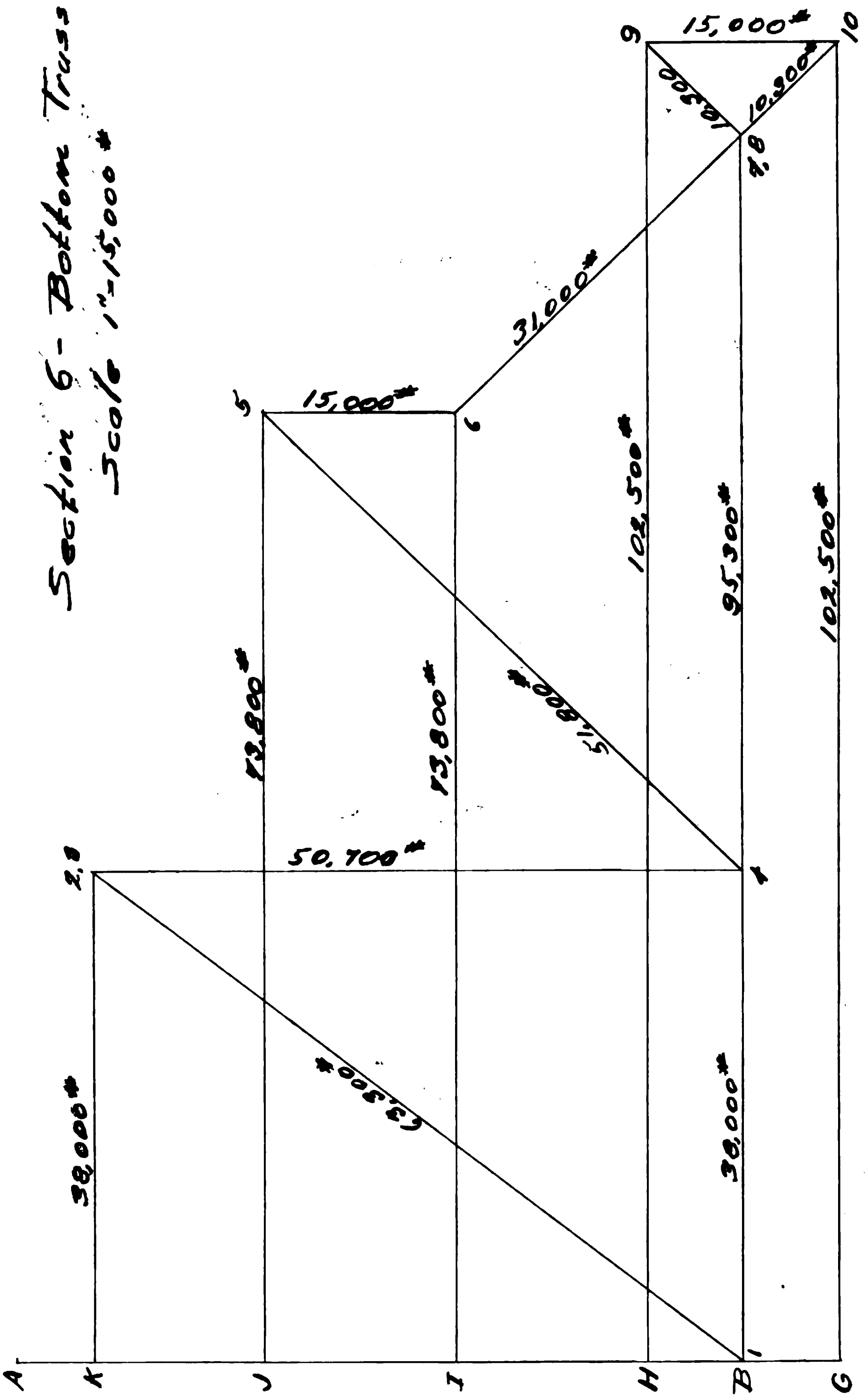
Scale 1" = 20,000#



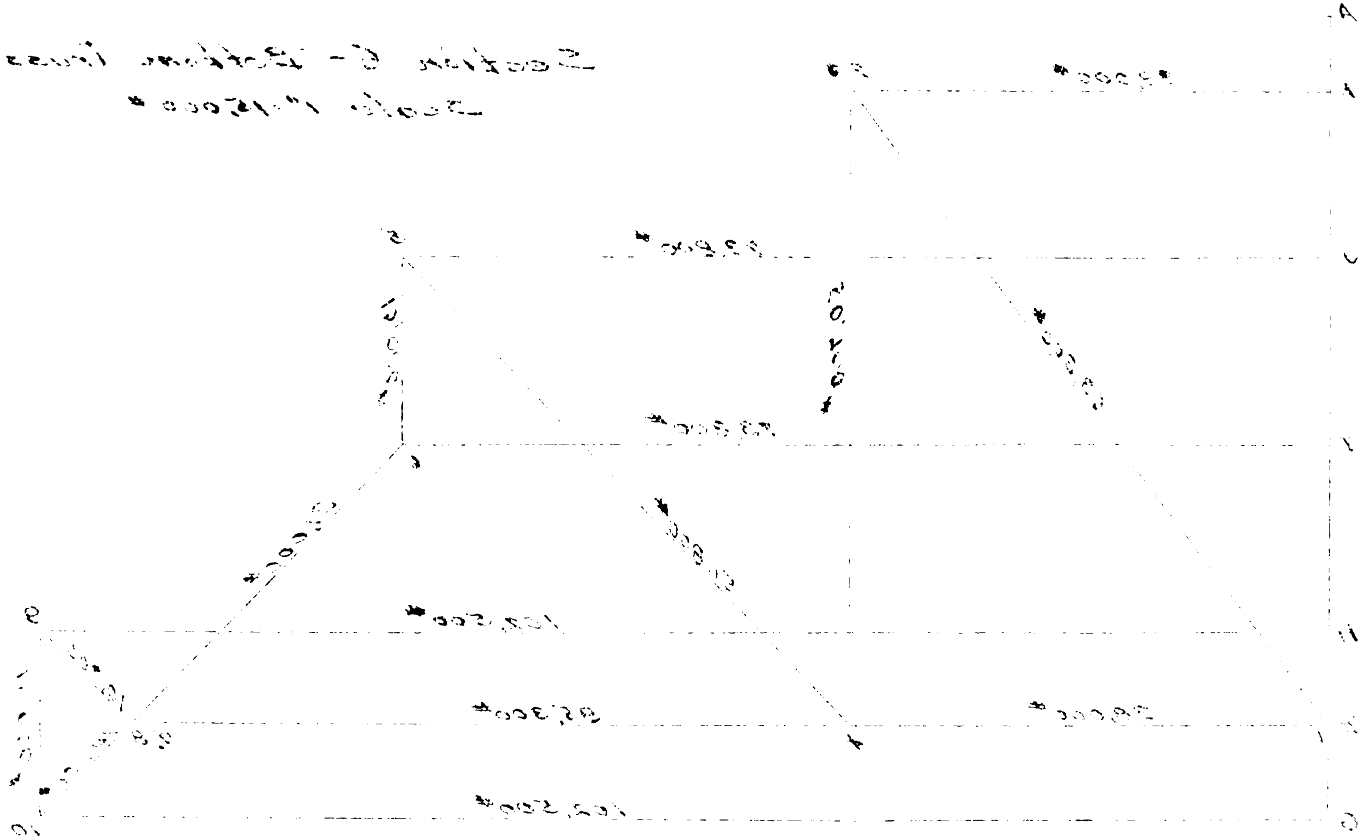

Section B Bottom Truss
Scale 1"=20000#


Section 5- Top Truss
Scale 1" = 20,000#
53,000#
89,000#
71,000#
105,500#
103,500#
21,000#
73,500#
143,500#
133,500#
143,500#
73,500#
21,000#
A
K
J
I
H
B
G
1
1,3
4
5
6
9
10

