# THE DESIGN OF A TWO-STORY BUILDING

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

Robert N. Rosso, Jr.

1944

# The Design of a Two-Story Building

A Thesis Submitted to

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of

AGRICULTURE AND APPLIED SCIENCE

by

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Candidate for the Degree of

Bachelor of Science

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Acknowledgments

I wish to thank all the members of the Civil Engineering Staff for the aid they have given me in the completion of this design problem. I also wish to thank M. M. Caldwell, Lansing Building Inspector, and O. J. Munson, Lansing Architect, for their assistance on this problem.

I have used several text books to obtain standards, actual construction hints, and design methods. They are as follows: Structural Theory by Hale Sutherland and H. L. Bowman; Structural Design by Carlton T. Bishop; Steel Construction ( A. I. S. C. ); Wood Trusses, a National Lumber Manufacturer's Association Publication; Construction Estimates and Costs by H. R. Pulver; Reinforced Concrete Design by Hale Sutherland and Raymond C. Reese; and The ACI Building Code.

Contents

Purpose of the Thesis:

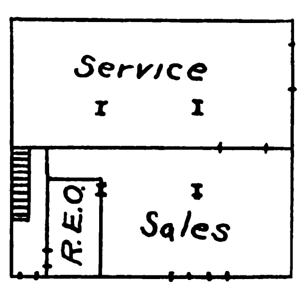
The purpose of this thesis is to design a two-story building, with a sixty foot frontage and fifty-five foot depth. The design is one that involves both an engineering and economic viewpoint and may be regarded as an actual problem that might face an individual with college training.

The building is being designed so that it may be used for many purposes in a town of approximately five to ten thousand population.

The first floor of this building may be used for such purposes as: automobile sales and service, super food market, farm equipment sales and service, or frozen food lockers. It may also be grouped into two or three stores such as: bakery, electrical appliances sales and repairs, auto parts store, and real estate office.

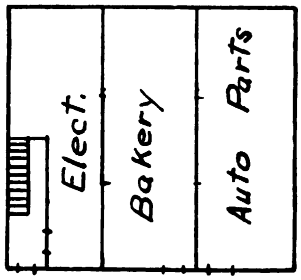
The second floor may be rented or leased as one unit to a lodge or similar organization; or it may be divided into business offices and apartments.

With the above combinations in mind, some of the possible floor plans are: ( Scale 1" equals 40' )



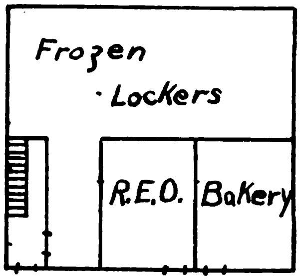

Auto Sales and Service
Real Estate Office ( R. E. O. )

First Floor



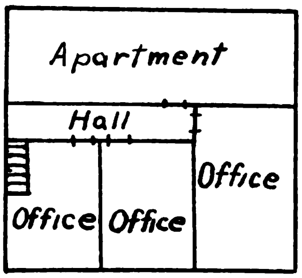

Electrical Appliances
Auto Parts
Bakery

First Floor



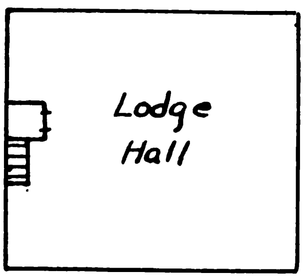

Frozen Food Lockers
Real Estate Office ( R. E. O. )
Bakery

First Floor




Apartment
Business Offices

Second Floor

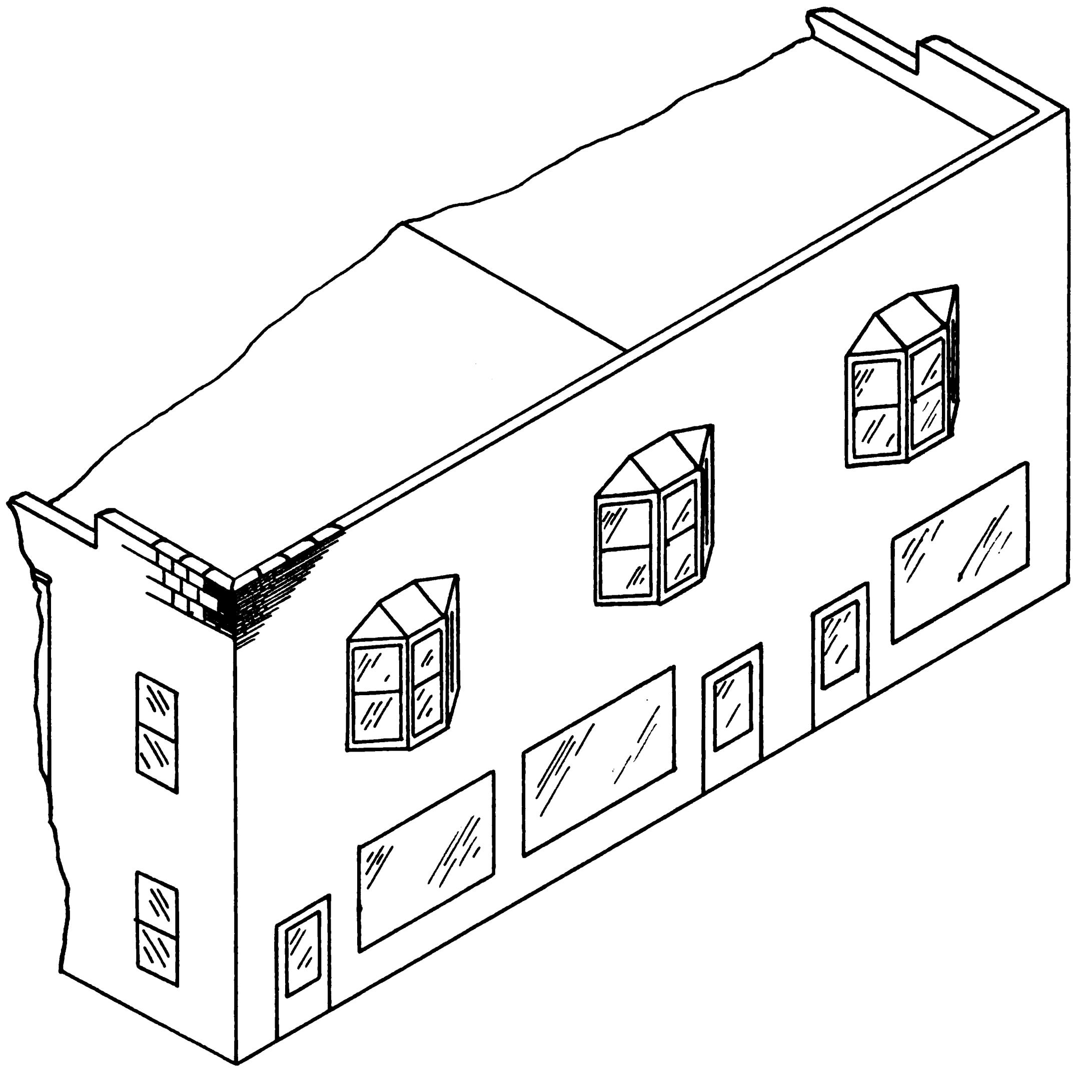



Lodge Hall

Second Floor

The preceding sketches illustrate the type of building to be constructed and also aids in the possible architectural design of the building.

Following the sketch of the proposed building, the thesis will continue with the discussion of the individual design steos taken.



Proposed Building

( Scale 1" equals 12' )

Roof Design:

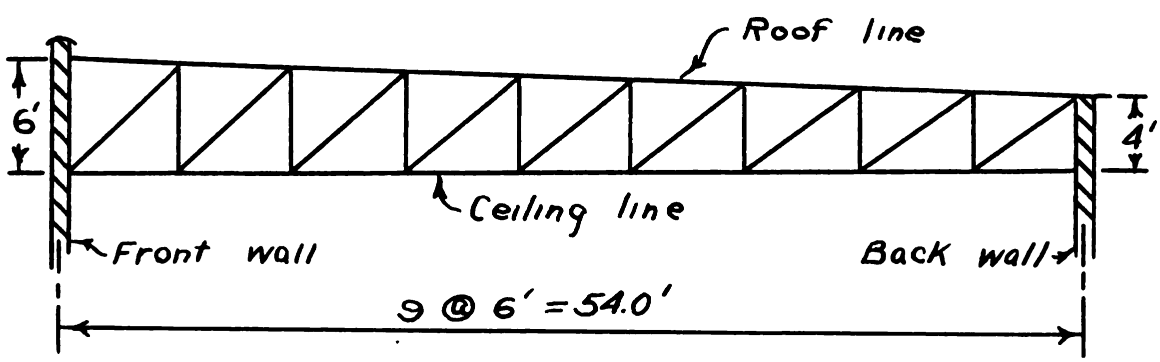
In the roof design, two assumptions were made that will have to be verified before actual construction of the building. One assumption was that the second floor was to be rented as one unit not for living purposes. With this in mind, the window specifications of the locality concerned must be consulted if the second floor is going to be used for apartments. This might necessitate the building of a skylight but it would not effect the truss design due to the fact that the material of the skylight will weigh approximately the same as the roofboards, rafters, and the surfacing of the original design. The second assumption was that the camber of the span would be negligible. This does not mean that it is negligible but, due to the time allotted to the senior problem, it will be impossible to check the truss for camber.

Another assumption was made before the actual computations were started, this being the assumed weight of the roof and truss. The estimated value was fifty pounds per square foot of roof area which was divided up as follows:

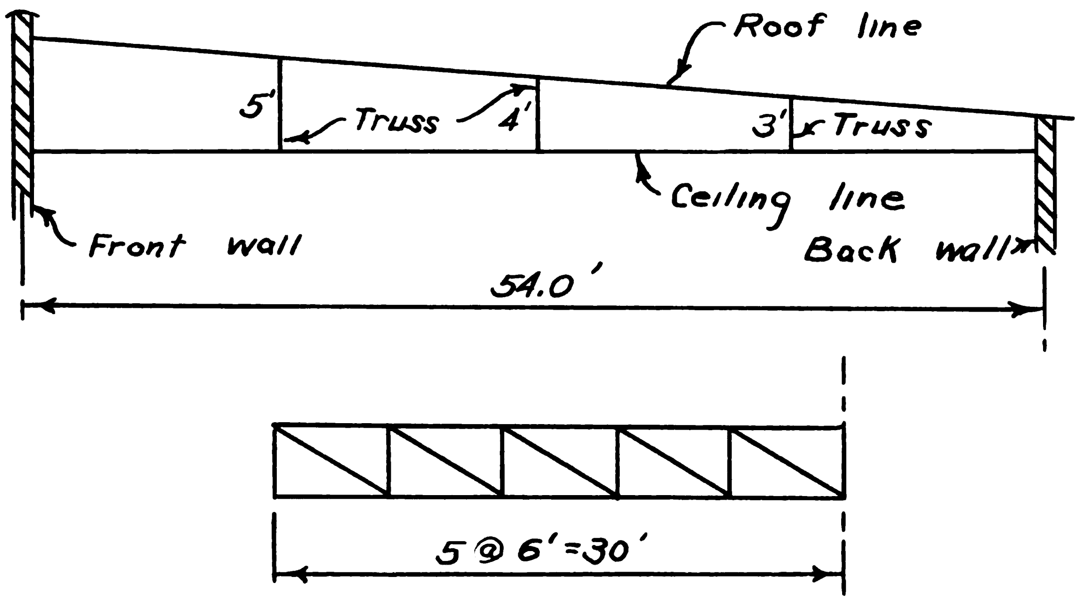
| | | |
|---|---|---|
| L. L. | 30#/sq.' | Snow load |
| D. L. | 8#/sq.' | Roofboards, rafters, surfacing |
| D. L. | 7#/sq.' | Steel |
| D. L. | 5#/sq. ' | Plaster |

Three different truss systems were designed: the first consisted of two trusses placed at the third points of the building front and running perpendicular to the front; the second is composed of three trusses sixty feet long, but with

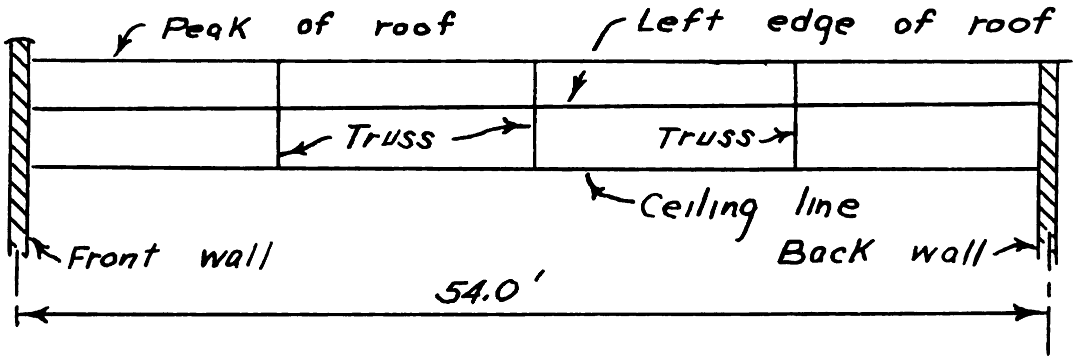
varying heights of three, four, and five feet which are running parallel to the front of the building; the third consists of three identical trusses running parallel to the store fronts. Sketches of the designs are as follows:
( Scale 1" Equals 10' )

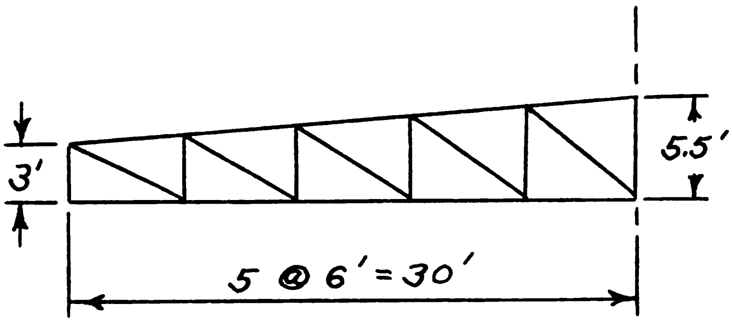



First Design




Second Design




Third Design

After the three truss systems were completely designed, it was decided that the third design was the one to be used in the construction of the structure. Each design was given due consideration and was analyzed as follows:

Truss design number one was running perpendicular to the building front and, for the proper design of the structure, it is necessary to have the trusses rest on pilasters which would have to be built at the front and back of the building. This would not permit a possible large door opening in the front of the building which might be necessary if floor space be rented for an auto or farm equipment salesroom.

The second truss system was not used because it involved the construction of three different size trusses which would increase the truss cost.

The third and final design was chosen because it ran parallel to the building front which eliminates the pilasters from the front of the building. This truss system also has three identical trusses which will give the cheapest material cost.

The calculation sheet for this truss system will now be discussed. Many of the numerical answers will not be repeated in this section of the thesis but complete details appear in the computation section under Roof Design.

The length of the truss is sixty feet and it is divided into ten panels of six feet each. With the assumed load and the known panel lengths, a total load of 4050 pounds was obtained for each panel.

With the panel load known, the truss was then analyzed for the stress values of its members. The majority of the stress values were obtained by the three main methods used in design work--the summation of moments about a point equals zero, the summation of the horizontal forces equals zero, and the summation of the vertical forces equals zero. The other method used in the analysis was that of the free-body.

After obtaining the stresses in the members, the structural shapes were selected. Two angles were used for every member, with the angles placed back to back. The angles were selected for members in both tension and compression as specified in "Steel Construction" with three-quarter-inch rivets being assumed.

Next the gusset plates were sketched out. Half-inch material was used for the plates and the details appear on the drawing showing the complete roof design.

The roofboards and roof-rafters were then designed, the rafters being fourteen feet long due to the fact that the trusses are spaced thirteen and one-half feet apart. No length specification for the roofboards was given because they are placed end to end.

The assumed weight was checked next. A one foot section of the truss was taken directly through the center of the truss to determine the weight of the steel to aid in checking the assumed load. As is shown in the computations, the actual load is slightly larger than the assumed load. This is not considered alarming, however, due to the high snow load considered for localities around the central and southern part of the state.

Floor Design:

The second main design step in the designing of this building was that of computing the second floor slab, its supporting members, columns, its column footings, the first floor slab, and the basement.

The first steps in the design of the second-floor were that of arranging a possible floor-beam and girder plan then assuming a likely live and dead load for the floor. The next step was that of making a floor entry for the stairs that would prove serviceable for a number of occasions such as that of accomodating large crowds and clearance for the moving of office furniture and supplies. (The stairs were designed with the standard seven-and-one-half inch riser with a nine inch tread.)

After that was completed, a four inch floor slab design was computed. The reinforcement steel running from the front to the back of the building was then computed. (This included both positive and negative steel.) Since this steel was running in one direction, it was necessary to use temperature steel. Its calculations are also shown in the computation section.

With the second-floor slab designed, the next step was that of determining the structural sizes of the floor beams and girders. The 12 WF was used throughout to give a uniform thickness to the floor design of the second floor and a uniform first floor ceiling line. Beam connectors were also designed to fasten floor beams and girders.

The columns supporting the second floor were then designed. Their design coming almost directly from the A. I. S. C. tables. The design of the column footings followed, and their design was taken from the method described by Sutherland and Reese. It involves the use of formulas which they developed in their text; for future reference their text should be consulted.

The first-floor slab was then designed by using a concentrated load spread over a six inch square. The proportion of the floor over the basement was computed on a live and dead load basis, however. Reinforced steel was placed in just one direction except over the basement where temperature and negative steel were used. The girder system supporting the slab over the basement was designed similar to that supporting the second-floor slab.

The basement slab was not designed due to the fact that it will act as a wearing surface mainly. The five inch slab will prove sufficient for the basement.

Store-front Design:

This design was submitted to enable a possible large door opening in the front of the building, if needed.

Foundation Design:

The foundation was designed with the walls considered as being composed entirely of blocks. The second floor also gives an additional weight to the wall; this was considered by assuming that half the floor beam span was reacting down the wall, with the total floor weight being used.

The proportion of the building over the basement gives a three-story arrangement; therefore, a sixteen-inch wide outer wall must be used for the basement. The rest of the building was considered as a two-story building and only a twelve-inch wall was used. This arrangement conforms with the building specifications of Lansing, Michigan.

No steel was used in the foundation because of the light wall loads.

The basement wall is composed of blocks, and the foundation is of concrete as shown in the computation sketch.

The eight-inch block wall, used for the interior wall of the basement, requires no foundation because there is no wall load. However, the foundation design given for this wall should be used to give security.

