Thesis,

Department of Architecture.

1877

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Massachusetts Institute of Technology,
Department of Architecture.

Thesis, explanatory of a design for a Rail Road Station in a small town, showing the treatment of a trussed roof, the strength of the floors, stability of an arch, etc.

Signature Redacted

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The Station is supposed to be situated in a small city, and is to be built along side of the tracks.

The building is to be of brick with stone trimmings. The floor beams are of wood.

The roof is of wood covered with blue slate. The foundation walls are of stone resting upon piles driven in a loose sand.

The building consists of a main two story part, 32' x 44', with a Tower 87' high, at the south corner, and of an L, 32' x 53', one story in height.

A wooden awning supported upon iron brackets projects eight feet from the wall, and is continuous on three sides of the L, and across one end of the main building.

The main entrance is through the tower, in which is a Vestibule, which is separated from the Hall proper by an archway; out of this Hall lead the stairs to the second floor, and those to the Basement, the latter being directly under the former.

From the Hall access is had to the Ladies' waiting room on the right, and to the Gentlemen's waiting room on the left.

The Ticket-office is situated at the end of the Hall under the landing of the stairs, and between the two Waiting rooms. A door at the right of the vestibule opens into a Lunch room, which communicates with the Ladies' waiting room by means of a window.

There are also two Water Closets, a Store room for Lanterns Oil etc. and a Baggage Room.

On the second floor is a General Freight Office, an Agent's room, and a room for the storage of papers or any such use.
Also a Battery room, a Telegraph Office, and a Waiting room for same, from which access may be had to the clock tower.
In the Basement is placed the Steam heating apparatus, the coal bins, and a large Tank to hold water for the boiler supply, the water from three sides of the main roof being brought to the Tank by means of copper conductors.
The water from the roof of the L drains off onto the wooden awning, and is carried by copper conductors from the gutter upon this, back to the wall, and down the side of wall to the drain pipes.

Thickness of Walls.

The walls of the first story of the main building, and those of the L, are 16" in thickness, the walls above second story floor are 12" thick. the walls of the tower are 30" thick at the base, and are battered to a thickness of 20" at level of second story floor, from which point they are of that dimension.

These figures are taken from "The Building laws of the city of Boston"

The thickness of the same walls, as calculated by the formulae deduced by Gwilt from measurements of a large number of buildings in the old world, differ but little from the results as given above. By his formulae, the 16" walls should be but 15" while the 12" ones would be 14" thick.

Stability of the Arch.

Over the main entrance door way in the foot of the tower the wall is carried by an arch, having a span of six feet (6'), and a rise of one foot (1').

The proper depth for the key stone as given by Rankine's formula is, (depth of keystone of any single arch = $\sqrt{.12 \text{ radius of arch}}$) .84 of a foot. The tables in Trautwine's Hand book give it as .98 of a foot.

In consideration however of the fact that this arch is loaded very heavily upon the haunches, it was deemed best to give it a depth greater than either of these values, and it has been taken as 1.15 feet.

There are three things which need to be taken into account in investigating the stability of an arch, these are, 1st, Its tendency to rotate inwards about the inner edge of a voussoir.

2nd, Its tendency to rotate outwards about the outer edge of a voussoir.

3rd Its tendency to slip upon some one of the radial joints.

The two latter ways of failing are however of very rare occurrence.

The problem, simply stated, is, to ascertain the greatest thrust, applied at the crown of the arch, necessary to prevent rotation inwards, and then (first making sure that it is not so large as to cause rotation outwards, and is also sufficient to prevent slipping on any joint) providing an abutment, which shall have weight enough to withstand this pressure at the crown.

To accomplish this, a diagram

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Thrust at O necessary to prevent rotation inwards = 7190 lbs
" " " " " cause " outwards = 7856 lbs
" " " " " prevent slipping = 6375 lbs

Joint of rupture in 1st case 11-5
" " " " 2nd " 9-3
" " " " 3rd " 12-6.