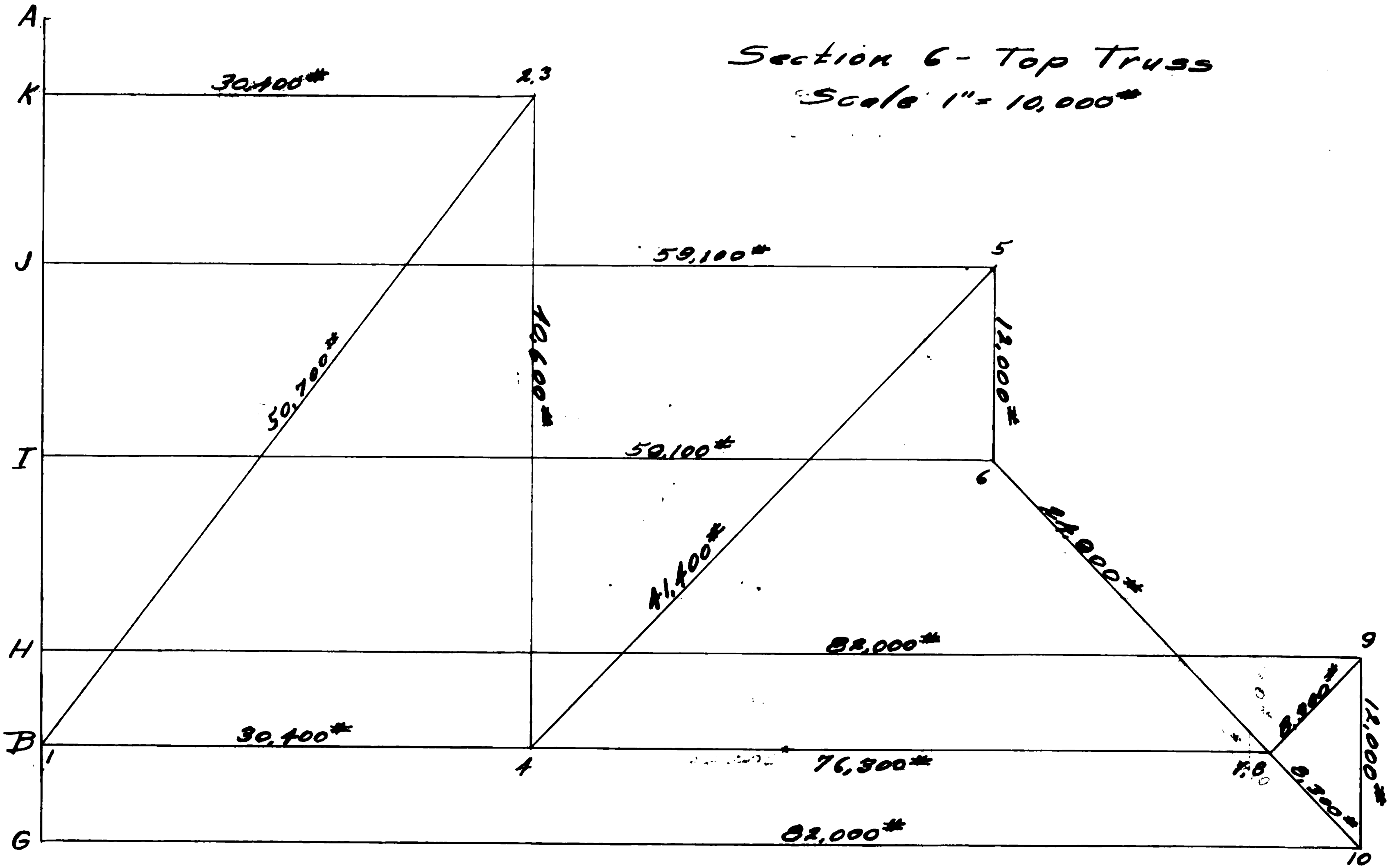


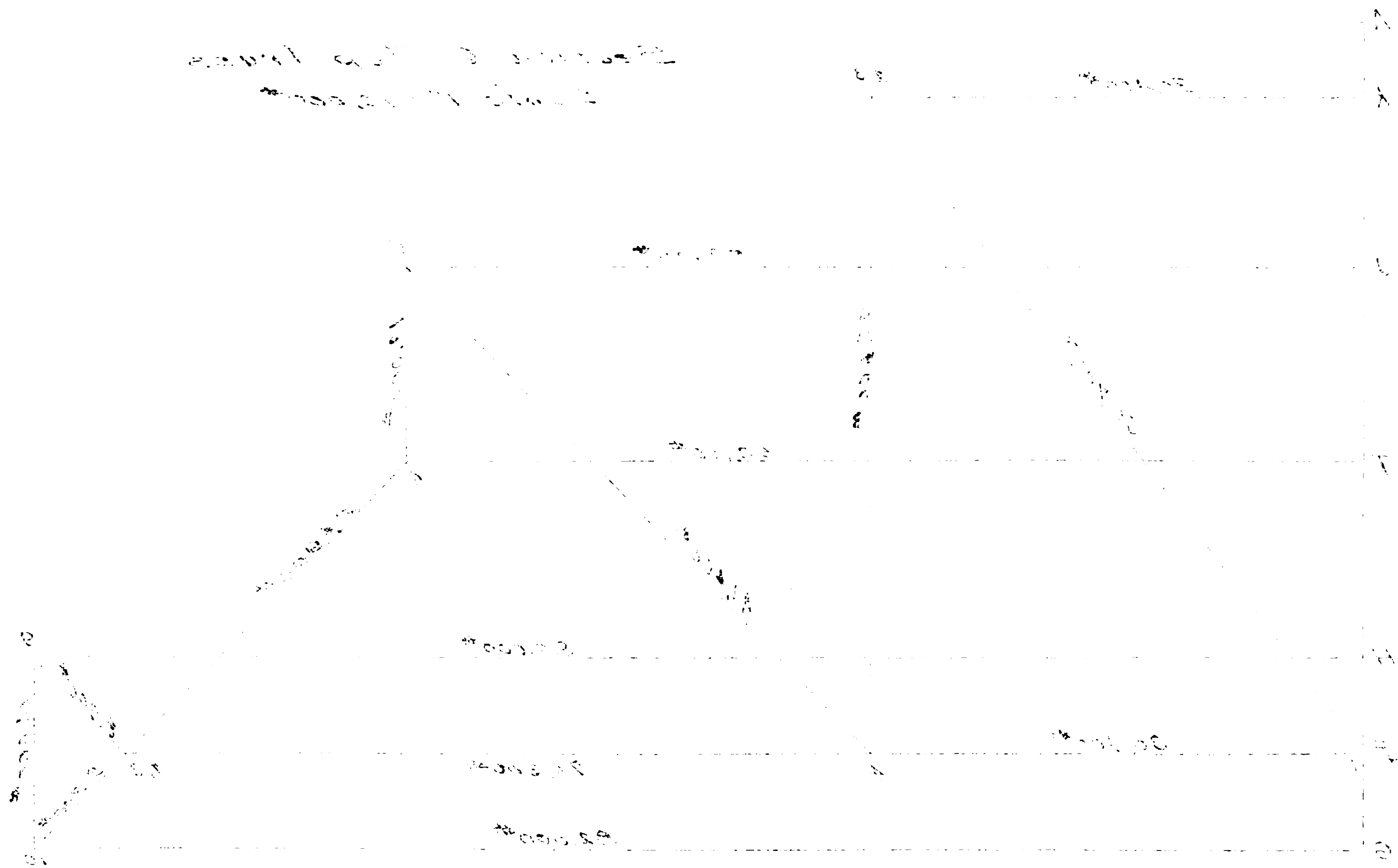

Section 6 - Bottom Truss
Scale 1"=15,000#
A
K
J
I
H
B
G
1
2,3
4
5
6
7,8
9
10
38,000#
63,300#
50,700#
73,800#
73,800#
51,800#