The five-inch basement-floor slab should be poured with the foundation around the basement.

Lintel and Bearing Plate Design:

The problem of designing the lintels was a simple one, and the designs are given in the computation section. The lintels were made of either a channel or angle welded to a plate one-quarter-inches thick and nine-inches wide.

The bearing plate design method used throughout is that prescribed by the A. I. S. C.. The three-quarter-inch plate was used in all cases, because this was the thinnest plate advisable to use from a practical standpoint.

The plates are to be set in poured concrete with anchor bolts. The holes in the blocks supporting these plates should be filled with concrete from the foundation on up. The proportion of the structure resting on these plates will be tack-welded in place.

Specifications:

The building is one of two-stories and was described in the "Purpose of the Thesis". It was the purpose of the thesis to design the building; such matters as detailing and forming the bill of material were not undertaken, but may be easily obtained by careful analysis of the computations.

The foundation is to be of poured concrete throughout. The front wall is to be made with a brick face and an eight-by-eight back-up block. The other three walls are to be made of eight-by-twelve-by-sixteen standard block. The outer basement wall is to be made sixteen-inches thick, with the inner basement wall eight-inches thick.

All concrete is to have at least three thousand pounds per square inch compressive unit stress in the extreme fiber of concrete in flexure. Three-quarter-inch aggregate will be the maximum allowed in the mix.

All the lumber used in the construction of this building should have at least a twelve thousand pound per square inch bending stress in the extreme fiber.

Steel used should be of the twenty thousand pound per square inch category, except the reinforcement steel, which is to be deformed bars with at least an eighteen thousand pound per square inch value for tension.

The roof is to be made of roofboards, heavy felt--not less than thirty pound felt--, two-ply felt is then to be lain and mopped with asphalt pitch. Next, special mesh is to be laid with asphalt pitch and a cover of crushed slag or one-quarter to five-eighth-inch well screened gravel. The roof is to be flashed with sixteen ounce copper at the wall boundaries and the peak.

The stairs are to be of concrete with permanent steel forms being used if possible.

Plastering is to be done on metal lath, tack-welded to the steel. ( It is more than likely that small angles will have to be added to give the proper support, but this will be left for the detailer to decide, however. )

Building tile is to be used in partitioning the walls for the separate divisions--four-inch tile will be suitable, but this may vary.

All windows should be of three-foot sash width and may be of wood or steel, but this should be decided upon by the future owner before the building is detailed.

Four-foot doors are to be used in the building front, but three-foot doors may be used in the interior.

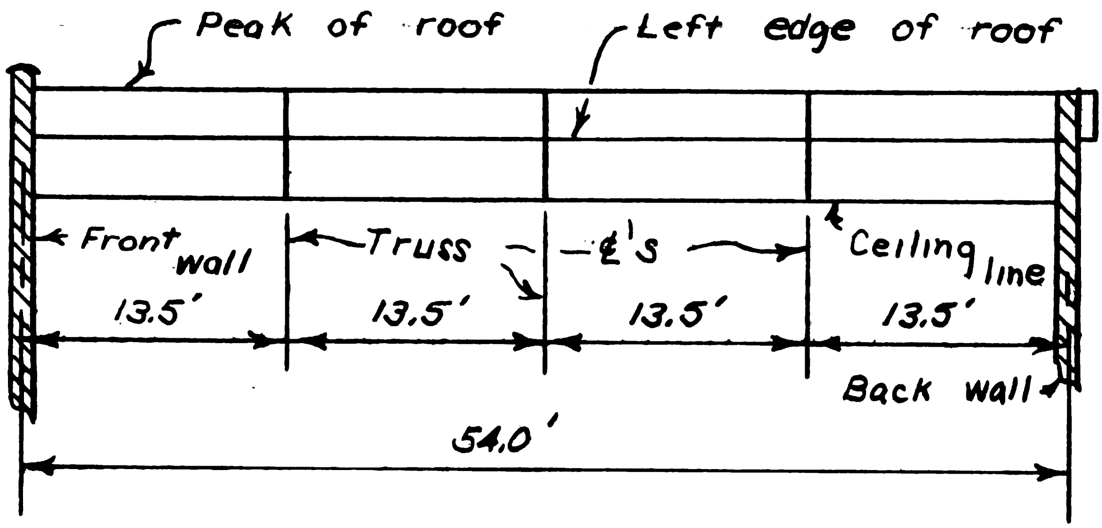
Heating, plumbing, electrical wiring, basement drainage, and any other item that has not been discussed will be left up to the discretion of the owner or contractor, which ever the case may be.

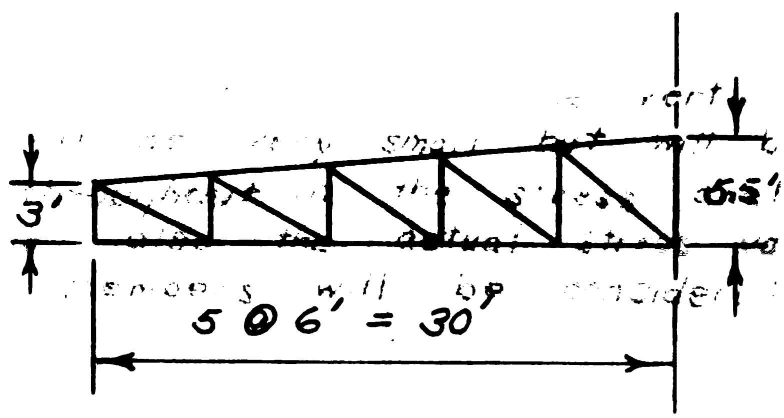
The building specifications for the locality where the building might be erected should be consulted before the complete building arrangements are made.

Computations

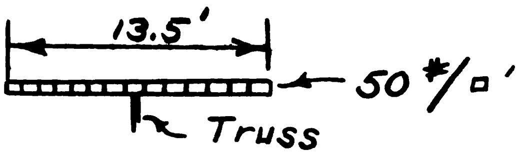
# ROOF DESIGN :

Three roof trusses running parallel to the store fronts.

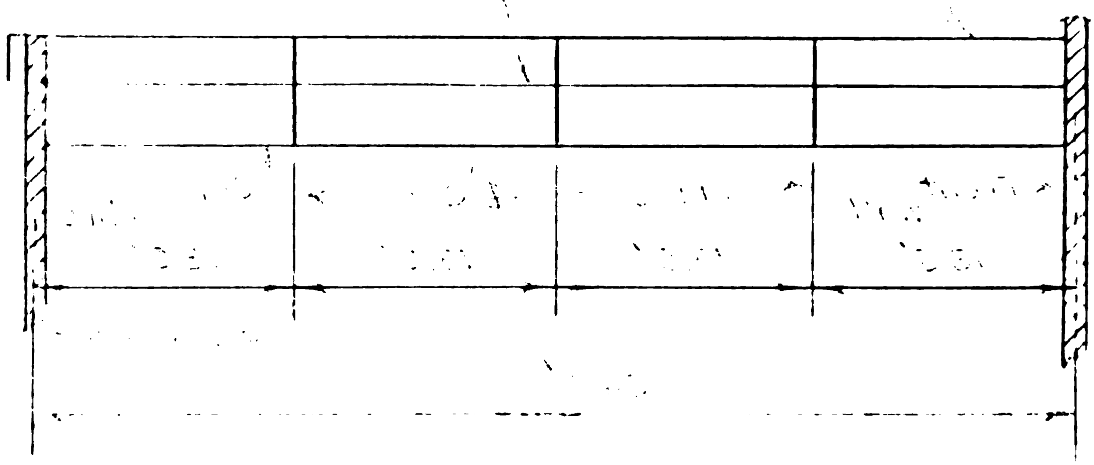


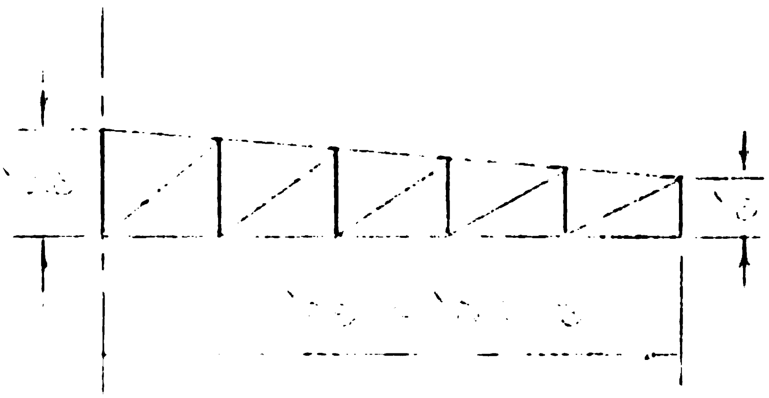



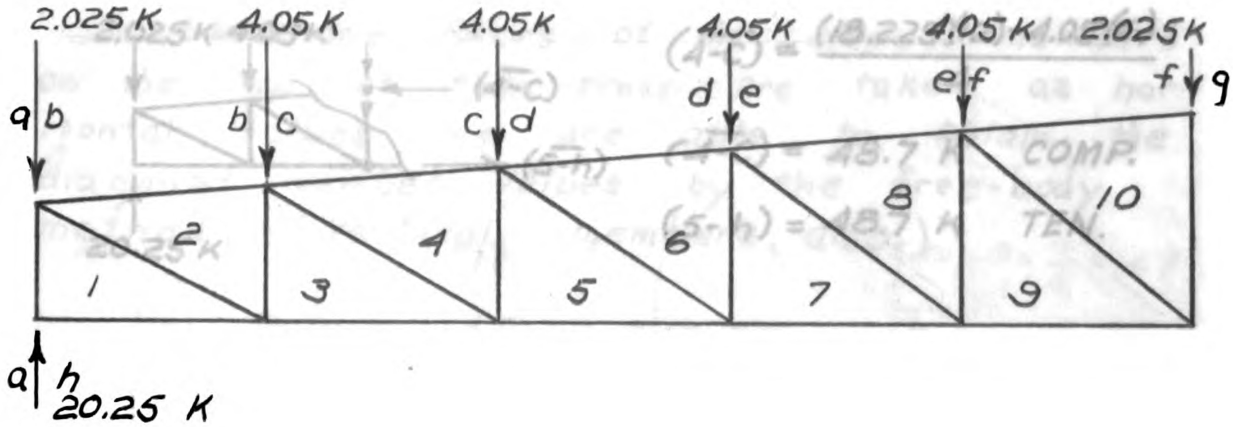

Considering a total Live & Dead load of 50#/□' of roof area, and a truss with 6' panels; we obtain a total load of 4050# on each panel.




$50(13.5)(6) = 4050^{\#}$

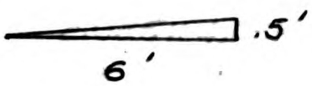






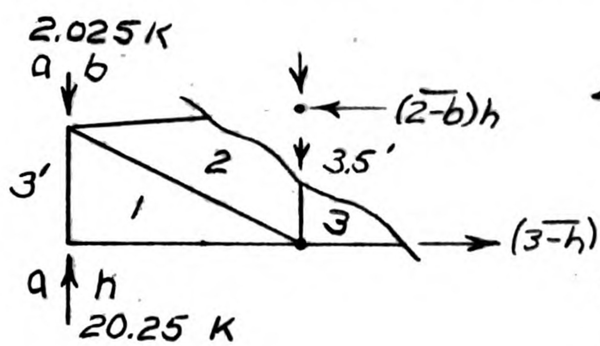

The $50^{\#}/\square'$ is considered as acting in a vertical direction to the roof surface, thereby producing a vertical load of 4050# on each panel.

.5' rise in each panel gives an ∠ of approximately 4.8°.




$\tan\theta = \frac{.5}{6}$ $\theta = 4.8°$

Due to the small ∠ the vertical load change will be very small but will be considered throughout in the stress analysis of the truss; also the actual stress value of the top members will be considered as having the same value as the horizontal component, because of the natural cos value .997.

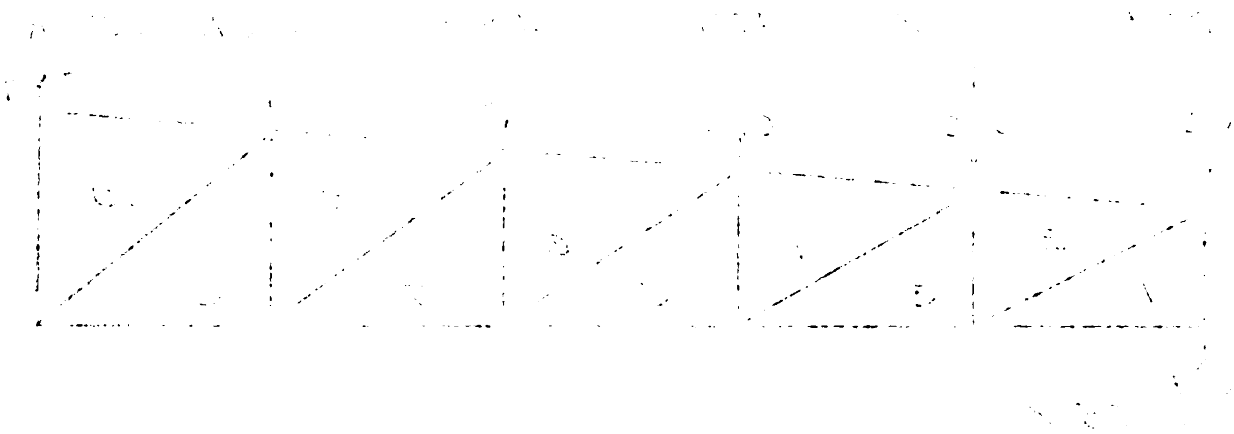


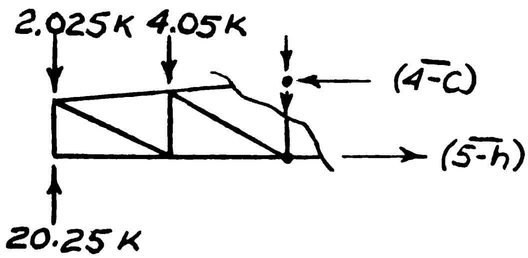

$\Sigma M = 0$

$$(\overline{2-b})_h = \frac{(18.225)(6)}{3.5}$$

$(\overline{2-b}) = 31.2$ K COMP.

$(\overline{3-h}) = 31.2$ K TEN.

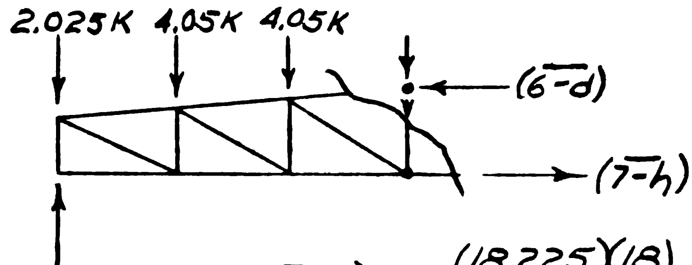





$$(\overline{4-c}) = \frac{(18.225)(12) - 4.05(6)}{4}$$

$(\overline{4-c}) = 48.7$ K COMP.

$(5-h) = 48.7$ K TEN.

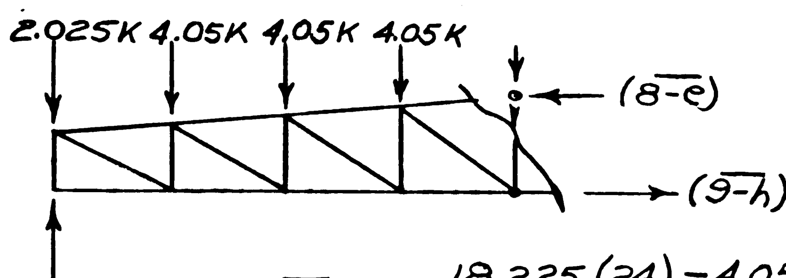



20.25 K

$$(\overline{6-d}) = \frac{(18.225)(18) - 4.05(6+12)}{4.5}$$

$(\overline{6-d}) = 56.7$ K COMP.

$(\overline{7-h}) = 56.7$ K TEN.

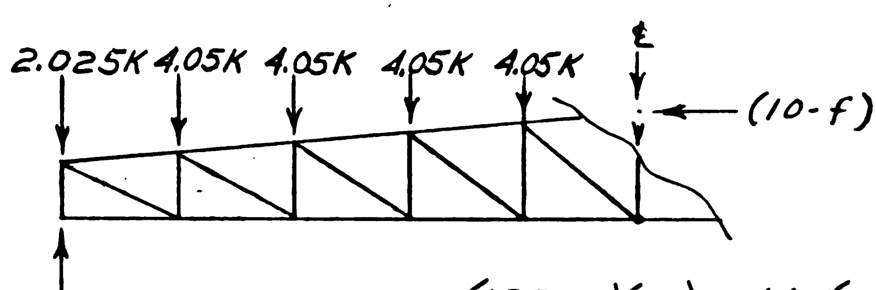



20.25 K

$$(\overline{8-e}) = \frac{18.225(24) - 4.05(6+12+18)}{5}$$

$(\overline{8-e}) = 58.5$ K COMP.

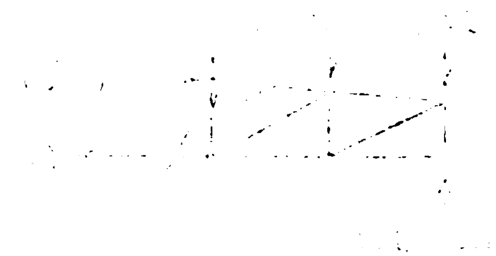
$(\overline{9-h}) = 58.5$ K TEN.

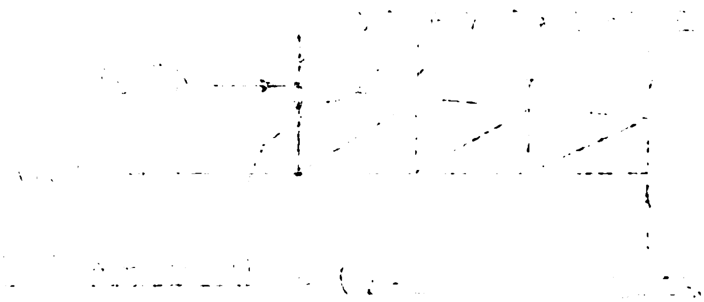


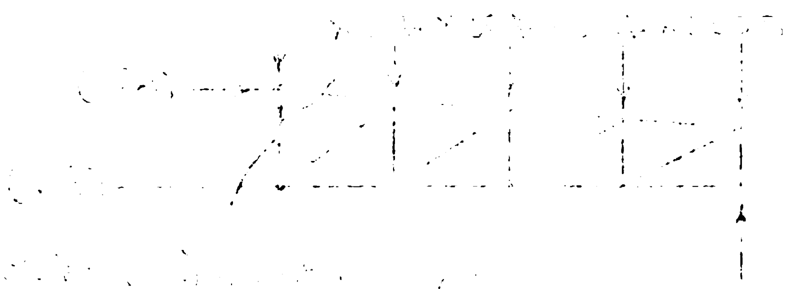

20.25 K

$$(\overline{10-f}) = \frac{(18.225)(30) - 4.05(6+12+18+24)}{5.5}$$

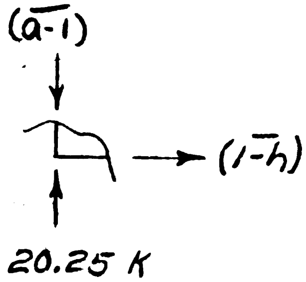
$(\overline{10-f}) = 55.3$ K COMP.