Thickness of abutment necessary for equilibrium 2.24'
Abutment is 4' thick
Moment of thrust is 84842 lbs
Resistance to it is 195725 lbs

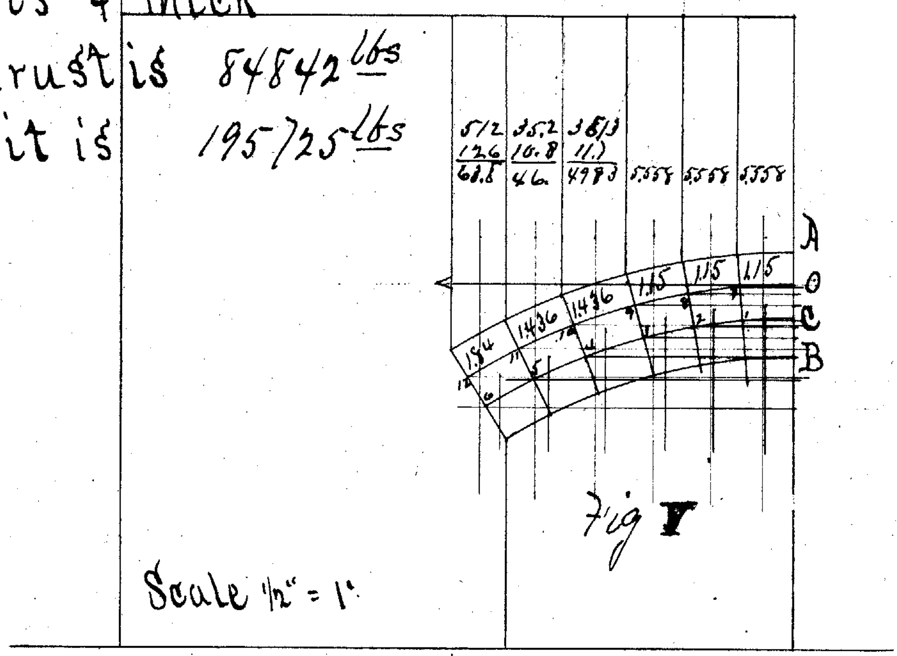

Fig V

of which Fig V is a sketch, on a scale of 1" to a foot was made showing the division of the arch into voussoirs of any convenient size, and vertical joints were drawn from the outer edges of the radial joints, dividing the superincumbent wall into vertical sections.

As, to insure stability, the line of resistance must pass through the middle third of the arch, two arcs, concentric with the extrados were drawn, passing through 0 and 6, and dividing AB into three equal parts.

To find the thrust at 0 necessary to prevent the part to the right of the joint 1-7 from rotating inwards about 1, by taking the moment of that part of the arch to the right of the joint 1-7, plus the moment of that portion of the superimposed load which rests upon the same portion of the arch, and divide the sum by the vertical distance from the point 1 to 8.

In the same manner find the thrust at 8 necessary to prevent rotation for each joint. The largest of these results will be the thrust requisite for stability.

The thrust necessary to cause rotation outwards, is found in a similar manner, taking moments about the outer edges 7, 8, 9, etc. instead of about 1, 2, 3, etc., as before.

The amount of thrust required to prevent slipping at any joint may be found by substituting in the formula, $T = W \tan(\phi - e)$, where T = thrust. W = weight of arch and superimposed load to the right of the joint under consideration. ϕ = the angle made by the joint with a horizontal line, and e = the angle of repose of the material dealt with.

By performing the operations indicated, it was found that the greatest tendency to rotation inwards was at the joint 5-11, and that the thrust at O necessary to prevent it, was 7190 lbs.

The greatest tendency to rotation outwards was found to be at the joint 3-9, where a thrust of 7856 lbs would be needed at O to produce this effect, while the tendency to fail by slipping was found to be at the joint 6-12, and that a thrust at O of 6375 lbs would prevent its occurrence.

From this we know that if the abutment is made heavy enough to withstand an amount of pressure that will prevent rotation inwards the arch cannot fail by rotating outwards or by slipping, as the thrust will be too small to produce the former, and too large to allow the latter.