15,000#
31,000#
102,500#
95,300#
102,500#
38,000#
15,000#
10,800#
10,900#

Section C - Bottom Truss
Scale 1" = 15000#



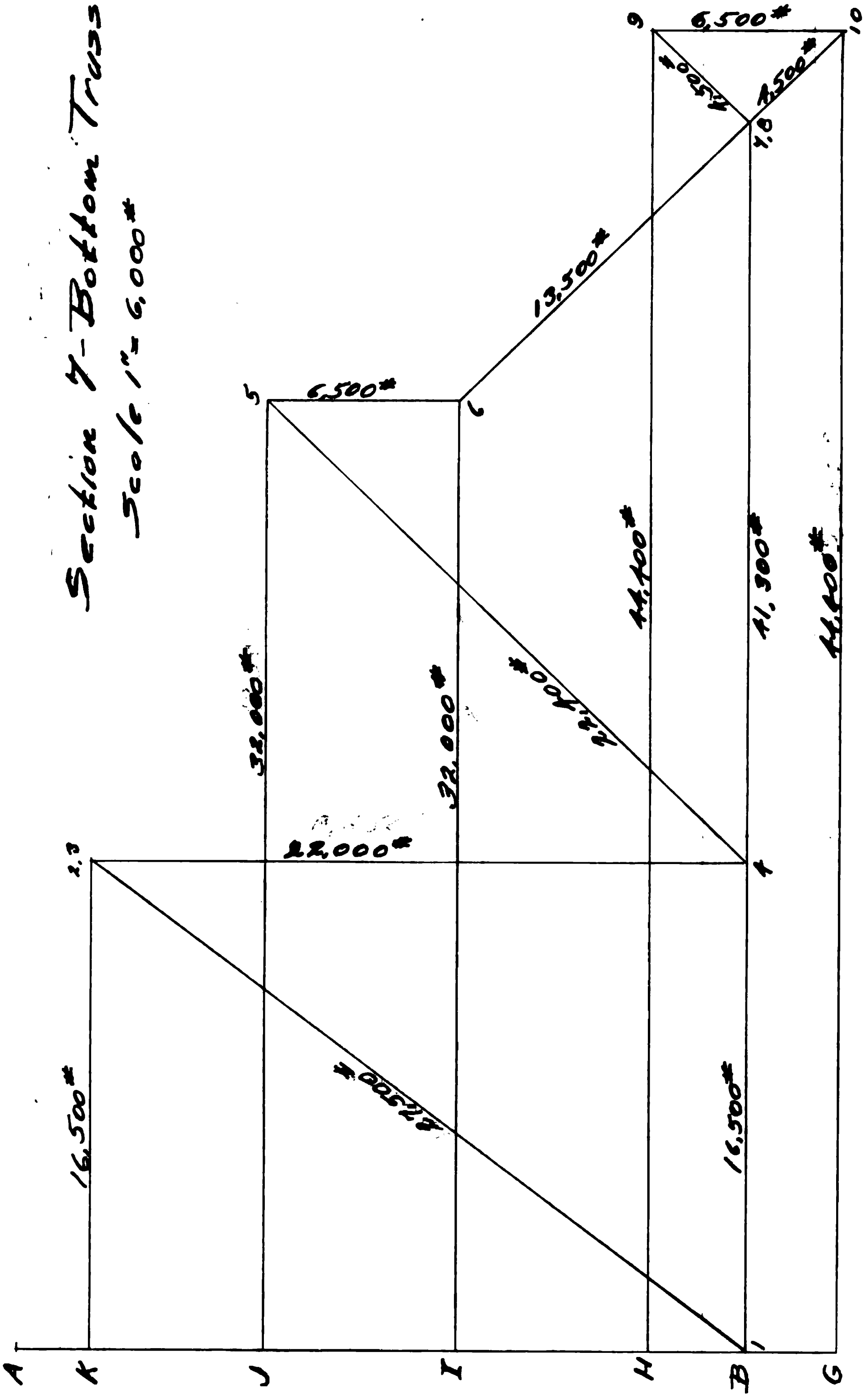
Section 6 - Top Truss
Scale 1" = 10,000#