Since the forces of the sloped members on the top of the truss are taken as horizontal values we are able to obtain the diagonal member values by the free-body method. (Vertical members, also.)

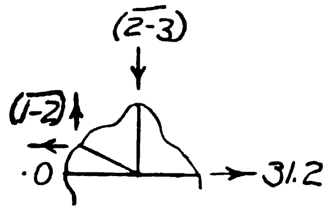



$\Sigma V = 0$

$(\overline{a\text{-}1}) = 20.25\ K$ COMP.

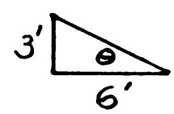
$\Sigma H = 0$

$(\overline{1\text{-}h}) = 0$




$\Sigma H = 0$

$(\overline{1\text{-}2})_h = 31.2$



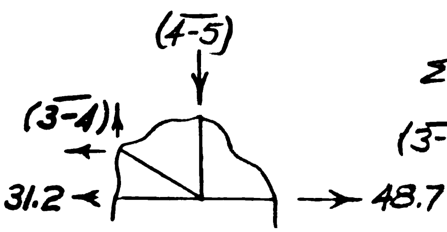

$(\overline{1\text{-}2}) = \dfrac{31.2}{\cos 26.6^\circ}$

$\tan\theta = \frac{3}{6} = .5$

$(\overline{1\text{-}2}) = 35.0\ K$ TEN.

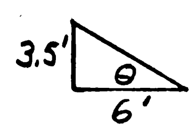
$\theta = 26.6^\circ$

$(\overline{2\text{-}3}) = 35.0(\sin 26.6^\circ) = 15.6\ K$ COMP.




$\Sigma H = 0$

$(\overline{3\text{-}4})_h = 48.7 - 31.2$



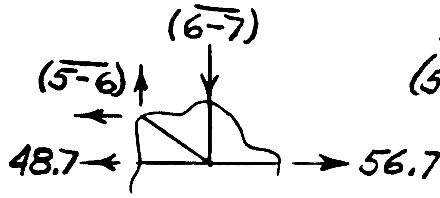

$(\overline{3\text{-}4})_h = 17.5$

$\tan\theta = \frac{3.5}{6} = .583$

$(\overline{3\text{-}4}) = \dfrac{17.5}{\cos 30.2^\circ} = 20.3\ K$ TEN.

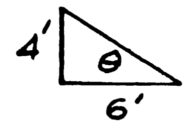
$\theta = 30.2^\circ$

$(\overline{4\text{-}5}) = 20.3(\sin 30.2^\circ) = 10.25\ K$ COMP.




$\Sigma H = 0$

$(\overline{5-6})_h = 56.7 - 48.7$



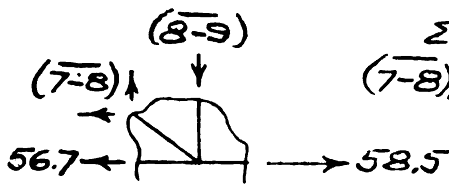

$(\overline{5-6})_h = 8.0$

$\tan\theta = \frac{4}{6} = .67$

$(\overline{5-6}) = \frac{8}{\cos 33.7^\circ} = 9.63\ K$ TEN.

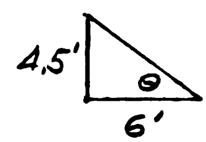
$\theta = 33.7^\circ$

$(\overline{6-7}) = 9.6(\sin 33.7^\circ) = 5.35\ K$ COMP.




$\Sigma H = 0$

$(\overline{7-8})_h = 58.5 - 56.7$



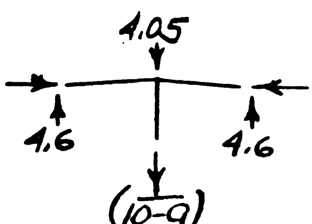

$(\overline{7-8})_h = 1.8$

$\tan\theta = \frac{4.5}{6} = .75$

$(\overline{7-8}) = \frac{1.8}{\cos 36.9^\circ} = 2.25\ K$ TEN.

$\theta = 36.9^\circ$

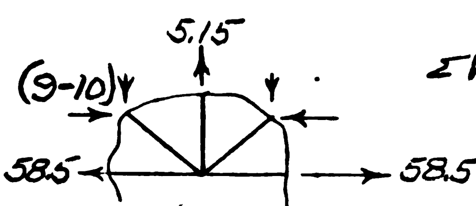
$(\overline{8-9}) = 2.25(\sin 36.9^\circ) = 1.35\ K$ COMP.




$\frac{6}{55.3} = \frac{.5}{X}$ $X = 4.6$

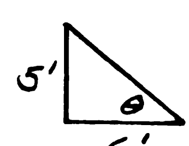
$\Sigma V = 0$ $4.6 + 4.6 - 4.05 = (\overline{10-9})$

$(\overline{10-9}) = 5.15\ K$ TEN.




$\Sigma V = 0$

$(\overline{9-10})_v = \frac{5.15}{2}$



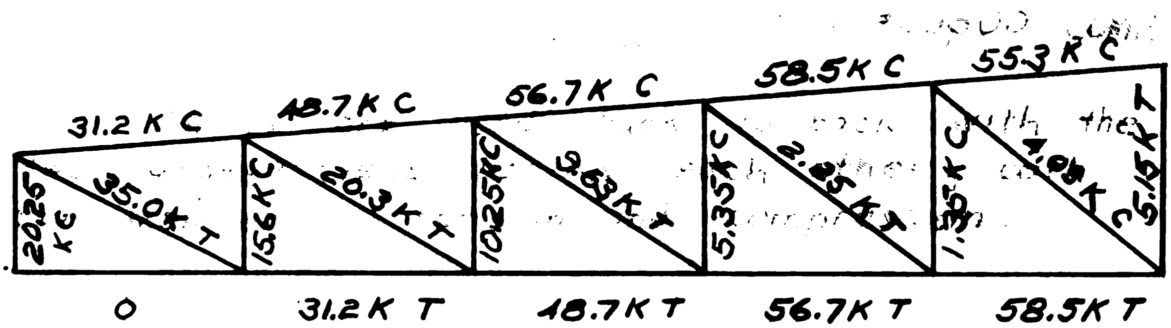

$(\overline{9-10})_v = 2.575$

$\tan\theta = \frac{5}{6} = .833$

$\theta = 39.8^\circ$

$(\overline{9-10}) = \frac{2.575}{\sin 39.8^\circ} = 4.03\ K$ COMP.

Results of the stress analysis :




Material for trusses : (Structural shapes)

Using "Steel Construction" (A.I.S.C.) for the structural shapes, sizes, and specifications.

$S = 20{,}000$ #/□" for tension steel

$S = 17{,}000 - .485\ \ell^2/r^2$ for compression steel when $\ell/r < 120$

Assuming the use of 3/4" rivets.

For members in compression : (2 ∠s back to back)

$\ell/r < 120$ $\quad \ell = 6.0'$ $\quad \frac{6(12)}{120} = r_{min} = .600"$

* Try 2 - $3\frac{1}{2} \times 3 \times \frac{1}{2}$ ∠s $\quad r = 1.07"$ $\quad A = 6.0$□"

$\left(17000 - .485\left(\frac{72}{1.07}\right)^2\right) = \frac{P}{A}$ $\quad \therefore P = 88{,}700$# COMP.

* Try 2 - $3\frac{1}{2} \times 3 \times \frac{3}{8}$ ∠s $\quad r = 1.09"$ $\quad A = 4.60$□"

$\left(17000 - .485\left(\frac{72}{1.09}\right)^2\right) = \frac{P}{A}$ $\quad \therefore P = 68{,}500$# COMP.

* Try 2 - $3\frac{1}{2} \times 3 \times \frac{1}{4}$ ∠s $\quad r = 1.11"$ $\quad A = 3.12$□"

$\left[17000 - .485\left(\frac{72}{1.11}\right)^2\right] = \frac{P}{A}$ $\quad \therefore P = 48{,}000$# COMP.

* Try 2 - $2\frac{1}{2} \times 2 \times \frac{1}{4}$ ∠s $\quad r = .78"$ $\quad A = 2.12$□"

$\left[17000 - .485\left(\frac{72}{.78}\right)^2\right] = \frac{P}{A}$ $\quad \therefore P = 27{,}200$# COMP.

* Try 2 - $2\frac{1}{2} \times 2 \times \frac{3}{8}$ $\angle$s   $r = .77$   $A = 3.10\square''$

$$\left(17000 - .485\left(\frac{72}{.77}\right)^2\right) = \frac{P}{A} \qquad \therefore\ P = 39{,}500\ \#\ \text{COMP.}$$

All $\angle$s are used back to back with the two longest legs facing each other, both for member in tension and compression.

For members in tension:

$S = \dfrac{P}{A}$   A = Area of the $\angle$s − the area of the rivet holes.

Area of 2 $\angle$s
$3\frac{1}{2} \times 3 \times \frac{3}{8} = 4.60\square''$
$3\frac{1}{2} \times 3 \times \frac{1}{4} = 3.12$
$2\frac{1}{2} \times 2 \times \frac{3}{8} = 3.10$
$2\frac{1}{2} \times 2 \times \frac{1}{4} = 2.12$

Reduction of area for $\frac{7}{8}''$ rivet hole.

Material thickness of $\frac{1}{4}'' = .219\square''$
$\frac{3}{8}'' = .328\square''$

$S = \dfrac{P}{A}$   For $P = 58{,}500\ \#$

$$\frac{58500}{20000} = A = 2.925\square''$$

Since $2.925 + (.219)(2) > 3.12\square''$

$\therefore$ Use 2 $\angle$s $3\frac{1}{2} \times 3 \times \frac{3}{8}$   $A = 4.60\square''$

The value $.219\square''$ is multiplied by 2 because 2 $\angle$s are used.

* For $P = 48{,}700\ \#$   $\dfrac{48700}{20000} = A = 2.435\square''$

Since $2.435 + (.219)(2) < 3.12$

$\therefore$ Use 2 $\angle$s $3\frac{1}{2} \times 3 \times \frac{1}{4}$   $A = 3.12\square''$

*For $P = 35{,}000$ #  $\frac{35000}{20000} = A = 1.75$ □"

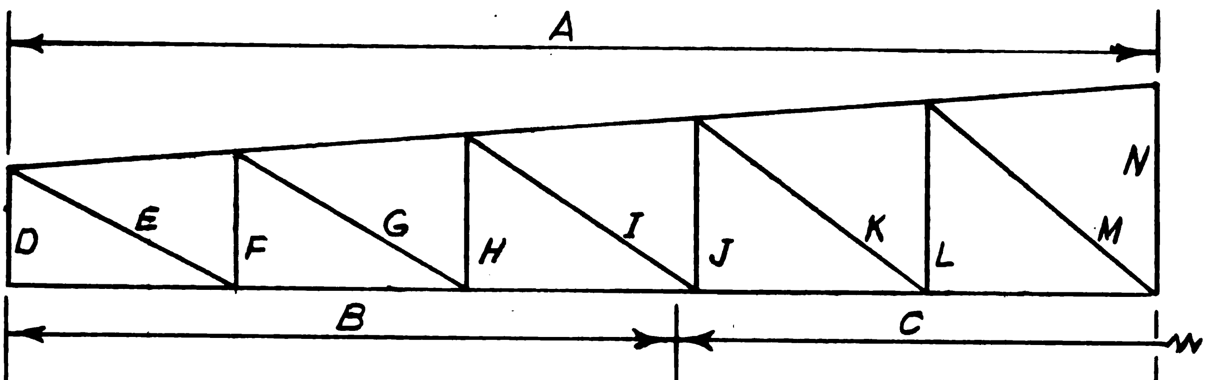
Since $1.75 + .438 > 2.12$

∴ Use 2∠s $2\frac{1}{2} \times 2 \times \frac{3}{8}$

*For $P = 20{,}300$ #  $\frac{20300}{20000} = A = 1.015$ □"

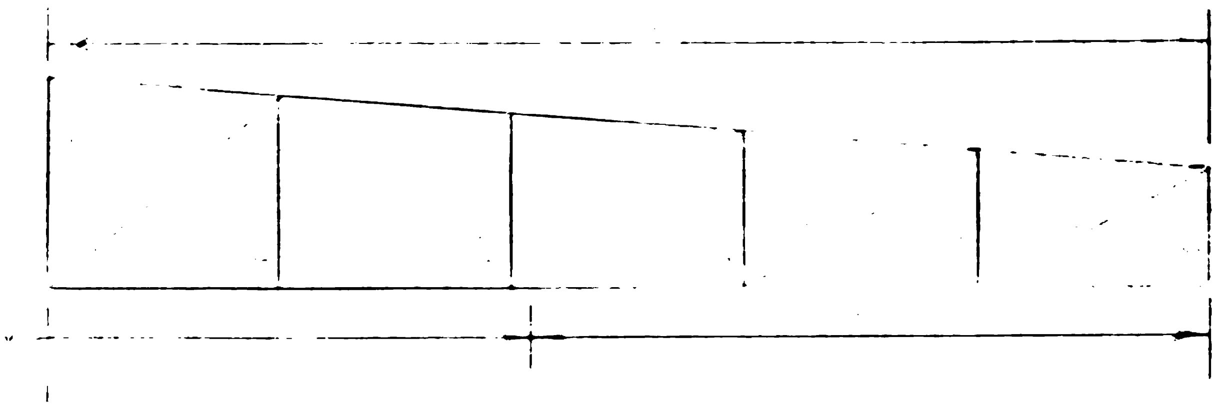
Since $1.015 + .438 < 2.12$

∴ Use 2∠s $2\frac{1}{2} \times 2 \times \frac{1}{4}$




Bill of material for one truss:

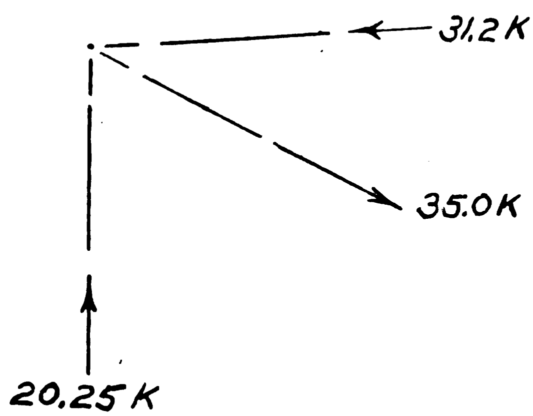
| 1 Letter | 2 No. Req. | 3 Size of ∠ | 4 Length |
|---|---|---|---|
| A | 2 | 3½ X 3 X ⅜ | 30'0" |
| B | 2 | 3½ X 3 X ¼ | 17'0" |
| C | 1 | 3½ X 3 X ⅜ | 25'10" |
| D | 2 | 2½ X 2 X ¼ | 2'6" |
| E | 2 | 2½ X 2 X ⅜ | 5'6" |
| F | 2 | 2½ X 2 X ¼ | 3'0" |
| G | 2 | 2½ X 2 X ¼ | 5'10" |
| H | 2 | 2½ X 2 X ¼ | 3'6" |
| I | 2 | 2½ X 2 X ¼ | 6'3" |
| J | 2 | 2½ X 2 X ¼ | 4'0" |
| K | 2 | 2½ X 2 X ¼ | 6'6" |
| L | 2 | 2½ X 2 X ¼ | 4'6" |
| M | 2 | 2½ X 2 X ¼ | 6'9" |
| N | 1 | 2½ X 2 X ¼ | 5'0" |



Rivets and Gusset Plates for the Roof Trusses:

Using 3/4" rivets — Shear 15000 #/□"
Bearing S.S. 32000 #/□"
D.S. 40000 #/□"

Using 1/2" gusset plates — since the smallest ∠s used are 1/4" thick, the 1/2" thickness will determine the bearing value, which is 13.1 Kip maximum load. (A.I.S.C.)




$\frac{31.2}{13.1} = 3$ Rivets

$\frac{35.0}{13.1} = 3$ Rivets

$\frac{20.25}{13.1} = 2$ Rivets

Minimum rivet spacing (A.I.S.C.) is 3 rivet diameters ℄ to ℄.