Therefore, taking this as the thrust for which an abutment must be provided, to insure stability, its moment about the outer edge of the abutment must be less than that of the half arch with its load, plus that of the abutment itself, about the same point. And as the moment of the thrust at O is 84842 lbs, the abutment must be more than 2'-3", and in the design it has a thickness of four feet (4'), which brings the line of resistance within the middle third of the pier, where it must come in order to have stable equilibrium.

Piling.

The foundation walls, which are carried ten feet (10') below the grade, are supported upon piles driven in sand.

According to Rankine piles will carry from 200 to 300 pounds per sq inch of the upper surface, and the load put upon them varies from 200 to 100 pounds for the same area, which leaves a factor of safety of from 2 to 10.

By a french rule, the weight to be borne by a pile should equal the weight of the Hammer, into the distance in feet that it falls the last blow, divided by the number of inches which it drives the pile; the same authority says, that in sand, piles should be driven until a Hammer weighing 1800 pounds, falling 20 feet, will not move the pile more than one inch and a half. Where an eight inch pile is driven. it would, by this rule. carry 3600 pounds, or 702 lbs per sq inch of section. In the present calculation the load has been assumed as 300 pounds per sq inch of section, or a load of 15378 lbs per pile.

Upon this basis it was found that 168 piles. 8" in diameter, would be required to support the weight of the building itself, plus the live load of the floors. The weight of the brickwork being taken as 125 lbs per cubic foot, and the foundation walls as 160 lbs per cubic foot.

In calculations where the weight of timber has been taken into account, it has been regarded as 45 lbs per cubic foot.

Strength of Floors.

The floor of the L is in two bays, of 57' x 14'-6" each, and is composed of wooden sticks, 2.5" x 10" in section, spaced 16" apart on centres.

Taking the constant for pine as 450, (which was found to be very nearly the true value, in some experiments made in the Physical Labratory during the first half of the present year) and using 6 as a factor of safety, the safe load for the beam was found to be

$$\frac{2.5 \times 10 \times 10 \times 450}{14.5 \times 6} = 1289^{lbs}$$ applied at the

centre of the stick, from this deduct the weight of the beam and floor, which is a uniformly distributed load of 72 lbs + 108 lbs, equal to a load of 90 lbs at the centre of the stick. and we have 1289-90 = 1199 lbs.

as the actual safe load of beams. or, as the floor load will be a uniformly distributed one, 2398 lbs is the safe load. and as each beam carries an area of 14'-6"x 1'-4" = 19.3' the safe load per sq ft of floor is 2398÷19.3' = 124 pounds.

Both the first and second floors of the main building are divided into two bays, one 9'x39', and one 18'x39'.

The two 18'x39' bays are covered by wooden sticks, 4"x10" in section, and spaced 16" apart on centres, by a similar calculation, the safe load per sq. ft for these two bays is found to be 124 lbs.

The two 9'x39' bays are covered by wooden sticks, 2"x7", spaced 16" apart on centres, and their safe load is found to be 128 pounds per sq ft.

The covering of the floor beams, in all the floors, is inch and a quarter (1¼") pine stuff, which, supported upon beams 16" on centres, will sustain safely a load of 2029 pounds per sq.ft.

The formula used for the above calculations is $\frac{bh^2f}{ln} = W$, where

b = breadth of beam in inches.

h = depth " " "

l = length " " " feet.

f = the weight required to break a piece of the material used, one foot in length, and one inch square.

n = the factor of safety, and

W = the safe load to be applied in the centre of the beam.

This formula can only be used when beams of rectangular section are under consideration.

Roof Trusses.

The roofs are composed of trusses similar to the one shown in Fig VI, which is the one over the L. the dimensions of which are calculated by the aid of Greene's Graphical Analysis of Roof Trusses. The diagrams used in the calculation being shown in Figs I. II. and III.

Fig I is a skeleton of the truss, from which the distribution of the load, and the direction of the pieces composing it are obtained.

