Thesis,
Department of Architecture.
1877
Pierce P. Furber.
Thesis, explanatory of a design for a Rail Road Station in a square town, showing the treatment of a covered roof, the strength of the floors, stability of an arch, etc.

Signature Redacted

Class of 1877.
Massachusetts Institute of Technology.
Department of Architecture.

Thesis, explanation 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.

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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 corrugated iron with 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 north corner, and of an L, 32' x 53', one story in height.

A modern 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.

Of this Hall, leading 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, but between the two waiting rooms. A door at the right of the vestibule opens into a ramp which communicates with the Ladies waiting room by means of a window.

There are also two Water Closets, a Store room for Sanitary Oil, etc., and a Baggage Room.
On the second floor is a General Freight Office, a Register Room, and a room for the storage of papers or any such use. Also at Battery room, a Telegraph Office, and a Waiting room for travels, 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 conduits.

The water from the roof of the L drains off into the storm drains running, and is carried by copper conduits 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 2d, are 16" in thickness, the walls above second.
Egym floor are 12" thick. The walls of the lower are 16" thick at the base, and are thinned to a thickness of 10" at level of second story floor, from which point they age of that dimension.

These figures are taken from "The Building Laws of the City of Boston."
The thickness of the above walls, as calculated by the formulas derived by
Swill upon measurements of a large number of buildings in the old world, dif-
fer but little from the recules as given above. By his formulas, the 16" walls
should be 14 1/2", while the 12" ones
would be 14" thick.
Stability of the Arch.

Over the plain entrance door way in the
port of the lower the course 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 Newlin's formula is (depth of
Keystone) x (any single arch) = (1/2 radius of arch)

In consideration however of the fact that
this arch is loaded only heavily upon
the haunches, it was "accurately" decided to
give it a depth greater than either of
these values. As it has been taken
as 1.25 feet.

There are three things which need to
be taken into account in investigating
the stability of an arch. These are,
6

1st tendency to rotate inwards about the inner edge of a masonry.
2nd tendency to rotate outwards about the outer edge of a masonry.
3rd tendency to slip up on 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. But 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 armature, which shall have weight enough to withstand this pressure at the crown.

To accomplish this, a diagram
Thrust at Q necessary to prevent rotation inwards: 7190 lbs
... cause outswards: 1852 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 22.4
Abutment is 4' thick
Moment of thrust is 54842 lbs
Resistance to it is 195725 lbs

Scale 1" = 1'
of which Fig. 1 is a sketch, on a scale
of 1 = 2½ feet, made showing the division
of the arch into arc series by convenient
equispacing. The vertical joints were drawn
from the outer edges of the radials
joint, dividing the superimposed
wall into vertical sections.

As to secure stability, the lines of
resistance, being parabolic through the middle
third of the arch, two arches, concentric
with the extrados were drawn, passing
through 0 at G, with dividing the into
three equal parts.

To find the thickness at 0 necessary to
prevent the part to the right of the joint
1-7 from rotating upwards about 1, take
the overhang of that part of the arch
to the right of the joint 1-7, plus the mo-
center of that portion of the superior part of the
which rests upon the upper portion of
the arch, and divide the same by the
vertical distance from the point 1 to 0.
In the same manner find the thrust
at 0 necessary to prevent rotation for each
joint. The largest of these results will
be the thrust requisite fir stability.
The thrust necessary to cause rota-
tion downwards, is found in a similar
manner, taking moments about the outer
edges. \( P = \frac{5}{9} \) etc. instead of about \( 1 \div 2 \)
\( \text{etc. as before.} \)
The amount of thrust required to pre-
vent slipping at any joint may be
found by substituting in the formula,
\[ T = W \tan (\phi - \epsilon) \]
where \( T = \text{thrust} \),
\( W = \text{weight of arch} \), \( \phi = \text{angle imposed} \)
load to the right of the joint under con-
cideration, \( \epsilon = \text{the angle made by the} \)
paper with a horizontal line, and \( e \) is the angle of repose of the material dealt with.

By performing the operations indicated, it was found that the greatest tendency to rotating inwards was at the joint 5-11, and that the thickest at 6 necessary to prevent it, was 7190 lbs.

The greatest tendency to rotating outwards was found to be at the joint 3-9, where a thrust of 7552 lbs could be used at 8 10 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 9 of 6376 lbs would prevent its occurrence.

From this we knew that if the armament is made heavy enough to 15 Girls, and an amount of pressure that
will prevent rotation inward, as the arch cannot fail by rotating outwards or by slipping, as the thrust will be less equal to produce the former, and less large to allow the latter.

Therefore, taking this as the thrust, for which an abutment must be provided, the moment due to the weight of the pier must be less than that of the half arch with its load, plus that of the abutment itself, above the same piers. And as the moment of the thrust at P is 54\(\sqrt{2}\)\(\text{ft-lbf}\), the abutments must be more than 2-5\(\text{"}\) and in the design it has a thickness of 4\(\text{"}\), which brings the face of resistance within the middle third of the pier, where it must come in order to have stable equilibrium.
Filing.