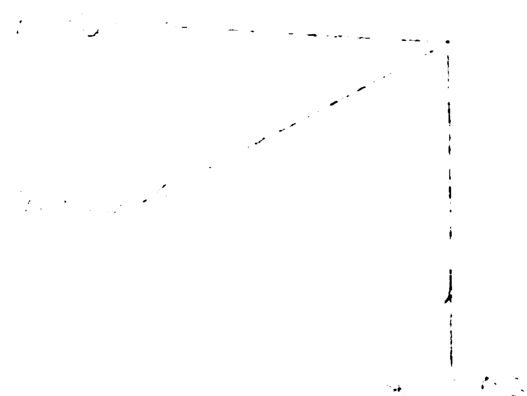
$(.75)(3) = 2.25"$ Use 2.5" ℄ to ℄

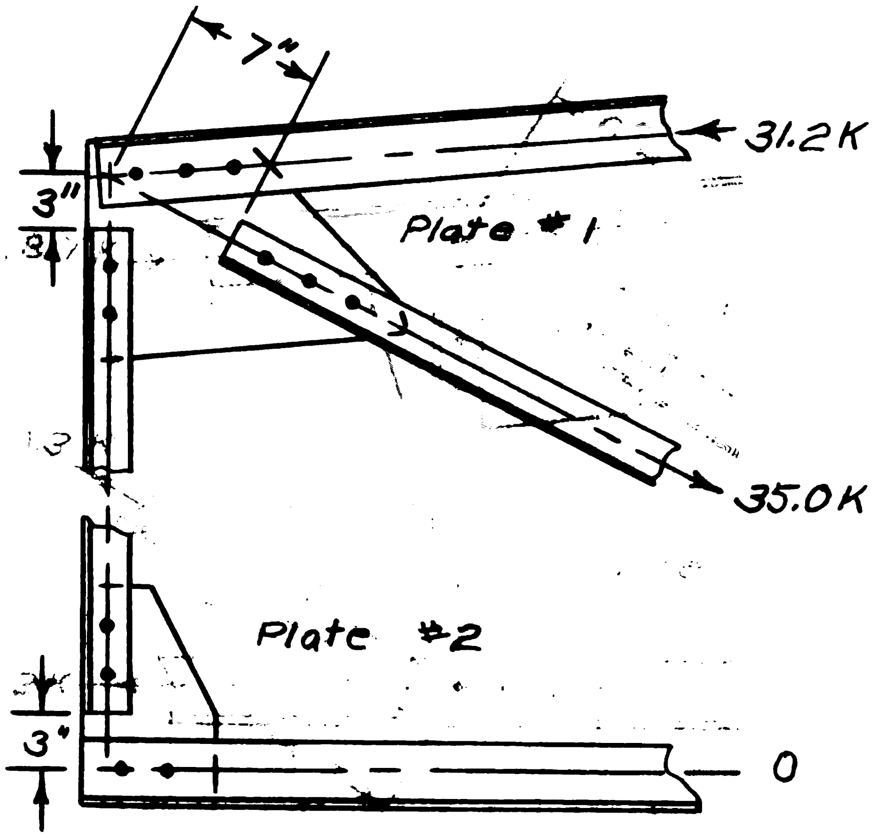
For end spacing use 2.0"

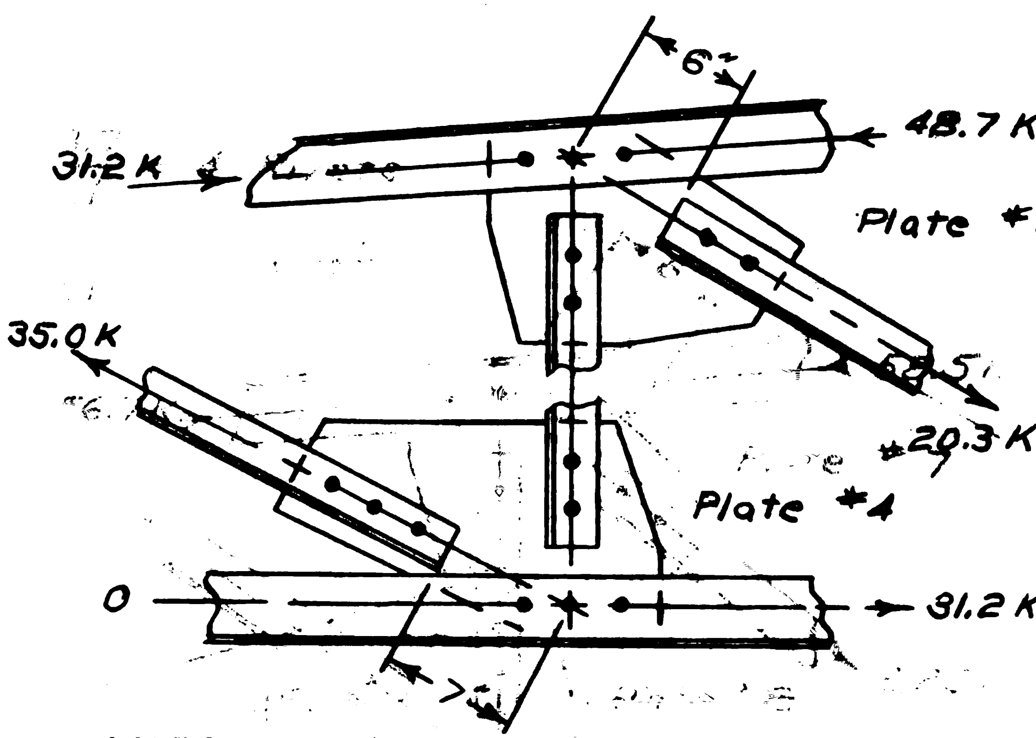
For a 1/2" gusset plate and a tension load of 35.0 K the cross-section area of the gusset plate at the 1st rivet should be—

$s = \frac{P}{A}$ $\frac{35000}{20000} = A = 1.75\,□"$

Sketches are drawn to the scale .1" = 1.0"

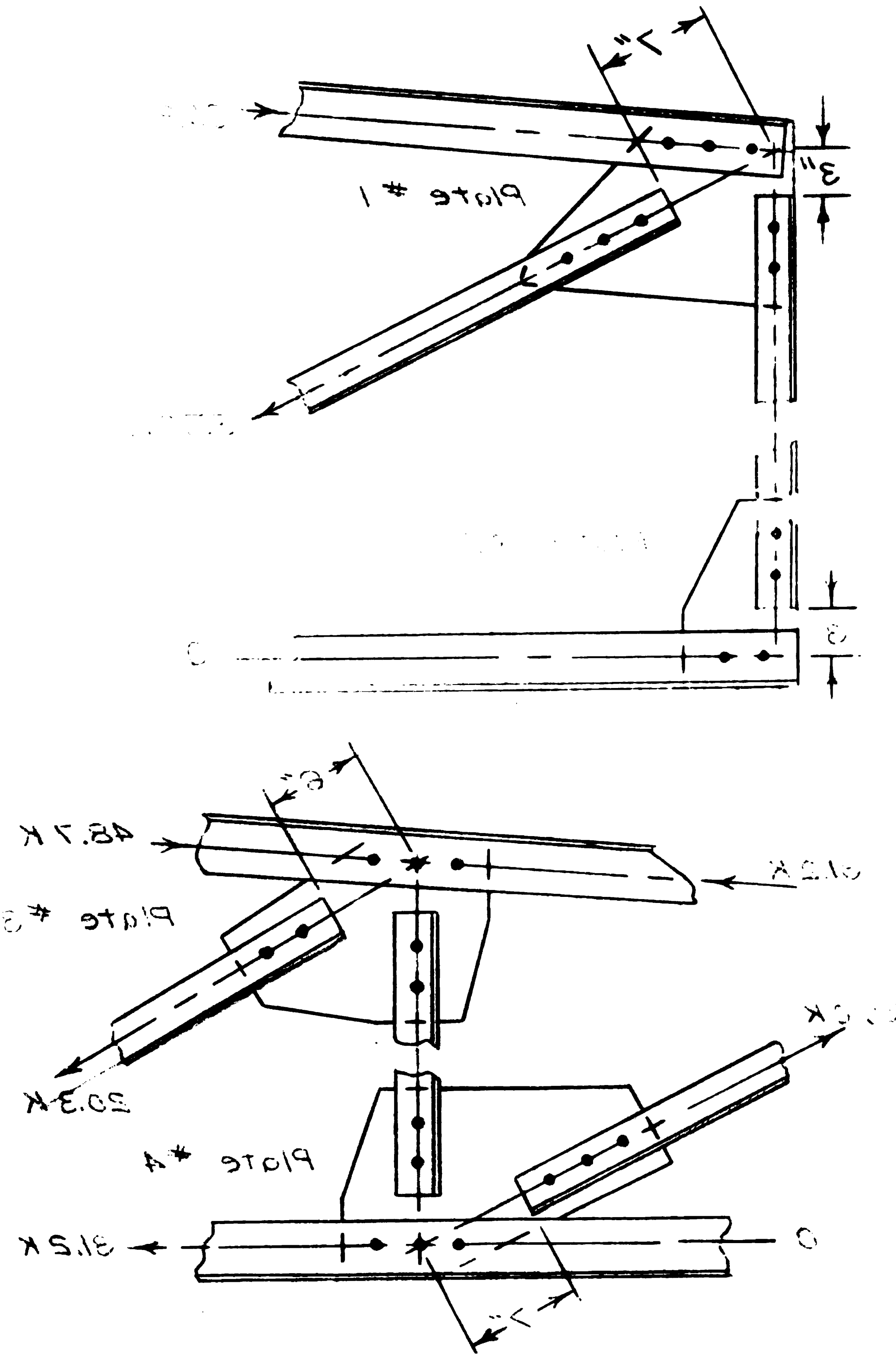




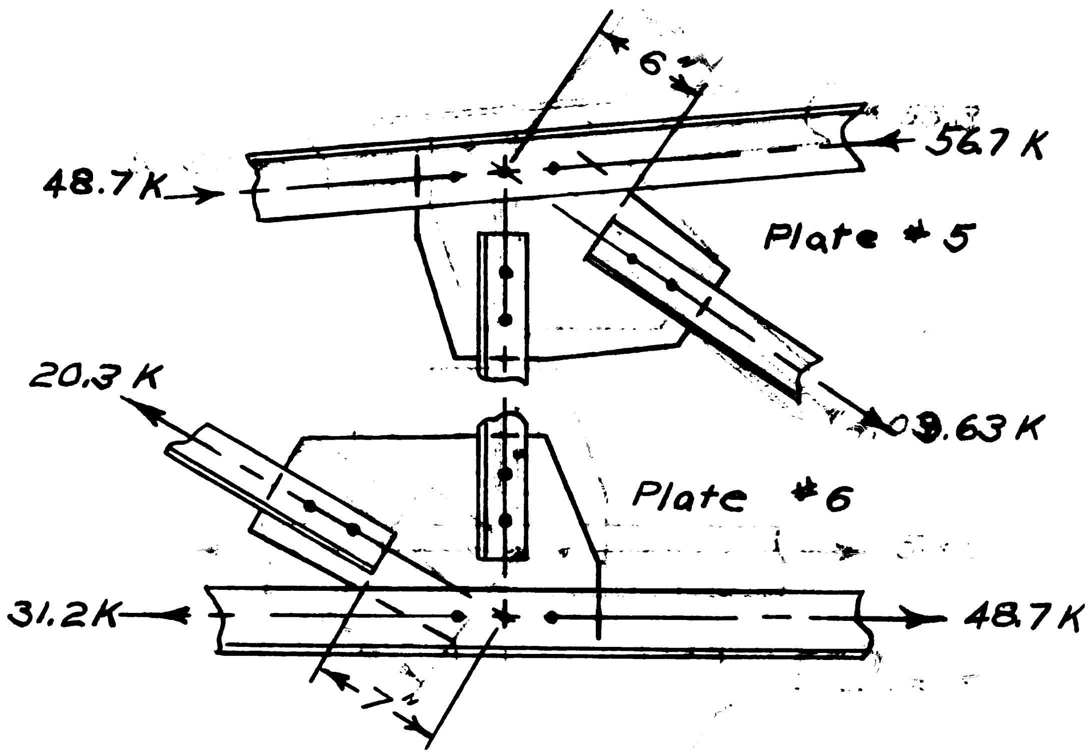




$\frac{48.7 - 31.2}{13.1} = 2$ Rivets  Use 3 Rivets

$\frac{20.3}{13.1} = 2$ Rivets  $\frac{31.2}{13.1} = 3$ Rivets

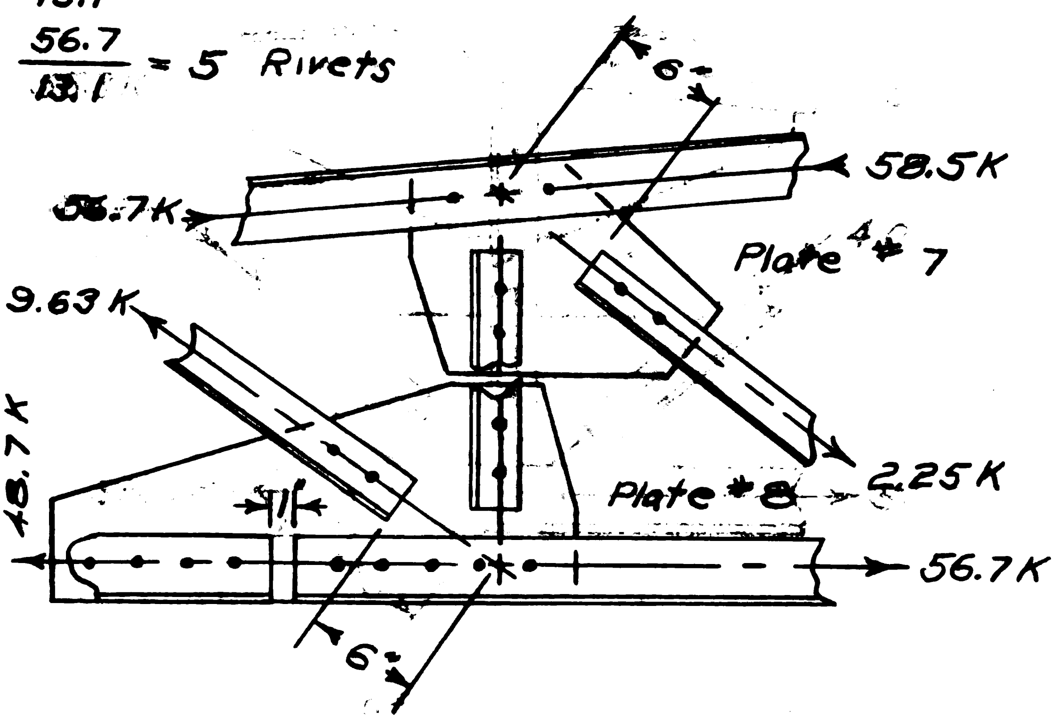


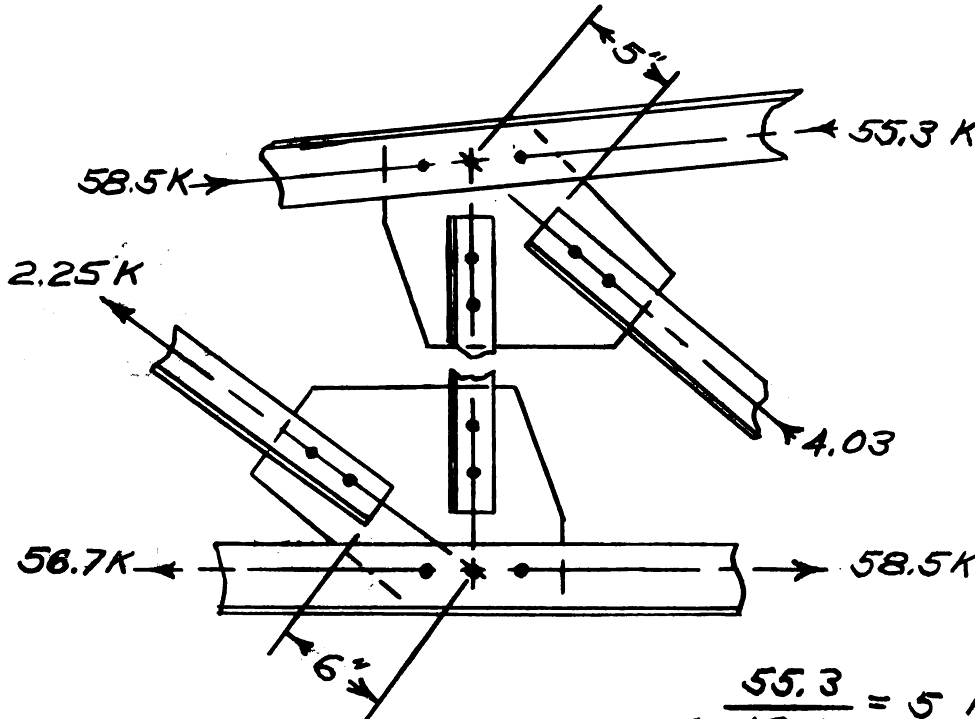
7"
3"
Plate #1
3
6"
48.7 K
31.2 K
Plate #3
20.3 K
Plate #4
31.2 K
7"