Fig II shows the strain in these pieces, due to the weight of the roof itself, the weight of a ceiling hung to the horizontal tie rod, and to the accumulation of snow upon the roof. The weight of the roof is 16 lbs per sq ft, that of the ceiling 10 lbs per sq ft, while the load of snow was taken as

it is given by Greene, or 12 lbs per sq ft.
The strains in the various members of the truss, due to these causes, are given in the table on the same page.
Fig III shows the effect upon the truss due to the force of the wind, which was regarded as acting normal to the slope of the roof, with a force of 26.5 lbs per sq ft, as is directed by Greene for a slope of 30°.
On the same sheet is given the total strain in each member, due to the combined effect of snow, wind, ceiling and wt of roof, from which table the dimensions shown in Fig VI were determined.
The size of the horizontal tie beam A.(18-K) due to the truss, should be 4"×14", but as it supports the ceiling beneath, it must have a transverse strength, in

addition, capable of supporting the ceiling. To carry this ceiling will require a beam 4"x 6.5", consequently this tie must have an area of 4"x 7.9" and has been put in 4"x 8".

The strain in the tie J. I. and of the truss, is 2934 lbs but as it must, in addition to this, transmit the weight of one half the ceiling up to the joint D. E. it must be large enough to carry 2934 lbs + 1620 lbs or 4554 lbs.

The Formula used in calculating the size of the struts is;

$$P s = \frac{A f}{1 + a \frac{l^2}{h^2}}$$

, taken from Greene, where

P = load in pounds.
s = factor of safety. (taken as 6)
A = Area of strut in inches,
f = a constant given as 7200 for timber.
a = " " " " " 1/250 " "

l = length of strut in inches, and
h = <u>least</u> dimension of strut in inches.

The ultimate tensile strength of iron was taken as 6000 lbs per sq inch, and that of timber as a little under 1000 lbs per sq inch, the factor of safety used being six.

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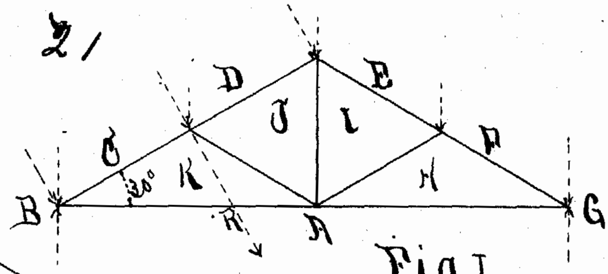