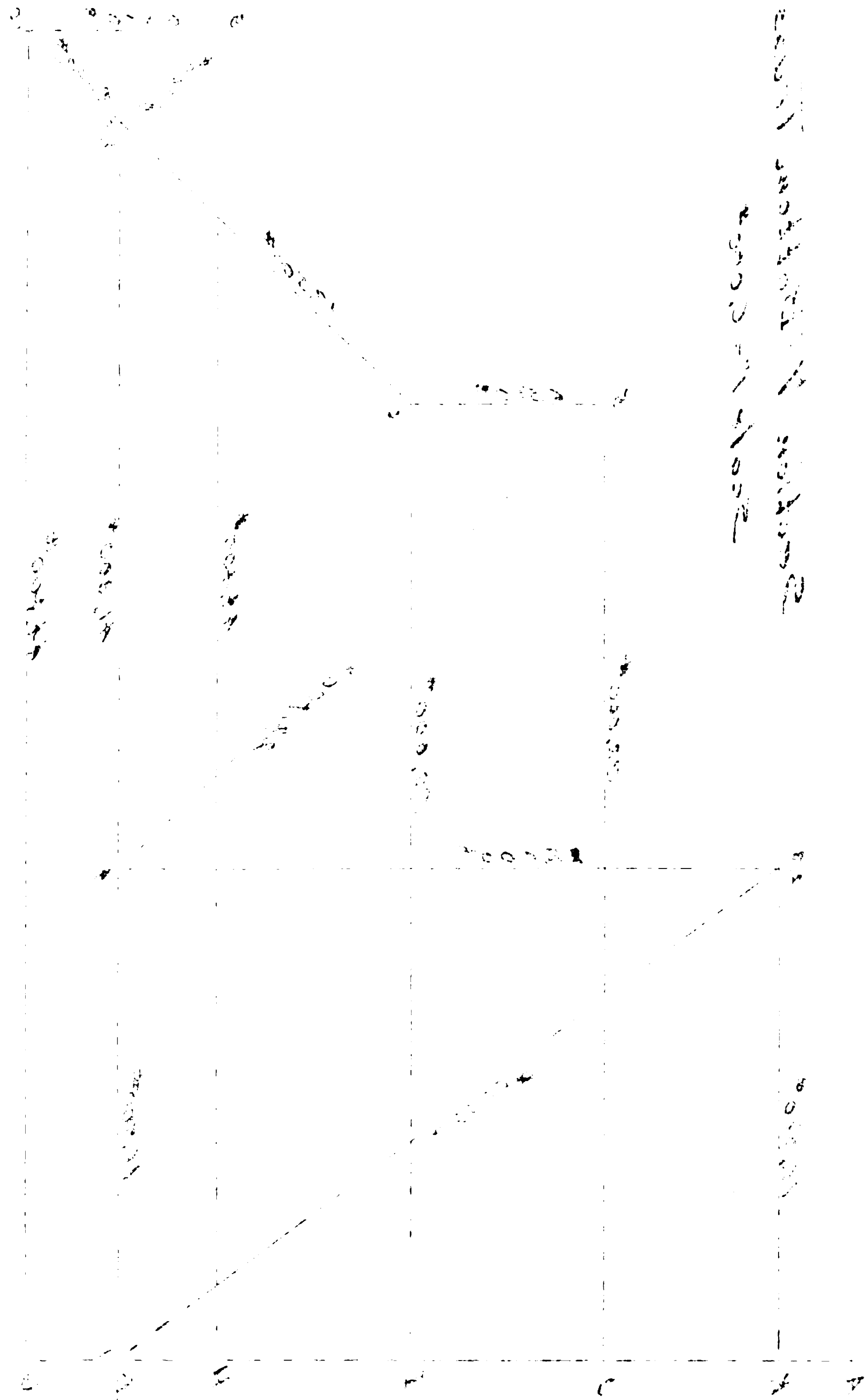




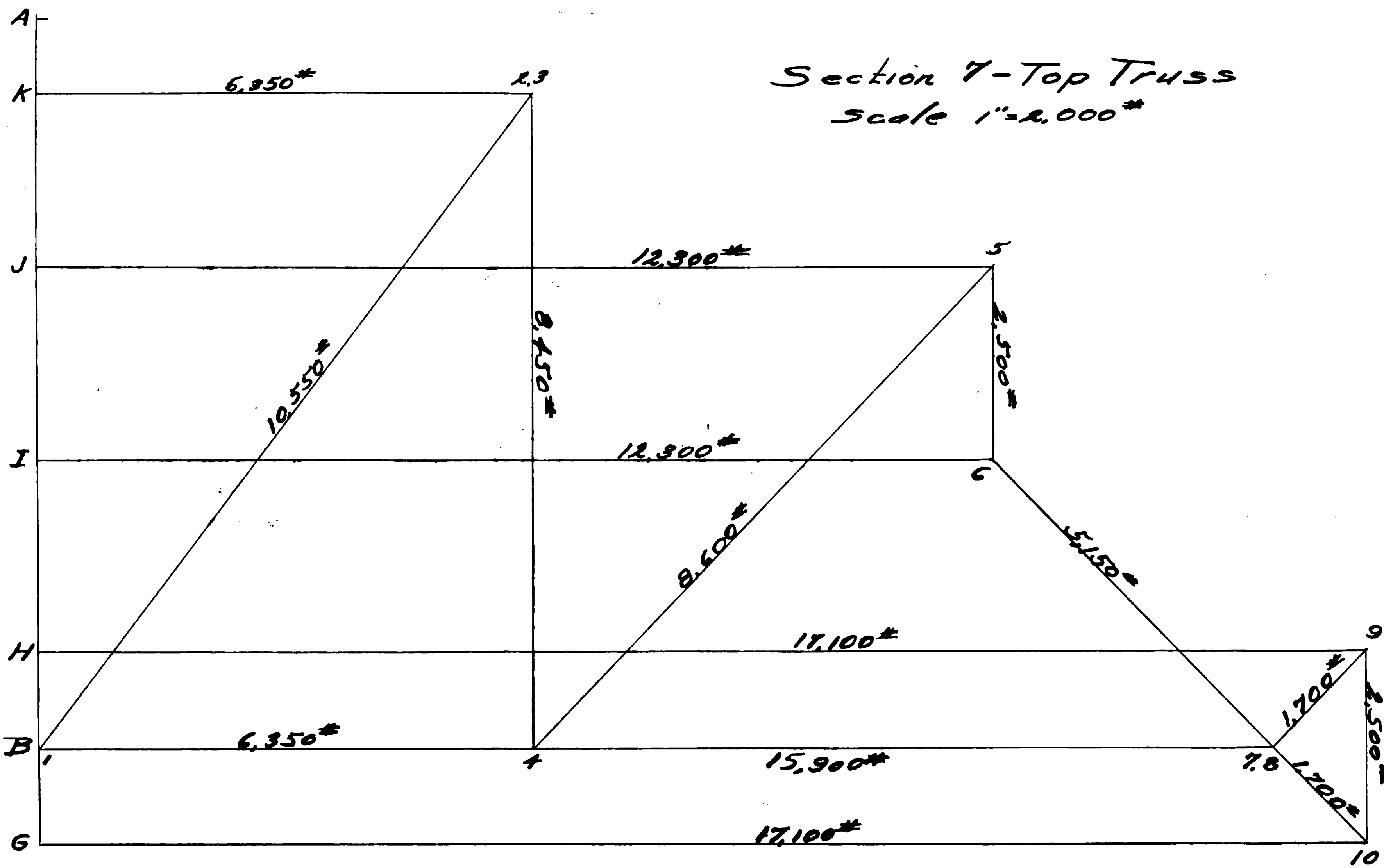
Section 7 - Bottom Truss

Scale 1" = 6,000#

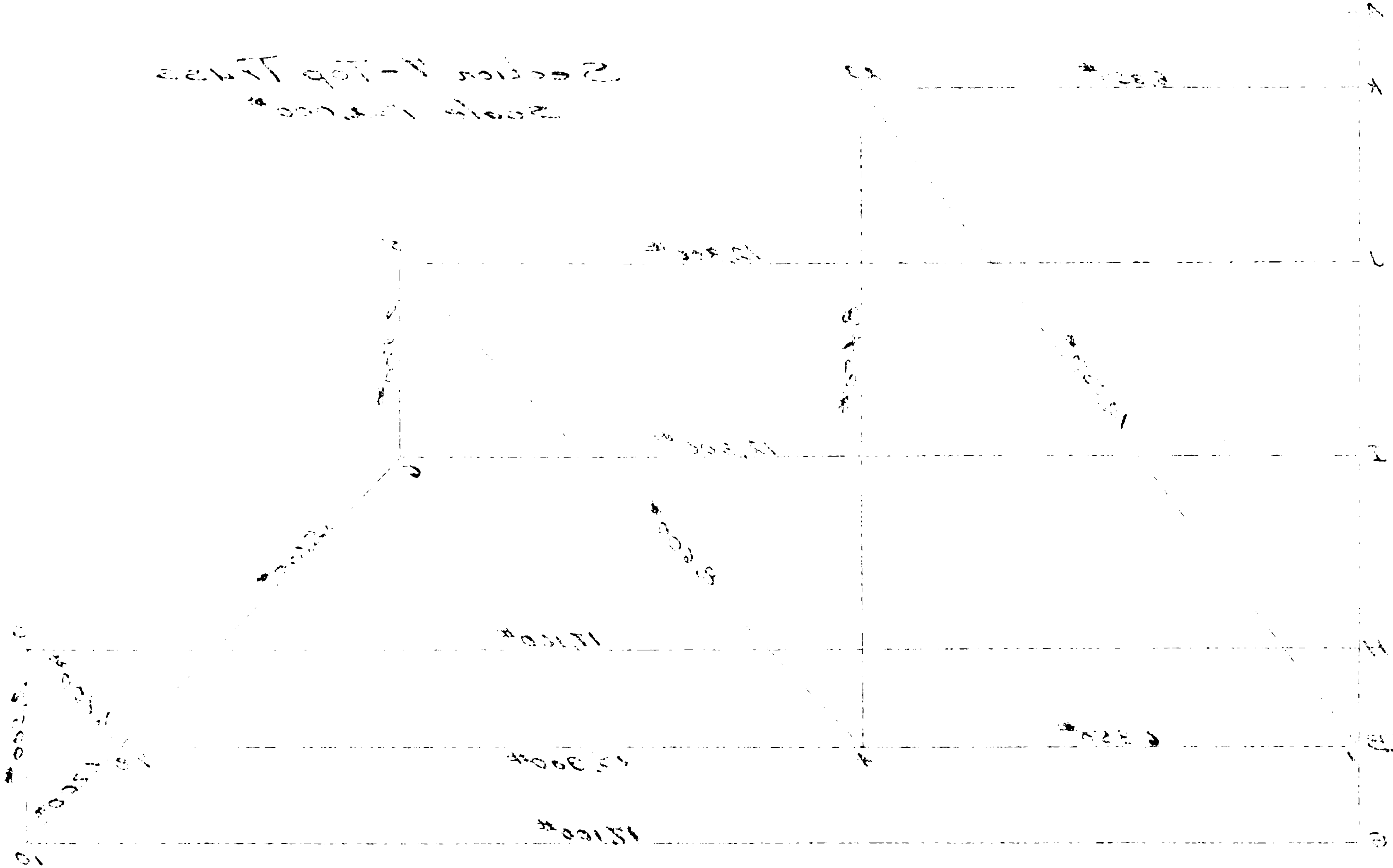




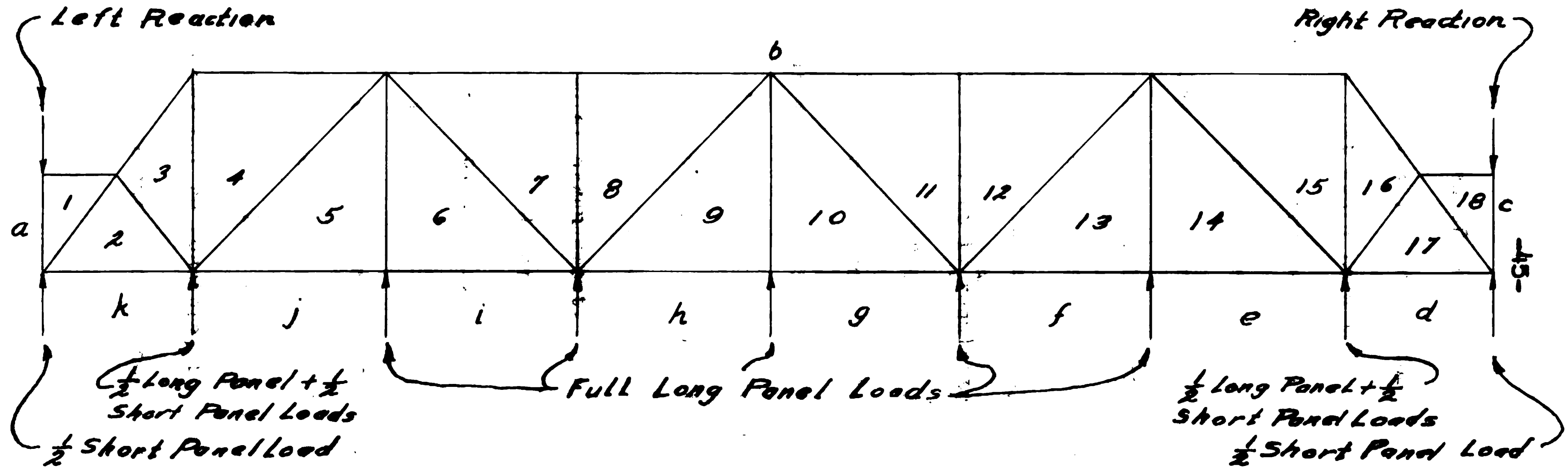
Section 7-Top Truss
Scale 1"=2,000#



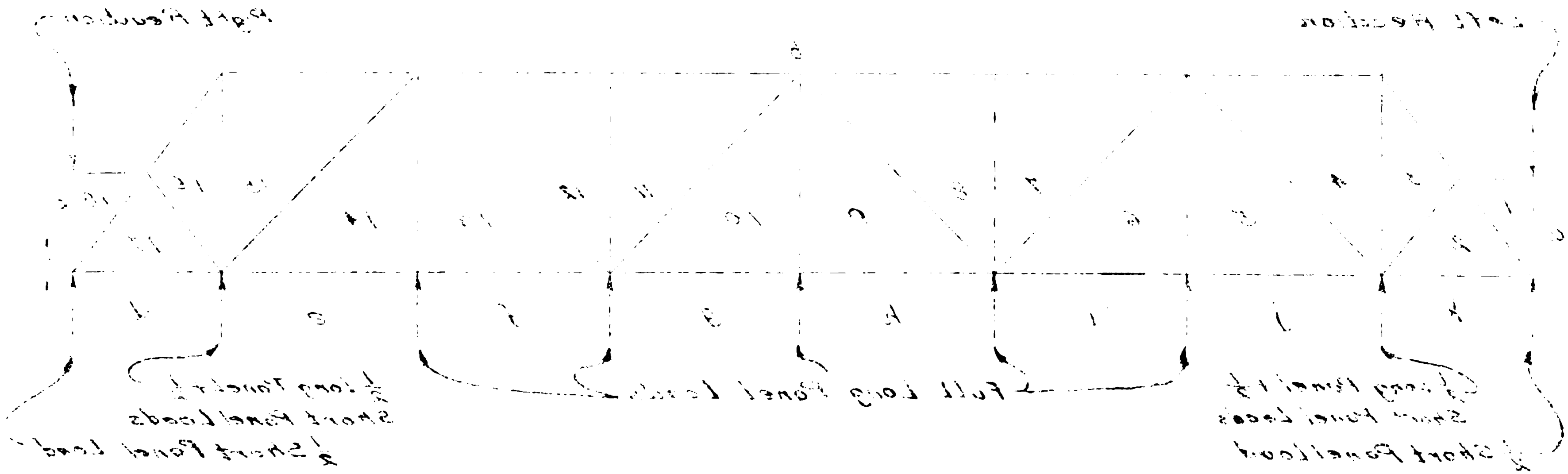
Section 4 - Top Truss
Scale 1" = 2000#



Horizontal Truss



Horizontal Truss



By having the stresses of the different members of each truss compiled, as on pages 27 to 29, there is one thing that is very obvious. The amount of steel that would have to be used to construct a movable dam of this type would be very large. This conclusion is drawn by checkin the compression members. These have the largest stresses of any of the members. As they are compression members, the unit stresses in the members may be reduced to as little as 16,000 # / sq. in. This would make the size of these members very large, and the weight a large amount. With this large weight, I believe the project would be economically impracticable; as light weight is a requisite of this type of dam.

Therefore, another type of dam should be tried. The only other type of dam that I can think of is one where the members would be mostly tension members. This would be something like a folding drum dam. That is, all of the dam would be placed under water until put in use. When it is put in use, the folds would pull up from the bottom of the canal. The folds, of course, would have to be anchored to the bottom of the canal by ties or some simular arrangement. However, the objection that I have to this type is that it could not be easily inspected. In fact the only way that it could be checked would be to raise it into the position that it would be in when in use.

In conclusion, about all that I can say is that more reserch must be done on different types of dams for this canal until some one is found that is more economically suited to the conditions than the ones that I have tried. This may not be possible, but it should be tried. Further investigation and experimentation has not been done here as time does not permit it.