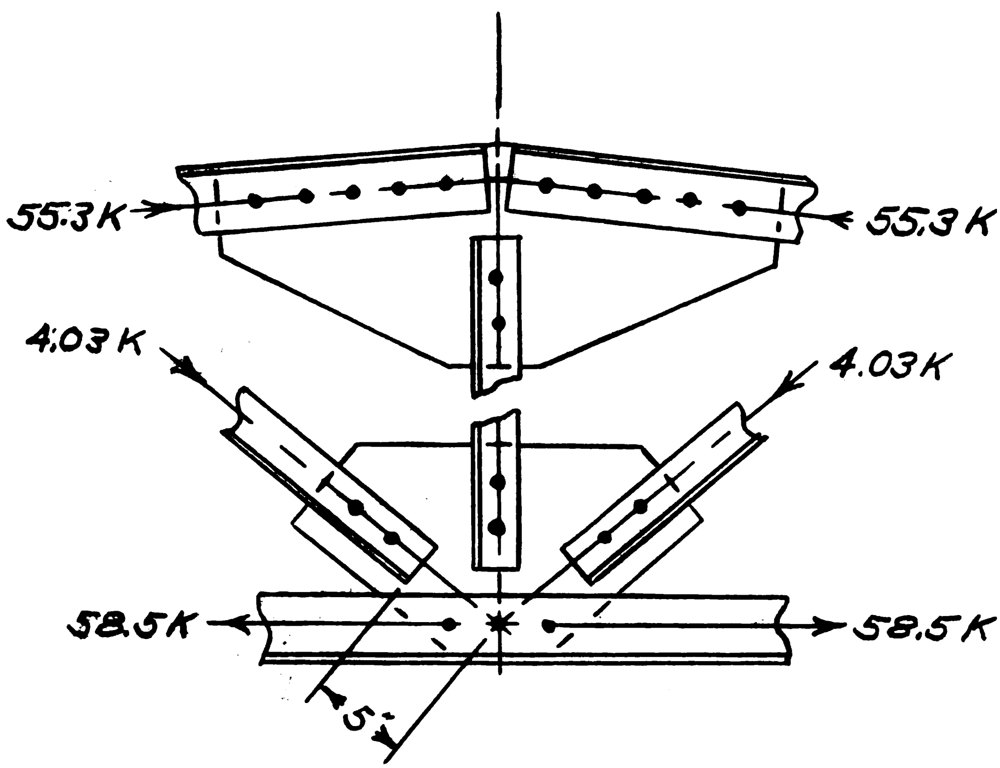

$\frac{48.7}{13.1}$ = 4 Rivets

$\frac{56.7}{13.1}$ = 5 Rivets

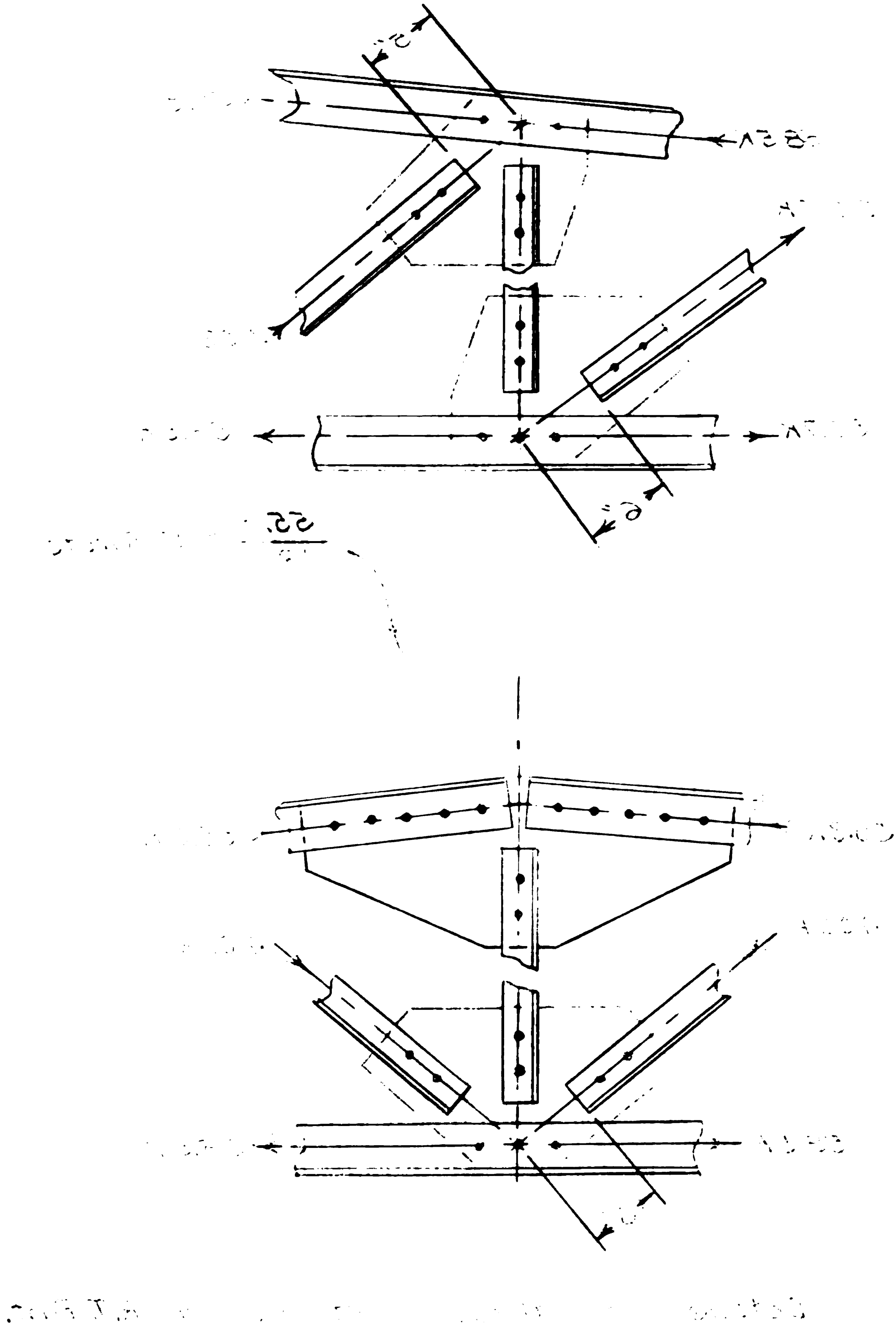





$$\frac{55.3}{13.1} = 5 \text{ Rivets}$$




Details of these sketches on R.T. Print.

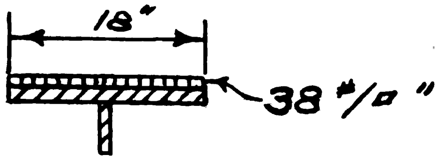


Roof rafters and roof boards :

Rafters :

$$S = \frac{Mc}{I} \qquad S = \frac{M}{S} \qquad S = \frac{I}{c}$$

Assume $S = 1200\,\#/\square''$ for wood.



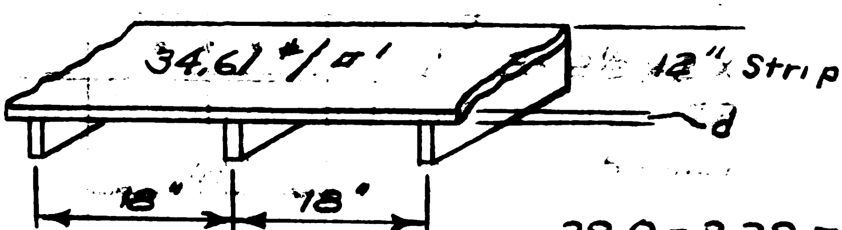

| | | |
|---|---|---|
| 30 #/□' | L.L. | snow |
| 8 #/□' | D.L. | rafters |
| 38 | | roof boards |
| | | roofing |

$$M = \frac{wl^2}{8} = \frac{(38)(\frac{18}{12})(13.5)^2}{8} = 1300\,'\#$$

$1300(12) = 15600\,''\#$ $\qquad S = \frac{15600}{1200} = 13.0\,''^3$

Use 2X8 X 14' $\qquad$ Wt = 3.39 #/'

Roof boards :




$38.0 - 3.39 = 34.61\,\#/\square'$

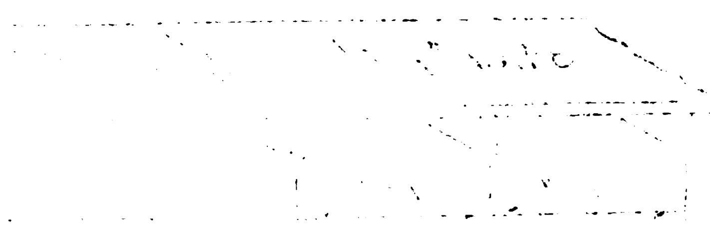
$$M = \frac{wl^2}{12} = \frac{(34.61)(1.5)^2}{12} = 6.5\,'\# \qquad S = \frac{6.5(12)}{1200}$$

$$I = \frac{bd^3}{12} \qquad c = \frac{d}{2} \qquad S = \frac{I}{c} \qquad S = .065\,''^3$$

$$.065 = \frac{\frac{12d^3}{12}}{\frac{d}{2}} \qquad .065 = 2d^2 \qquad d^2 = .0325$$

$$d = .18''$$

Use 1X8 material

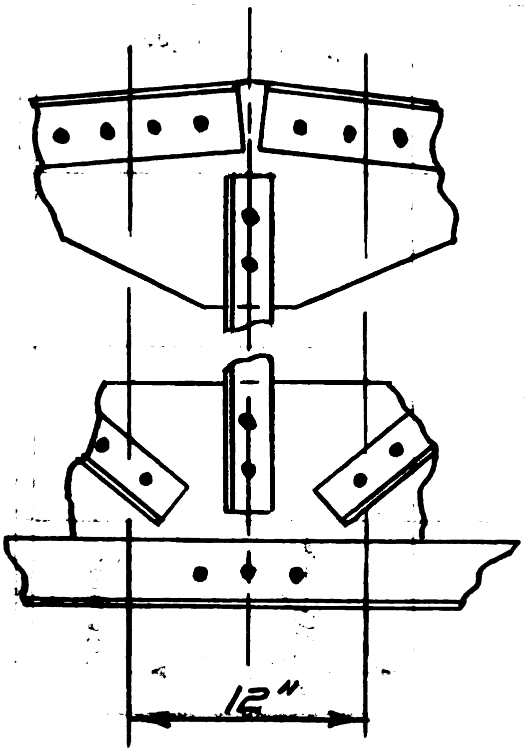


Weight of roof boards based on 40 #/cu. ft. for wood. (A.I.S.C.)

$$\frac{40}{X} = \frac{12}{1} \qquad X = 3.33 \ \#/\square'$$

Check on assumed weights:

Cut a section 1' wide down the center of the truss.




Assume G.P. steel to weigh 10#/36 cu."

$$G.P. = \left(\frac{12 \times 12 \times 1}{36}\right) 10 = 40.\#$$

Rivets = 45#/100

10 Rivets = 4.5#

2-3½ x 3 x 3/8
15.8#/1'
2(15.8) = 31.6#

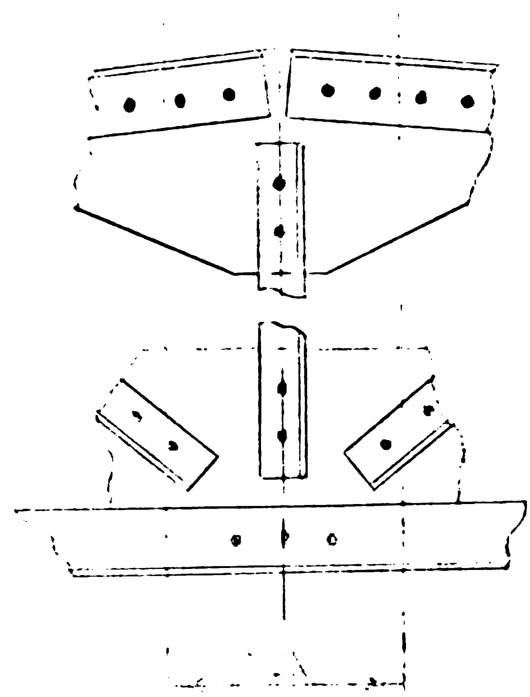
2-2½ x 2 x ¼
7.2#/1'
5(7.2) = 36.0#

112.1#

$$\text{Total } \frac{112.1}{13.5} = 8.8 \ \#/\square'$$

| | |
|---|---|
| Snow | 30.0 #/□' |
| Steel | 8.8 |
| Roof | 8.0 |
| Plaster | 5.0 |
| | 51.8 #/□' |

The check is 1.8 #/□' high but is not considered alarming due to the high snow load.

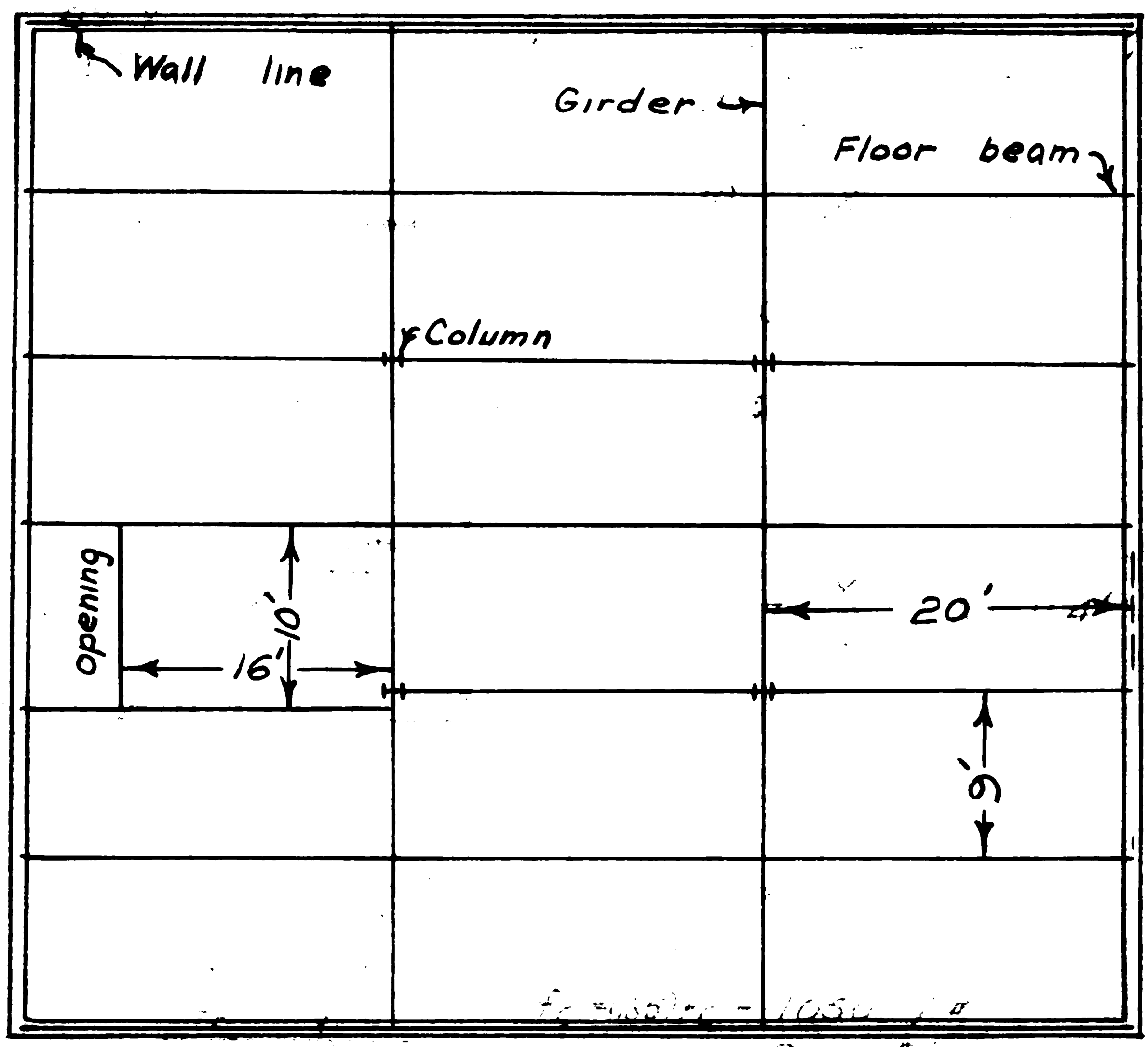


FLOOR DESIGNS :

Second floor :

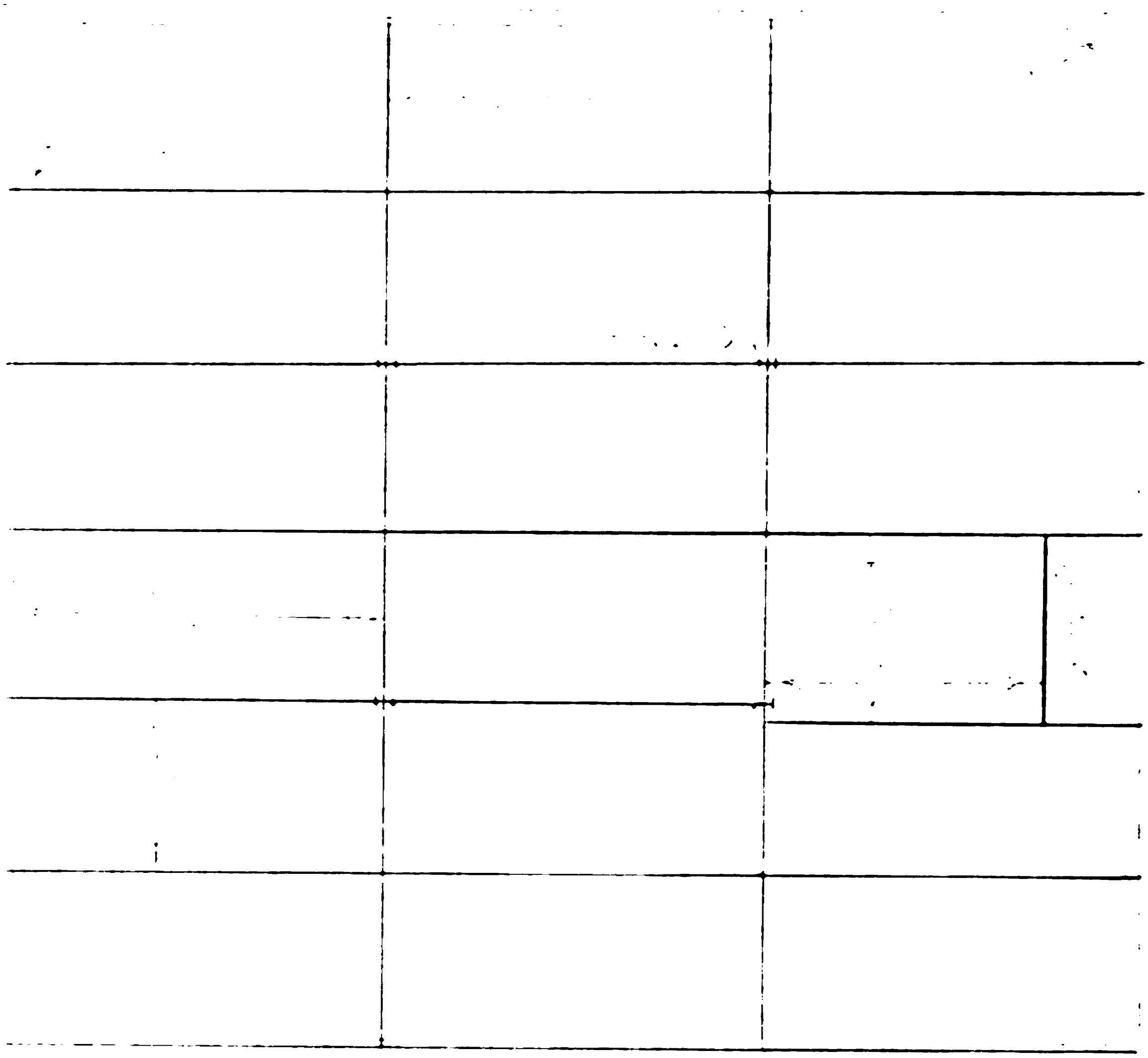
Live Load 100 #/□'

Dead Load 50 #/□' 4" reinforced slab




Building front

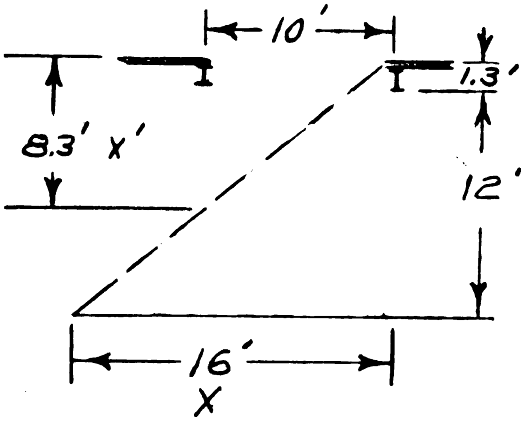
Floor beam and girder plan.