Fig I

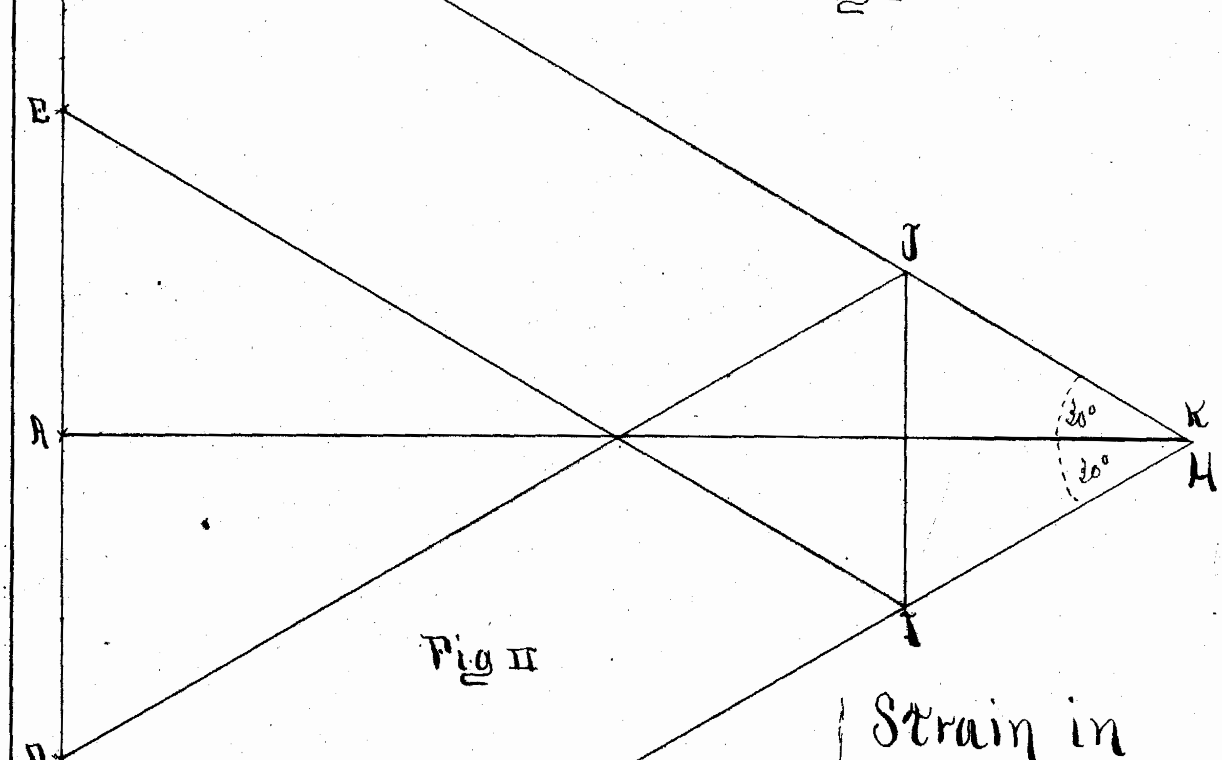


Fig II

Scale: 1 inch = 1000 pounds.

	Roof	Ceiling	Snow	Total
BC = FG =	625 +	810	+ 242	= 1677
CD = EF =	1250 +	1620	+ 484	= 1734
DE =	1250 +	1620	+ 484	= 3354

Strain in

CK = FH =	6760.	C
DJ = EI =	5100.	C
JK = IH =	1750.	C
AK = KH =	5850.	T
JI =	1734	T

Total Strain on

JI = 1734 + 1620 = 3354

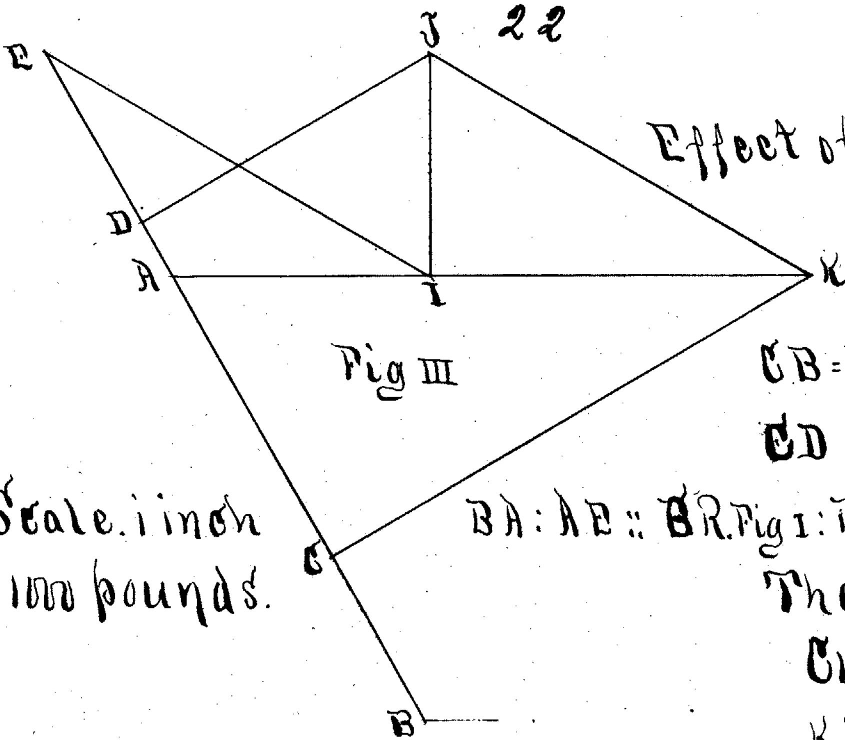


Fig III

Effect of wind.