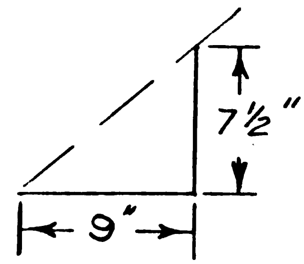


Stairs:

First to second floor:

7 1/2" riser
9" tread



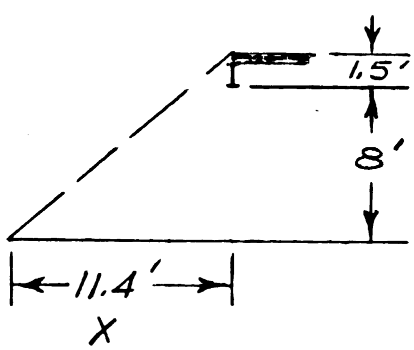

$$\frac{7.5}{9} = \frac{13.3}{X} \qquad X = 16.0'$$

$$\frac{10}{16} = \frac{X'}{13.3} \qquad X' = 8.3'$$

$$-\ 1.3$$

Clearance 7.0'

Basement to first floor:




$$\frac{9}{7.5} = \frac{X}{9.5} \qquad X = 11.4'$$

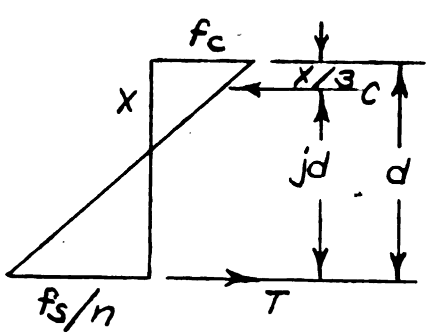
Slab design – second floor:

Use: $n = 12$ $\quad f'_c = 3000$ #/□"

$f_c = (.35)f'_c = 1050$ #/□"

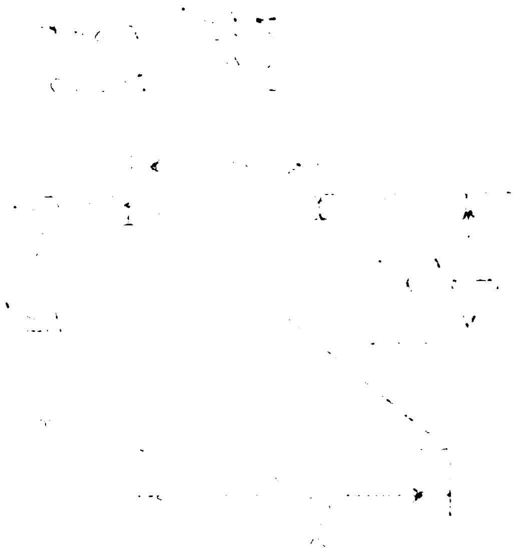
$f_s = 18000$ #/□"

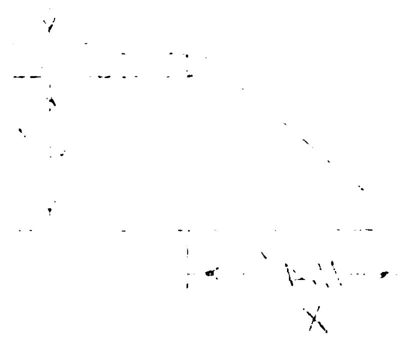
$f_s/n = 1500$ #/□"




$$\frac{d}{X} = \frac{1500 + 1050}{1050} \qquad X = .412d$$

$$1.0d - \frac{.412d}{3} = jd = .863d$$




X

$$M = \frac{wl^2}{12} = \frac{150(9)^2}{12} = 1010(12) = 12150 \text{ ''\#}$$

$$C = (.412d)(12)\left(\frac{1050}{2}\right) = 2590\,d$$

$$(C)(jd) = M$$

$$2590d(.863d) = 12150$$

$$d^2 = \frac{12150}{2235} = 5.45 \qquad d = 2.34''$$

Use $2.5''$

Total necessary thickness $2.5'' + 1.0'' = 3.5''$

$$C = T = 2590\,d = 2590(2.5) = 6480\ \#$$

Area of steel $= \frac{P}{S} = \frac{6480}{18000} = .36\ \square''$

Area of $\frac{1}{2}\phi = .20\ \square''$

Therefore use $2 - \frac{1}{2}\phi$ / ft. for ⊕ steel.

$\frac{1}{2}\phi$ @ $6''$

Check for bond : $\frac{1}{2}\phi$ Circumference $= 1.571$

$$\mu = \frac{V}{\Sigma o\, jd} = \frac{150(4.5)}{2(1.571)(.863)(2.5)} = 99\ \#/\square''$$

$$\mu < (.05)(f'c) \qquad (.05)(3000) = 150\ \#/\square''$$

Therefore bond is satisfactory.

Check for shear :

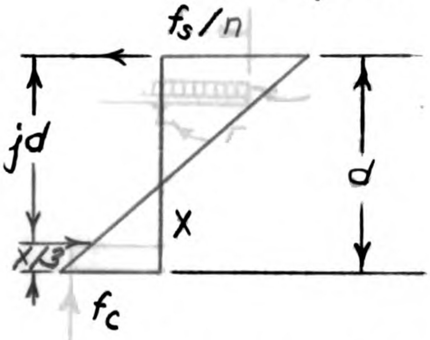
$$v = \frac{V}{bjd} = \frac{150(4.5)}{(12)(.863)(2.5)} = 26\ \#/\square''$$

$$v < .03f'c \qquad (.03)(3000) = 90\ \#/\square''$$

Therefore shear is satisfactory.

Since the steel is running in one direction temperature steel will be used : $A_s = .002\,A_c$
Using $3/8\phi$ bars : $.11 = .002(4)(\text{spacing})$ $\quad 3/8\phi$ @ $13.5''$

For negative steel:




Using the same values as for + steel.

$C = 2590d \qquad jd = .863d$

$14600''\# = M = \frac{wl^2}{10} = \frac{150(9)^2(12)}{10}$

$14600 = (2590d)(.863d)$

$d^2 = 6.52\ \square'' \qquad d = 2.56''$

$A_s = \frac{2590(2.56)}{18000} = .37\ \square''$

Use $1/2\ \phi$ @ 6" for negative steel

Length of negative steel:

From A.I.S.C.—length of negative moment is 12/28 $l$.

length = length neg. mom. + (30)(bar dia.)

$\frac{12}{28}(9)(12) + 30(\frac{1}{2}) = 46.3 + 15 = 61.3''$

Use 5½' length. This will allow for the 10' span at the stairs.

Check on the 10' span:

$M = \frac{wl^2}{12} = \frac{150(10)^2}{12} = 1250(12) = 15000''\#$

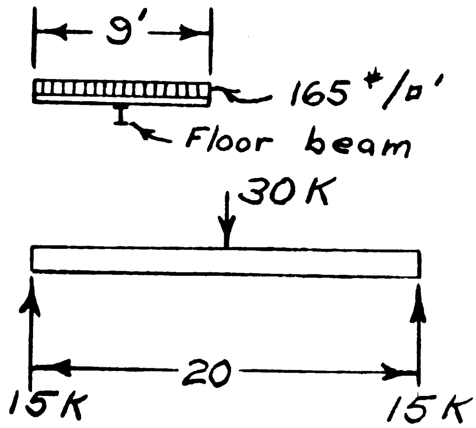
$(2590d)(.863d) = 15000 \qquad d^2 = 6.70 \qquad d = 2.59''$

$A_s = \frac{2590(2.59)}{18000} = 3.73\ \square''$

Therefore the 10' span will not cause a change in the slab design.

* Use d = 3" and t = 4" for actual design of the slab.

Floor beams and girders:




Floor beam:

| | |
|---|---|
| L.L. | 100#/□' |
| (concrete) D.L. | 50 |
| (plaster) D.L. | 5 |
| (beam steel) D.L. | 10 |
| | 165#/□' |

$9(165)(20) = 29700^{\#} = 30K$

The deflection of floor beams carrying plaster ceilings should be limited to not more than 1/360 of the span length.

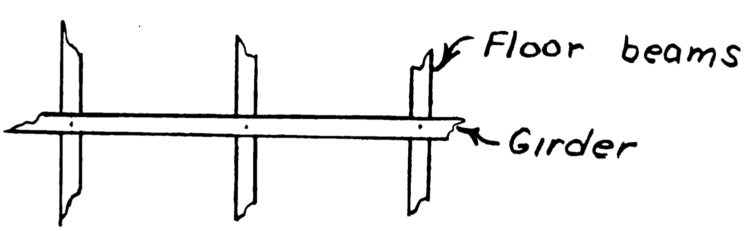
$(1/360)(20)(12) = .67''$

From A.I.S.C.:

12 WF 36 I deflection = .69"
20' Long load = 31 K

$\frac{30\ K}{31\ K} = \frac{X}{.69''}$   $X = .67''$   Therefore 12 WF 36 may be used.

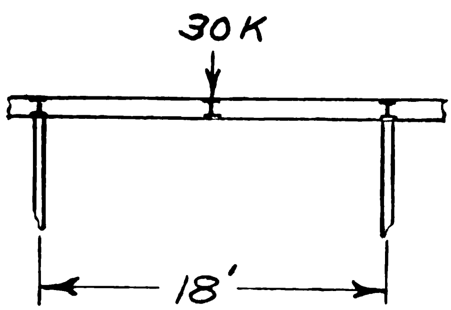
Girders:




From A.I.S.C.

12 WF 65

54' Long



To check the steel weight assumed:

Floor beams $(36)(20) = 720^{\#}$

Girder $(65)(9) = 585$

$1305^{\#}$

Surface area: $9 \times 20 = 180^{\square'}$

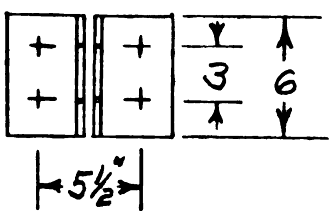
$$\frac{1305}{180} = 7.25^{\#}/_{\square'}$$ Which checks.

Beam connectors to be used throughout:

A.I.S.C. Method:

3/4" Rivets

Use 2 ∠s $4 \times 4 \times 3/8$




7/8" holes

Double Shear t = 5/16 9.38K

Double Shear t = 3/8 11.30K

Shear = 15.0K

Enclosed bearing on the web 30.0t

12 WF 36 t = .305"

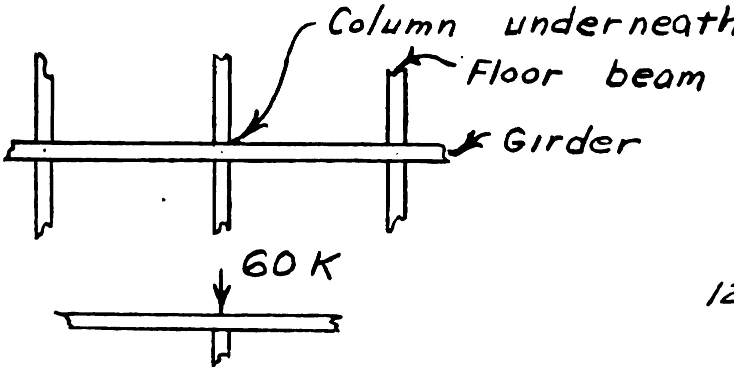
12 WF 65 t = .390"

O.S. Legs (4 Rivets)

$(4)(11.30) = 45.2\ K$

Web Legs $\frac{15}{(30)(.3125)} = 1.6$ Rivets — use 2

Columns:

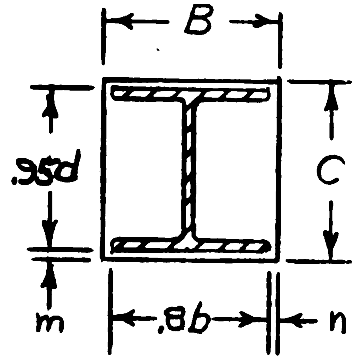



From A.I.S.C.

8 WF 31#

12½' Long

## Column Base Plates: A. I. S. C. Method




Bearing on concrete $= .375 f'_c$

$(.375)(3000) = 1125$ #/□"

$p = 1125$ #/□"

$L = 60K$

$b$ & $d = 8"$

$$A = \frac{L}{p} = \frac{60000}{1125} = 53.4 \text{ □"}$$

Assume $B$ & $C = 9"$ $\quad A = 81$ □"

$$p = \frac{60000}{81} = 740 \text{ #/□"}$$

$.80b = (.80)(8) = 6.4"$ $\qquad n = \frac{9 - 6.4}{2} = 1.3"$

$t^2 = .15pn^2 = .15(.75)(1.3)^2 = .191$ □" $\qquad t = .438"$

$.95d = (.95)(8) = 7.6"$ $\qquad m = \frac{9 - 7.6}{2} = .7"$

$t^2 = .15pm^2$ $\qquad t$ is governed by $.15pn^2$ since $n > m$

Use a 1" plate 9x9 because a plate under 1" is not used in actual practice for a base plate.

## Column footing: Sutherland & Reese Method

$f'_c = 3000$ #/□" $\qquad$ Use $v_c = .02 f'_c$

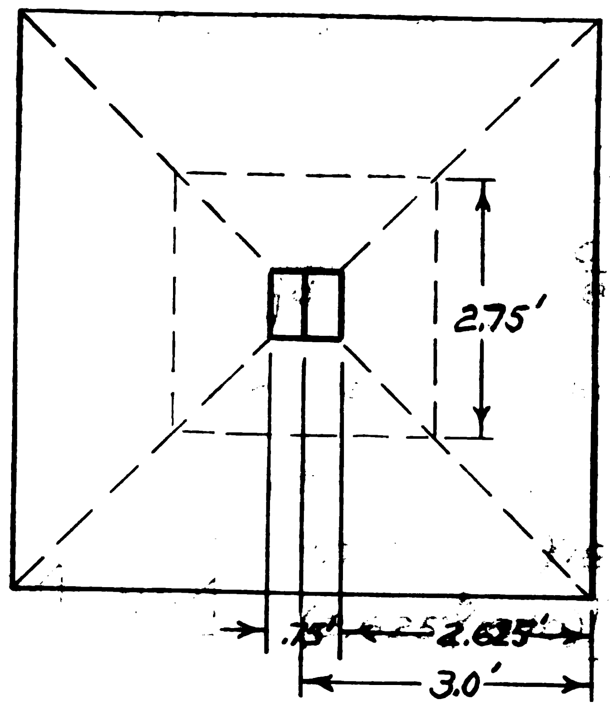
$f_c = 1050$ #/□"

Soil pressure $= 2000$ #/□'

$j = 7/8$

$f_s = 18000$ #/□"

| | |
|---|---|
| 60.0K | Load |
| .5 | (D.L. Column) |
| 6.0 | (Footing D.L.) |
| 66.5K | Total Load |




$$\frac{66500}{20000} = 33.25\ \square'$$

Therefore use a 6X6 footing.

$v_c = .02(3000) = 60\ \#/\square''$

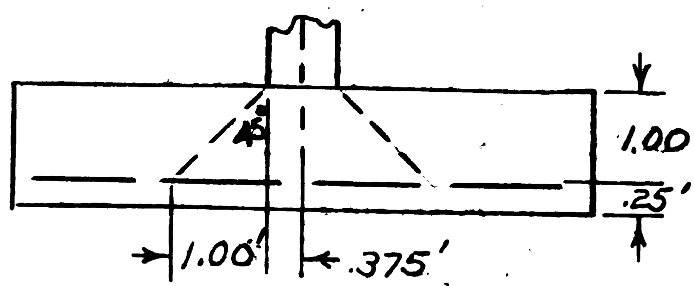
$j = 7/8 \qquad K = a/L$

$a = .75' \qquad L = 6.0'$

$K = \frac{.75}{6} = .125$

$C = \frac{w}{504\, v_c} = \frac{2000}{504(60)}$

$C = .066$




(Sketch out of scale.)

$$\frac{d}{L} = \frac{\sqrt{2C + 4C^2 + \frac{K^2}{4}} - \frac{K}{2}(1+4C)}{2 + 4C}$$

$$d = 6\left(\frac{\sqrt{2(.066) + 4(.066)^2 + \frac{(.125)^2}{4}} - \frac{.125}{2}[1 + 4(.066)]}{2 + 4(.066)}\right)$$

$$d = 2.65\sqrt{.0774} = 2.65(.279) = .74'$$

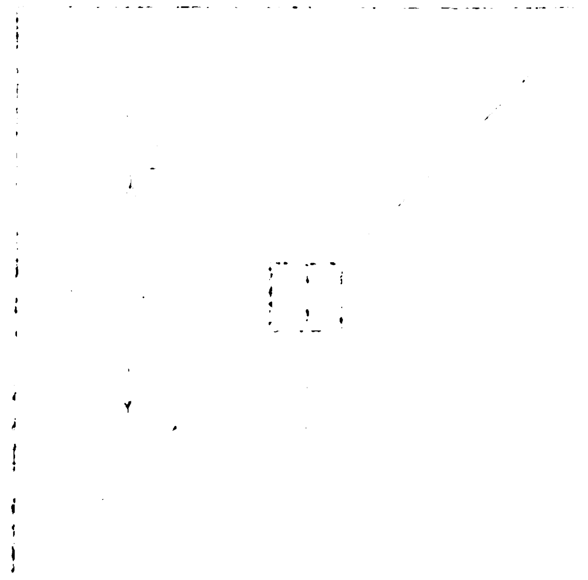
Actual weight $150(6)(6)(1.25) = 6750\ \#$

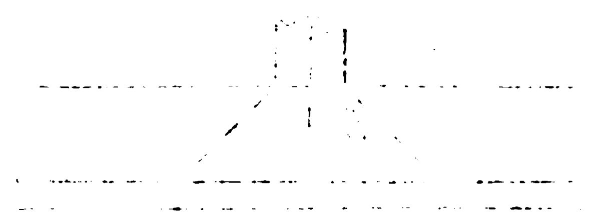
Other load $= 60500$

36 | 67250 #

net soil pres. = 1870 #/□'

* d used as 1' because .74' would be to low a value.