CB = ED = 1015

CD = 2030.

Scale. 1 inch = 1000 pounds.

BA : AE :: BR. Fig I : BR. Fig I

Therefore.

CK = 2950. C.

KJ = 2320. C.

DJ = 1750. C.

JI = 1200. T.

AK = 3375. T.

EI = 2400. C.

Total strain on CK = FH = 6760 + 2950 = 9710. C.

" " " DJ = EI = 5100 + 2400 = 7500 C.

" " " JK = IH = 1750 + 2320 = 4070 C.

" " " JI = 3354 + 1200 = 4554 T.

" " " AK = KH = 5860 + 3375 9225 T.

Sketch showing dimensions of the roof truss over L.

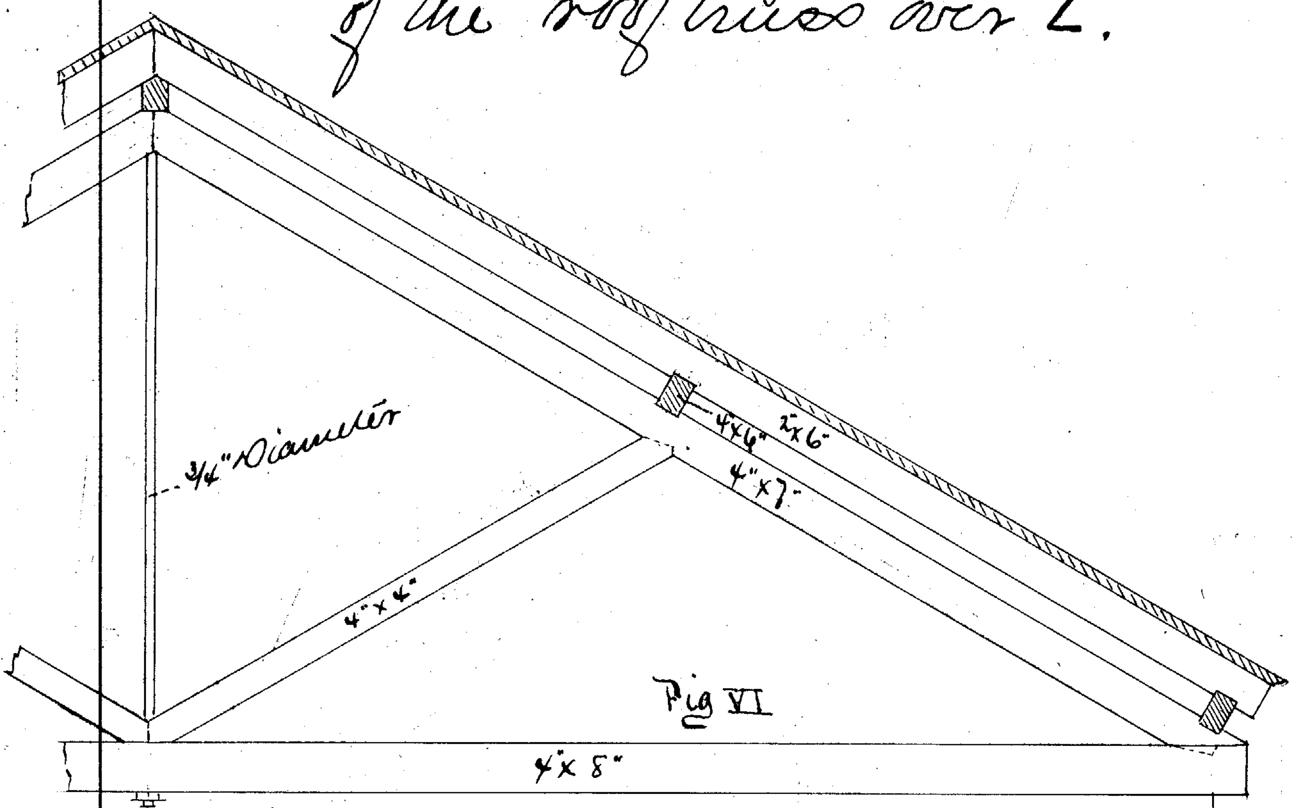


Fig VI

Scale 3/8" = 1'