Check on shear:

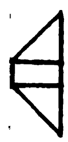
$$(6^2 - 2.75^2) = 36.0 - 7.55 = 28.45\ \square'$$

$$28.45(1870) = 53200\ \# = V$$

$$v_c = \frac{V}{bjd} = \frac{53200}{4(33.0)(.875)(12)} = 38.5\ \#/\square''$$

Shear is all right since $38.5\#/\square'' < 60.0\#/\square''$

Moment at the edge of cap:



$\square$ $$(.75)(2.625)(1870)\frac{2.625}{2} = 4830'\#$$

$2 \times \triangle$ $$(2.625)^2(1870)\left[\left(\frac{2}{3}\right)(2.625)\right] = 22600$$

$$27430'\#$$

$$M = 27430(12) = 329{,}000''\#$$

Effective width:

$$.75 + 2.00 + 1.625 = 4.375'$$

$$R = \frac{329000}{(4.375)(12)(12)^2} = 43.5$$

From the tables in Sutherland & Reese's text for $f'_c = 3000\ \#/\square''$ $R < 236$ — this checks.

$$\text{Area of steel} = \frac{329000}{(18000)(7/8)(12)} = 1.46\ \square'' \text{ in } 4.375'$$

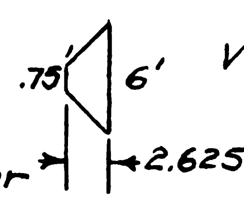
$$A_s = \frac{M}{f_s jd}$$

$$\frac{1.46}{A_s} = \frac{4.375}{6.0} \qquad A_s = 2.0\ \square''$$

Use 8 - 1/2 $\phi$ @ 9" 

A of 1/2 $\phi$ = .25 $\square''$

$$n = \frac{V}{o\,u\,jd}$$




$$V = \frac{.75+6}{2}(1870)2.63$$

$$V = 16600\#$$

$u = .062 f_c = .062(3000)$

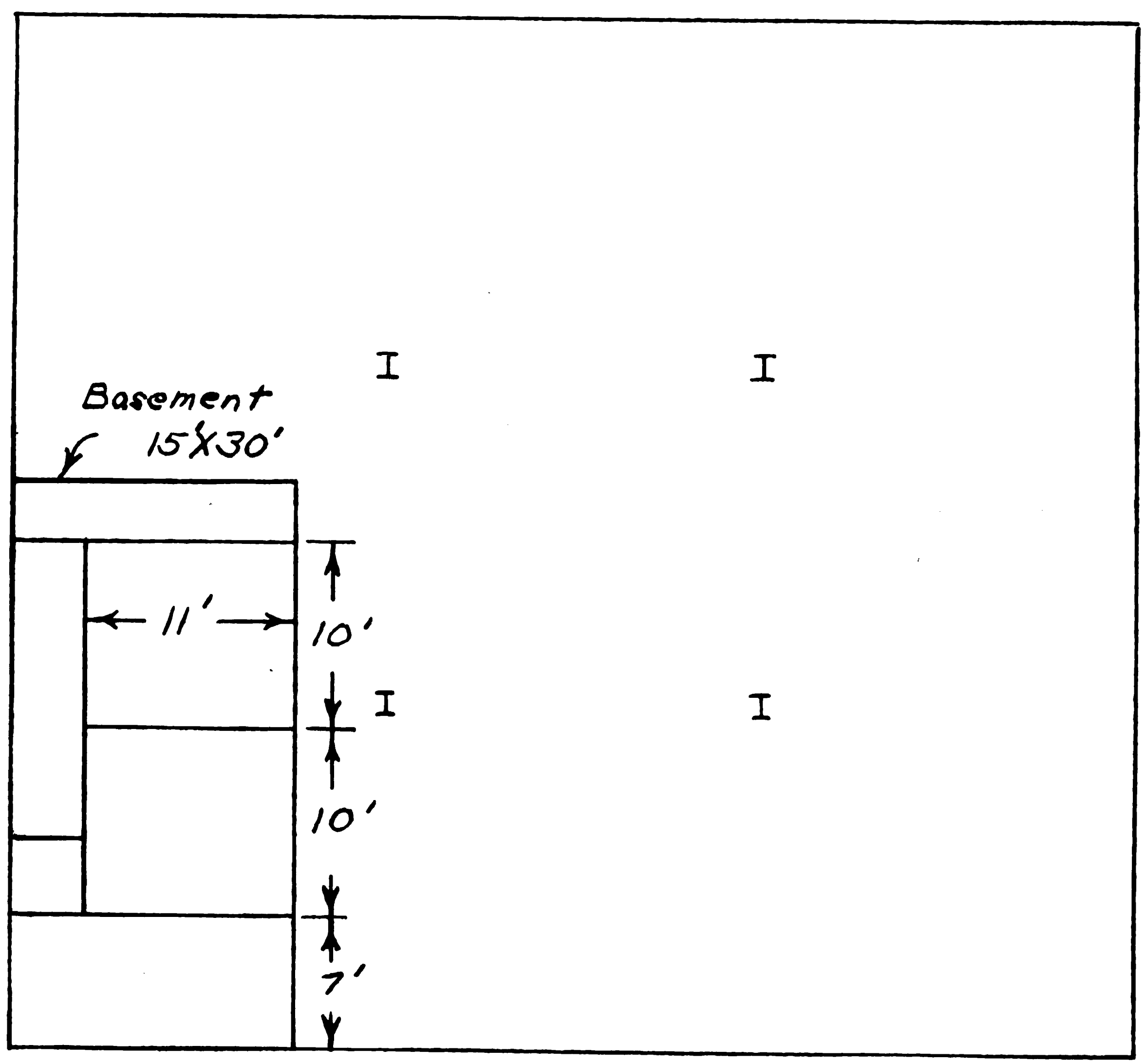
$u = 186\ \#/\square''$ $\quad$ o = perimeter

$$n = \frac{16600}{(2.0)(186)(7/8)(12)} = 4.25 \text{ rods}$$

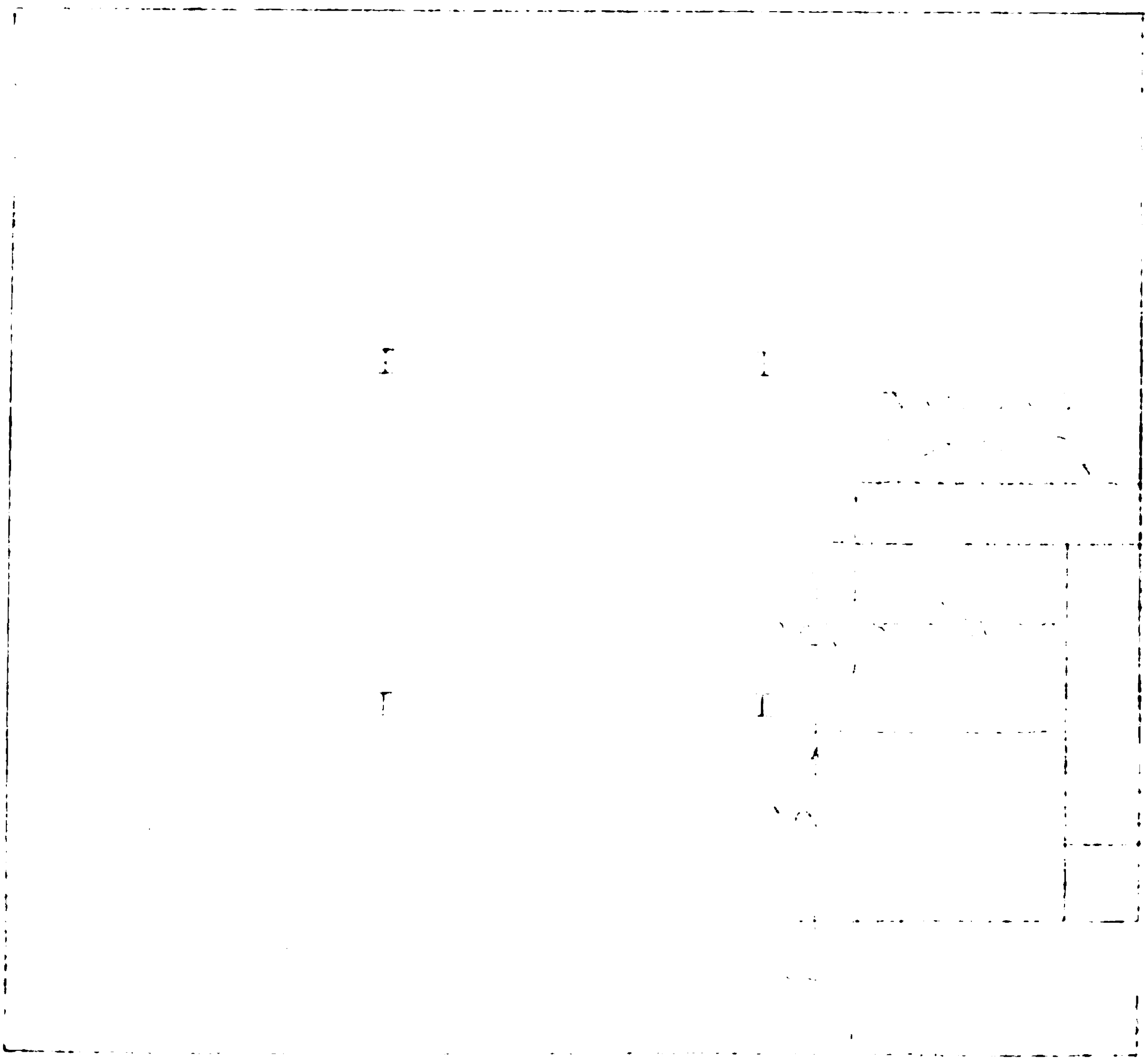
Since 8 rods were used the design is all right for bond.

The steel is to be laid in both directions and is to be 5.5' long.

First floor concrete slab:




First floor and basement plan



Assume a concentrated load of 2500 # on a square 6" on a side.

$f'_c = 3000$ #/□" $\qquad n = 12$

$f_c = 1050$ #/□" $\qquad v_c = .03 f'_c$

$f_s = 18000$ #/□" $\qquad j = .863$

Assume :

d = 5" $\qquad$ total thickness = 1+5 = 6"

Perimeter around the square = 36"

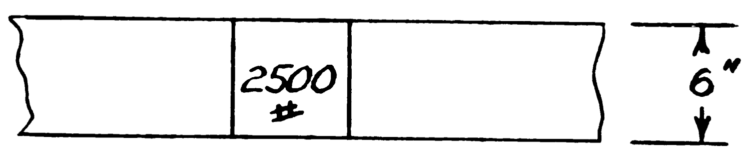
$$v_c = \frac{P}{A} \qquad \begin{array}{l} P = 2500\ \# \\ A = (\text{perimeter})(\text{thickness}) \end{array}$$

$$v_c = \frac{2500}{(36)(6)} = 11.6\ \#/\square''$$

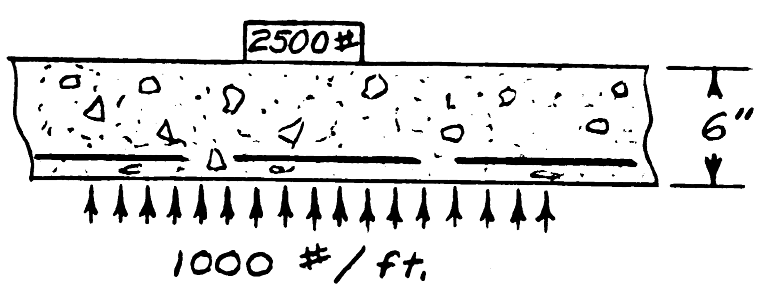
This value indicates a safe range for shear since the maximum $v_c$ = 90 #/□".

Check for moment:

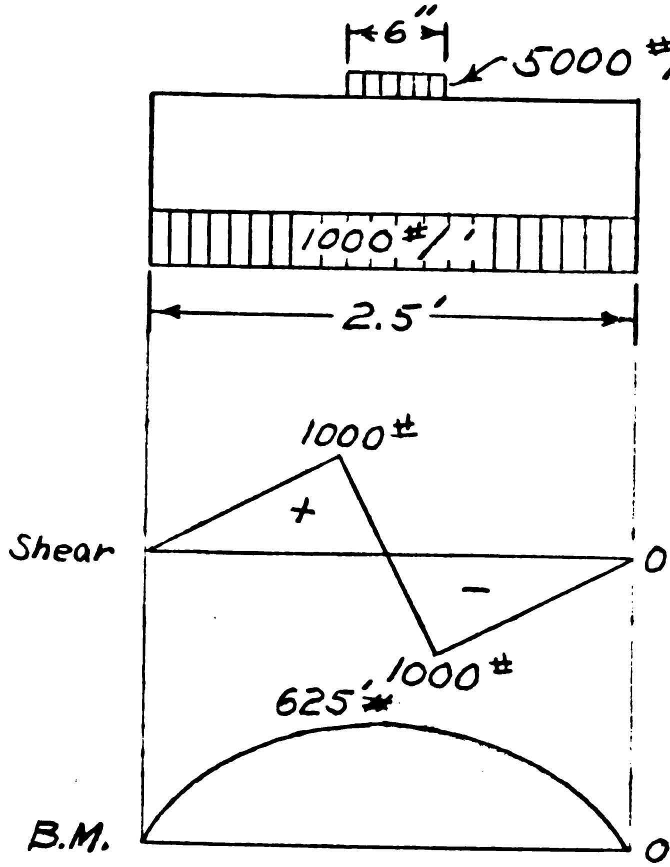
Assume a 6" slab section and bearing pressure of 2000 #/□' for soil.




$$\frac{2000}{6/12} = 1000\ \#/'$$



2500 # load = 5000 #/' for 6"




$$\frac{2500}{1000} = 2.5'$$

$$\frac{1.25\,(1000)}{2} = 625\ '\#$$

$$625\,(12) = 7500\ ''\#$$

$$M = Cjd$$

$$C = \frac{M}{jd}$$

$$C = \frac{7500}{(.863)(5)} = 1740\ \#$$

$$A_s = \frac{1740}{18000} = .097\ \square''$$

A of 3/8 $\phi$ = .11 □" Use 3/8 $\phi$ @ 6"

The above design will be poured over properly graded subsoil.

For the area over the basement use the following data and computations :

| | | |
|---|---|---|
| L.L. | 150 #/□' | |
| D.L. | 75 #/□' | |
| total | 225 #/□' | |

Use C = 2590 (d) from the second floor design.

$$M = \frac{wl^2}{12} = \frac{225(10)^2}{12} = \frac{22500}{12}(12) = 22500''\#$$

$$C\,jd = 22500$$

$$(2590\,d)(.863\,d) = 22500$$

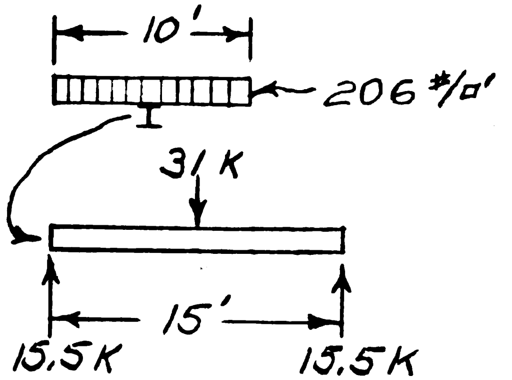
$$d^2 = \frac{22500}{2235} = 10.1$$

$d = 3.18''$ Use $3.5''$ $t = 4.5''$

$$A_s = \frac{2590\,(3.5)}{18000} = .503\,\square''$$

Use $1/2\phi$ bars $A = .25\,\square''$ $1/2\phi$ @ $6''$

WF beams to be used in the basement:




L. L. = 150 #/□'

$\left(\frac{4.5}{12}\right)150$ D.L. = 56 #/□'

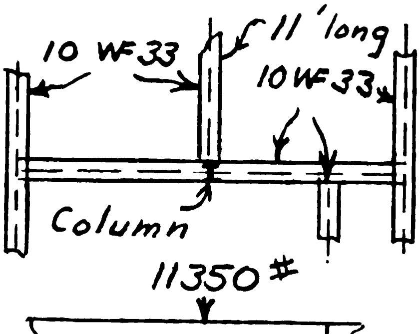
206 #/□'

$(10)(15)(206) = 30900\,\#$

Use 31 K

From A. I. S. C.

10 WF 33

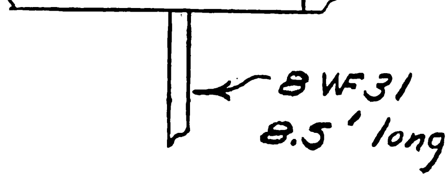



Column:

$(10)(11)(206) = 22700\,\#$

$\frac{22700}{2}$ = Reaction on column

R = 11350 #




Column 8 WF 31

Use same size base plates as before.

No negative steel will be use on the first floor slab — except over the basement.

Negative steel:

$C = 2590\,d \qquad jd = .863\,d$

$$M = \frac{wl^2}{10} = \frac{206(10)^2}{10} = 2060\,(12) = 24720\,''\#$$

$$C\,jd = M \qquad \frac{24720}{(2590)(.863)} = d^2 = 11.1 \qquad d = 3.34''$$

Use $d = 3.5''$

$$A_s = \frac{(2590)(3.5)}{18000} = .501\,\square''$$

Use $1/2\,\phi$ @ $6''$ $\qquad A = .25\,\square''$

Length of negative steel: $\qquad A = \frac{\pi d^2}{4}$

$$\frac{12}{28}(l'') + 30(\text{bar dia.}) = l \text{ of steel}$$

$$\frac{12}{28}(10)(12) + 30\left(\sqrt{\frac{4(.25)}{3.14}}\right) = 52'' + 17'' = 69''$$

$$l = 5.75'$$

Temperature steel:

$A_s = .002\,A_c \qquad$ Use $3/8\,\phi$ bars $\qquad A = .11\,\square''$

$.11 = .002\,(4.5)(\text{spacing}) \qquad s = 12^{+}\,''$

Use $3/8\,\phi$ @ $12''$

Both the temperature and negative steel were designed the same as that used in the second floor slab.

Footing for basement column: —

Assume a possible load of 30K to include everything except the footing D. L..

$f'_c = 3000\ \#/\square''$ soil $= 2000\ \#/\square'$

$j = 7/8$

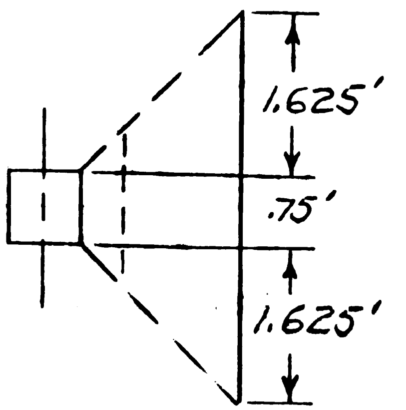
L.L. 30.0 K

D.L. 1.5 (Assumed footing wt.)

31.5 K

$\frac{31500}{2000} = 15.75\ \square'$  $4' \times 4' = 16\ \square'$

Assume a "d" value of 6" or .5'




$D.L. = (150)(16)(.75) = 1800\ \#/\square'$

$30000\ \#/\square'$

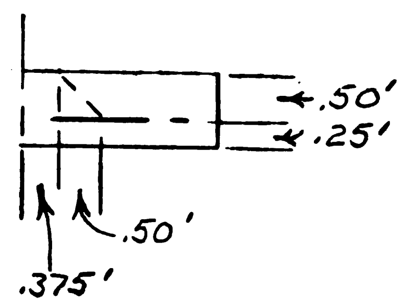
$16\ \overline{)\ 31800}$

$1990\ \#/\square'$

Check on shear :

$(4^2 - 1.75^2) = 16 - 3.06 = 12.94\ \square'$

$(12.94)(1990) = V = 25700\ \#$




$v_c = \frac{V}{bjd} = \frac{25700}{4(21)(.875)(6)}$

$v_c = 58.3\ \#/\square'' < 60\ \#/\square''$

Moment at the edge of cap :

$(1.625)(.75)(1990)\left(\frac{1.625}{2}\right) = 1975\ '\#$

2 → $(1.625)^2\left(\frac{2}{3}\right)(1.625)(1990) = 5700\ '\#$

$7675\ '\#$

$M = (7675)(12) = 92000\ ''\#$

Effective width :

$.75 + 1.00 + 1.125 = 2.875'$

$$R = \frac{92000}{(2.875)(12)(6)^2} = \frac{92000}{1240} = 74.2 < 236$$

$$\text{Area of steel} = \frac{92000}{(18000)(7/8)(6)} = .975 \,\square'' \text{ in } 2.875'$$

$$\frac{.975}{2.875} = \frac{A_s}{4} \qquad A_s = 1.36 \,\square'' \text{ in } 4.0'$$

Use 6 - ½⌀ @ 8"

Check on bond:

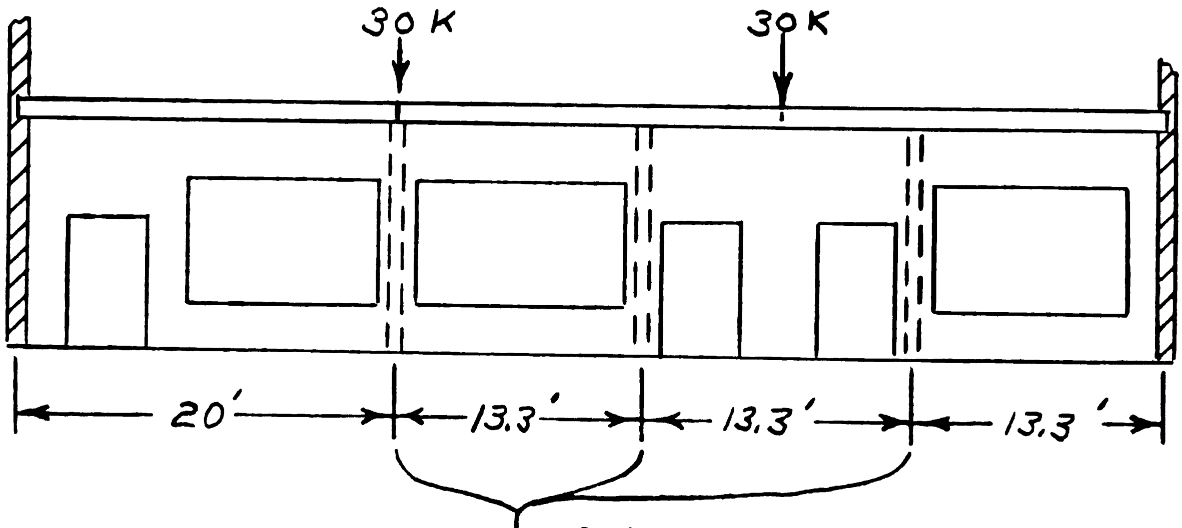
$$n = \frac{V}{o\,u\,jd} = \frac{(1990)\left(\frac{4.0+.75}{2}\right)(1.625)}{(2)(.875)(6)(186)} = \frac{7680}{1955}$$

$n = 3.93$ rods

Since 6 rods were used this checks satisfactory.

The method used in designing this footing was the same method used earlier in the design.

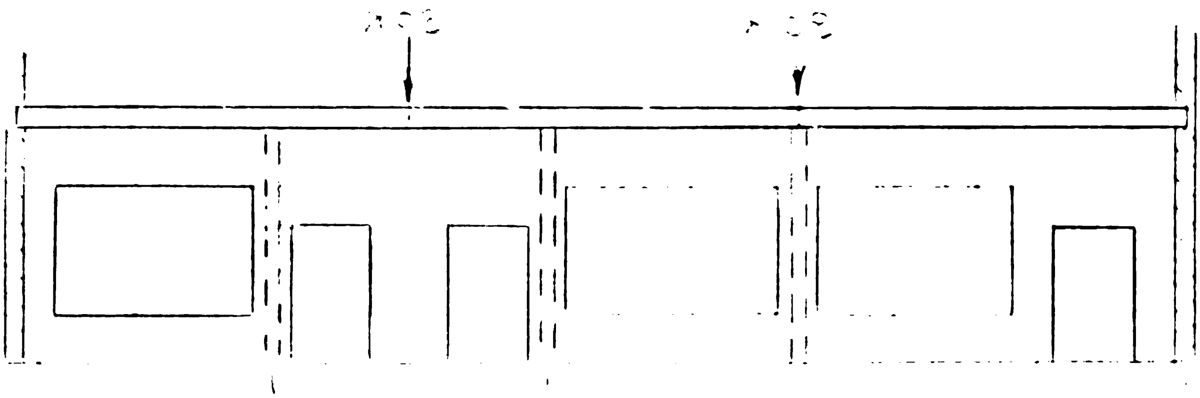
STORE-FRONT DESIGN :




Columns are set on a 9"X9" base plate — the base plate is set in the foundation then welded construction is used. The foundation is spread as shown on the print showing the area needed to build building.

From A.I.S.C. use 8 WF 24 for columns

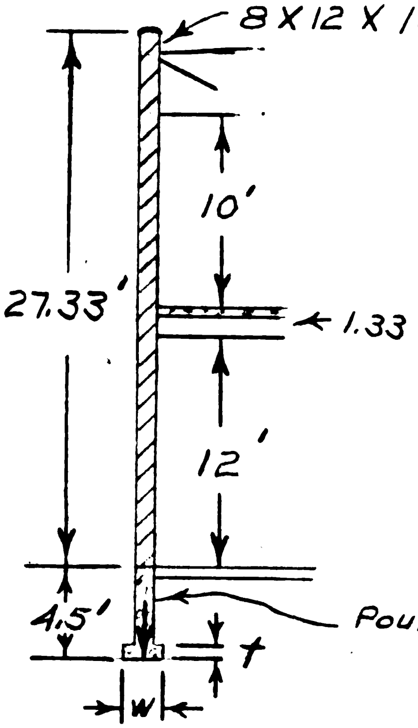
Base plate 1" X 9" X 9"



FOUNDATION DESIGN :

For the foundation sections not around the basement :

Assume a 4' frost line.




Concrete = 150 #/cu. ft.
8X12X16 = 65 #

Take 1' strip

$$\frac{1 \text{ cu. ft.}}{\frac{8(12)(16)}{1728}} = \frac{X}{65} \qquad X = 73 \text{ \#/cu.'}$$

Block = 73 #/cu. ft.
Use 75 #/cu. ft.

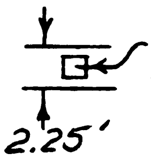
Assume 10' of the second floor to be included in wall weight. Second floor = 165 #/□'

Approximate $P = (75)(27.33) + 10(165) + 4.5(150)$

$P = 2050 + 1650 + 675 = 4375$ #

Assume bearing pressure = 2000 #/□' soil.

$$s = \frac{P}{A} \qquad (1)W = \frac{4375}{2000} = 2.187' \quad \text{use } 2.25'$$




P acting directly vertical on a 12" square

$v_c = 90$ #/□"

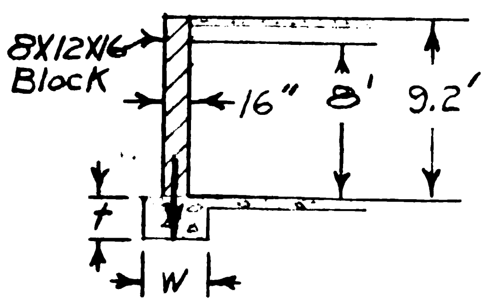
$$t(12)(4) = \frac{4375}{90} \qquad t = 1.01''$$

$t = 0.5'$ & $W = 2.25'$

For the section around the basement — outer wall:

Same loading above earth as before —

Assume $W = 3'$ & $t = 2'$




$$P = 2050 + 1650 + \left(\frac{16}{12}\right)(9.2)(75) + (2)(3)(150)$$

$$\begin{array}{r} 2050 \\ 1650 \\ 918 \\ 900 \\ \hline P = 5518\ \# \end{array}$$

Actual $W = \dfrac{5518}{2000} = 2.759'$

$t = \dfrac{5518}{(90)(48)} = 1.2''$

Use $W = 2.75'$ & $t = 1.0'$ for practical reasons.

Use $W = 1.0'$ and $t = 1.0'$ for the foundation under the 8X8X16 block wall which forms the inner walls.

Where pilasters are in the walls the foundation swings out around it with the same additional width added to it. (This is shown in the print showing the area needed to build this building.)

# LINTEL and BEARING PLATE DESIGNS :

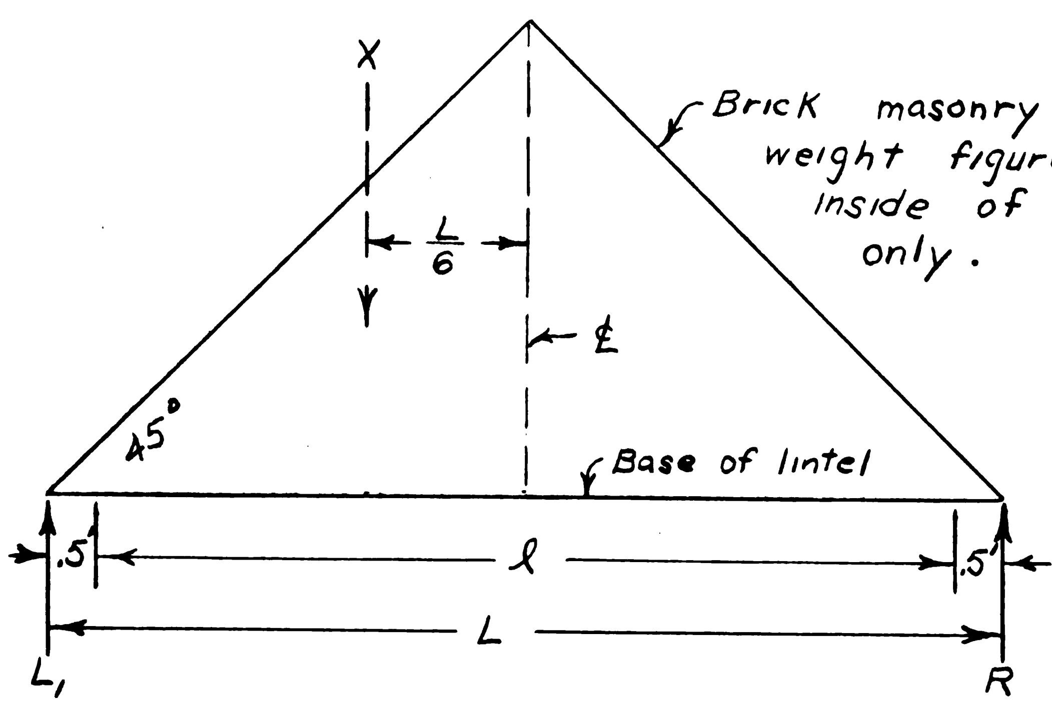
Lintels :

12" Brick masonry wall

Common brick = 120 #/ cu. ft.

S = 20000 #/□" for steel.

All lintel designs will refer to the following sketch :




ℓ = opening in wall
L = total lintel length
X = total weight of left Δ

For $\ell = 11'$:

$L = 12'$

$$L_1 = R = \frac{\frac{6(12)}{2}(120)}{2} = 2160\ \#$$

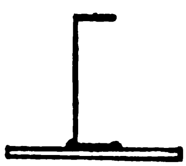
B.M.

$$2160(6) - \frac{6(6)}{2}(120)(2) = 12960 - 4320$$

$$B.M. = 8640\ '\#$$

$$S = \frac{M}{s} = \frac{(8640)(12)}{20000} = 5.2''^3$$

From A.I.S.C. use 7 [ 9.8 with a 1/4" X 9" plate welded to the [.



Drop bolts through the plate to fasten the window-sash.

For $\ell = 10'$ use the same structural shapes as those used when $\ell = 11'$. $L = 11'$ for $\ell = 10'$.

For $\ell = 3'$:

$L = 4'$

$$L_1 = R = \frac{\frac{4(2)}{2}(120)}{2} = 240\ \#$$

B.M.

$$2(240) - 240\left(\frac{2}{3}\right) = 320\ '\#$$

$$S = \frac{M}{s} = \frac{320(12)}{20000} = .192''^3$$

Use a $2\frac{1}{2} \times 2 \times \frac{1}{4}$ ∠ with a 1/4" X 9" plate welded to the ∠.

For $\ell = 4'$

$L = 5'$

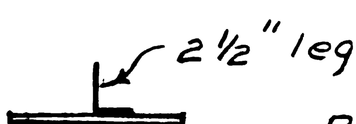
$$L_1 = R = \frac{\frac{(2.5)(5)}{2}(120)}{2} = 375\#$$

B.M.

$$(375)(2.5) - 375\left(\frac{5}{6}\right) = 625'\#$$

$$S = \frac{M}{S} = \frac{(625)(12)}{20000} = .375''^3$$

Use a $2\frac{1}{2} \times 2 \times \frac{1}{4}$ ∠ with a $\frac{1}{4}'' \times 9''$ plate welded to the ∠. ($2\frac{1}{2}''$ leg is in the vertical position.)




Position of ∠ the same for $\ell = 3'$ and $\ell = 4'$ lintel.

For $\ell = 12'$

$L = 13'$

$$L_1 = R = \frac{\frac{13(6.5)}{2}(120)}{2} = 2540\#$$

B.M.

$$(2540)(6.5) - 2540\left(\frac{13}{6}\right) = 11000'\#$$

$$S = \frac{M}{S} = \frac{11000(12)}{20000} = 6.05''^3$$

Use 7 [ 12.25 with $\frac{1}{4}'' \times 9''$ plate.

Bearing Plates:

For lintels:

No actual bearing plate is needed, but the 1/4" plate will act as such, and the length of the lintels overlap the masonry a 1/2' at each end of the opening.

Taking the maximum lintel reaction of 2540# and a bearing pressure for a masonry wall of 250#/□" we get:

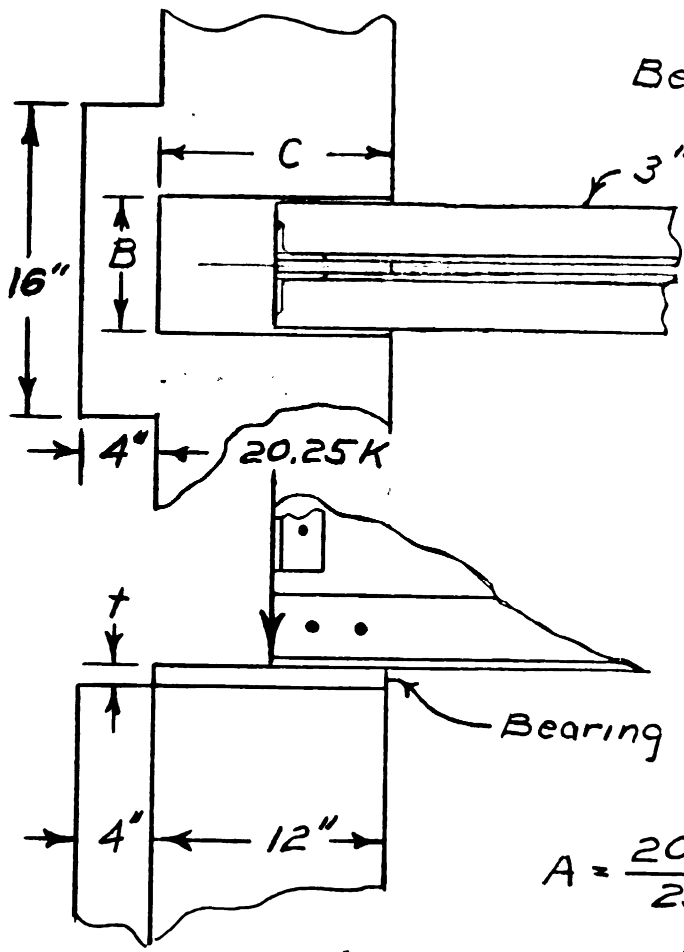
$$s = \frac{P}{A} \qquad A = \frac{2540}{250} = 10.2 \, \square''$$

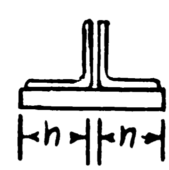
Since the walls are 12" wide — a 1" strip would prove a satisfactory bearing area. Therefore no bearing plate is needed.

Bearing plates for the roof truss:

A.I.S.C. Method

Bearing pressure = 250#/□"




$$A = \frac{20250}{250} = 81 \, \square''$$

Assume C = 12"  12 B = 81

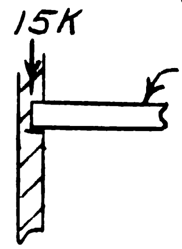
Let B = 7"

$$\text{net } p = \frac{20250}{12(7)} = 241 \, \#/\square'' \qquad n = \frac{B}{2} - 1 = \frac{7}{2} - 1 = 2.5''$$

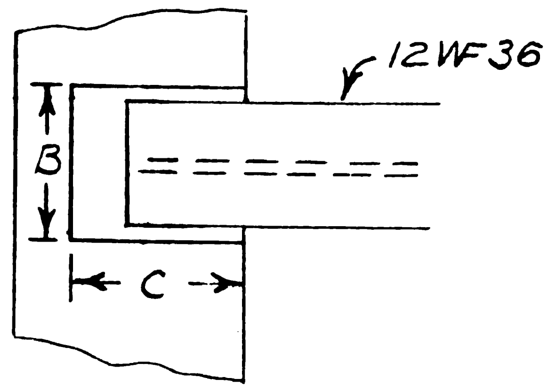
$$t^2 = .15 \, p n^2 = (.15)(.241 \, K/\square'')(2.5)^2 = .2260 \qquad t = .476''$$

Use 3/4" plate because this is the thinnest used in actual practice. Welded construction is used along with anchor bolts.

Bearing plates for the floor beams:




Using:




$$A = \frac{15000}{250} = 60 \square''$$

Assume $B = 8''$ $C = 9''$

$$\text{net } p = \frac{15000}{(8)(9)} = 208 \,\#/\square'' \qquad n = \frac{8}{2} - 1 = 3$$

$$t^2 = (.15)(.208)(3)^2 = .281 \qquad t = .53$$

Use 3/4" plate 8"x 9" — Use welded construction and anchor bolts.

[redacted]

Pocket has: Drawing no. 1
&
Drawing no. 2