A Trussed Partition

The partition between the Baggage room, and the Gentlemen's W.C. and the Supply room, is supported by a truss, a skeleton diagram of which is shown in Fig VII. From the joint I-E runs the partition between the supply room and the Gentlemen's W.C. which also is trussed, in such a manner as to throw one half its weight to the joint I-E. This, and the fact that a door was required between the tie G-F, and the joint B.A. led to the truss being of an unsymmetrical form, as shown.

The tables accompanying Fig VII give the loads at the joints, and the strains in the several members of the truss, as found by the aid of Fig VIII.

From these tables have been determined the proper dimensions for the members of the truss, which are as follows.

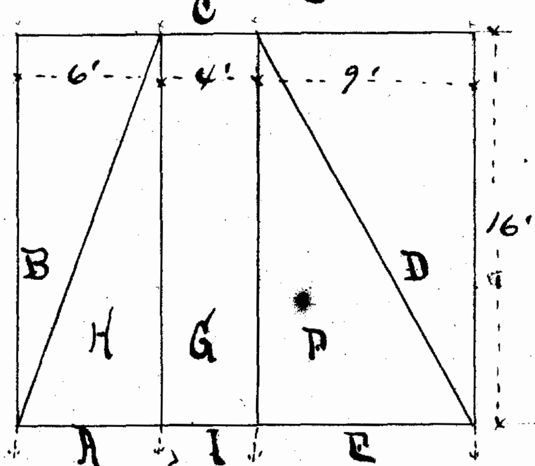
BH is wood.	2" × 2"	in compression.	
FD is wood.	2" × 3.4"	"	" " "
CG is wood.	2" × 3.4"	"	" " "
AIE / HGF "	"	4" × 4"	" tension.
HG "	iron	.42" in diameter,	in tension
GF "	"	.62" "	" " "

The same formulae have been used, to obtain the above values, that were taken when the orig truss was under consideration.

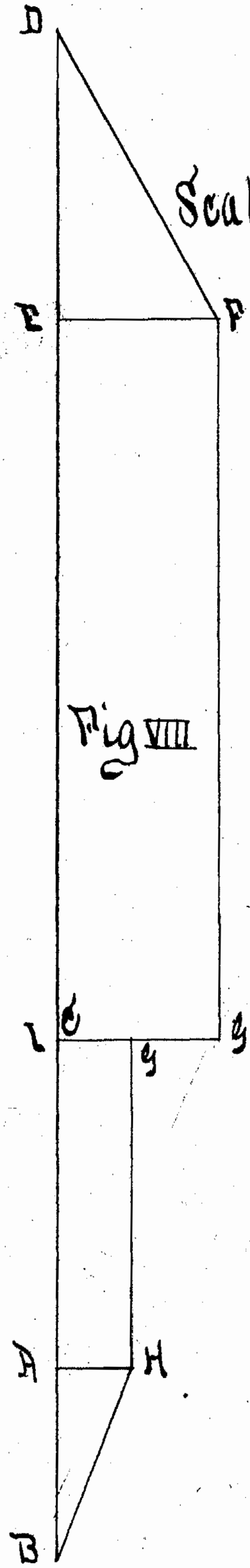
As the two struts BH and FD are supported by the building at points between their ends, the distance between these intermediate points has been taken as the value of l in the formulas.

The other unsupported partitions in the building are strengthened by a similar system of trussing.

Fig VII



Scale 1/8" = 1'



Scale 1" = 2000 lbs

Total weight of partition = 12960

Load at BA =		2046
" " AI =		3410
" " IE =	4433 + 3140 =	7573
" " ED =		3069

Strain on BH =		2200 C
" " AH =		800 T
" " DF =		3500 C
" " EF =		1700 T
" " CG = GI =	1700 + 800 =	2500
" " FG = EI =		7573
" " HG = AI =		3410