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

According to Hawkins, piles will carry from 2000 to 2500 pounds per sq inch of upper surface, and the loads per pile thereon varies from 200 to 1500 pounds. For the same area, which leaves a factor of safety of from 8 to 10.

By a French rule, the weight to be struck by a pile should equal the weight of the hammer, plus the distance in feet that it rises the last time, 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 500 pounds, falling 4 feet, will not move the pile more than one
inch and a half. These are eight inch piles of driven, it was by this once, carry 864 pounds of 702 lbs. per inch of section. In the present calculation the load has been assumed as in pounds per 25 inch of section; or a load of 10.378 lbs. per pile.

Upon this basis it was found that 168 piles, 8" in diameter, should be required to support the weight of the building itself, plus the loads of the floors. The weight of the brickwork being taken as 125 lbs. per cubic foot, and the foundation walls as 165 lbs. per cubic foot.

In calculations where the weight of timber has been taken into account, it has been regarded as 75 lbs. per cubic foot.
Strength of Floors.

The floor of the 2 is in two bays, of 57" x 14'-6" each, and is composed of modern joists, 2.5" x 10" in section, spaced 16" apart on centres.

Taking the constant for pine as 1,500 (which was found to be very nearly the true value, in some experiments made in the Physical Laboratory during the first half of the present year) and using 3 as a factor of safety, the safe load for the beam was found to be

\[
\frac{2.5 \times 10 \times 10 \times 150}{14.5 \times 6} = 1289 \text{ lb. applied at the centre of the stick, from this deduct the}
\]

\[
\text{weight of the beams and floor, which is an indefinitely distributed load of 77 lbs. per}
\]

\[
\text{square foot, or a load of 7.0 lbs. at the centre of the stick, and we have 1289-70=1219 lbs.}
\]
as the actual safe load of beams, or, as the floor load will be, a deflection by distribution may be in the safe load. 
and as each beam carries an area of 
14' 6" x 1' 4" = 19.3', the safe load per

Both the first and second floors of the
main building are divided into two bays, 
one 9' x 39' and one 15' x 39'.
The two 15' x 39' bays are covered by
modern trusses, 4' x 10' in section, and
spaced 16" apart on centres, by a circum-
linear calculation, the safe load per sq.
ft for these two bays is found to be 124 lbs.
The two 9' x 39' bays are covered by moder-

The actual safe load of beams, or, as the floor load will be, a deflection by distribution may be in the safe load. and as each beam carries an area of 

Both the first and second floors of the main building are divided into two bays, one 9' x 39' and one 15' x 39'. The two 15' x 39' bays are covered by modern trusses, 4' x 10' in section, and spaced 16" apart on centres, by a circumlinear calculation, the safe load per sq. ft for these two bays is found to be 124 lbs. The two 9' x 39' bays are covered by modern trusses, 2" x 7", spaced 16" apart on centres, and their safe load is found to be 128 pounds per sq. ft.
The engineering of the floor beams, in see the floors, is such that a quarter (1/4") pine stuff, which supported upon beams 16" on centers, may sustain safely a load of 2200 pounds per sq. ft.

The formula used for the above calculations is $W = \frac{b \times h^2 \times f}{l}$, where
- $b$ = breadth of beam in inches,
- $h$ = depth " " " " feet,
- $l$ = length " " " " feet,
- $f$ = the weight required to break a piece of the material used, one foot in length, and one inch square.
- $W$ = the fact of safety. and
- $W$ = the safe load to be applied in the center 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 requisite arms of which are calculated by the aid of Green'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 triangles in these pieces, and to the weight of the gong itself, the weight of a ceiling being 15 lbs per sq ft, and to the accumulation of snow upon the roof. The weight of the snow is 16 lbs per sq ft, while the load of snow was taken as
It is given by Greeny, or 12 kip per eft.

The strains in the various members of the truss, due to these causes, are given in the table on the same page.

Fig. 115 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 kip per eft, as is directed by Greeny on a slope of 10°.

On the same sheet is given the total strain in each member, due to the combined effect of snow, wind, ceiling, and not of roof, from which table the axial-dips shown in Fig. 115 were determined.

The size of the horizontal tie beam A.16-18 due to the trusses, should be 4'x14', but as it supports the ceiling beam, it must have a truss over it straight, i.e.
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", 4" has been put in x x 5".

The chain in the tie is 7.1. in the truss, is 29.5". But as it must, in additional to this transmit the weight of one half the ceiling up to the joint 10.6%. It must be large enough to carry 29.5/16 4/2 = 1.46 lbs.

The formula used in calculating the rise of the truss is:

\[ R = \frac{P^2}{1 + a \frac{P^2}{f}} \]

where

P  = load in pounds,
\( a \)  = factor of safety (taken as 6)
\( f \)  = area of chain, in inches,
\( f \)  = a constant given as 72 or for ladder.

\( a = \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldOTS
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\[ a = \text{length of steel in inches, and} \]
\[ h = \text{least dimension of steel in inches.} \]

The ultimate tensile strength of iron was taken as 30,000 lb per sq inch, and that of timber as a little under 1,000 lb per sq inch, the factor of safety used being six.
Fig 1

Strain in

\( CN = 760 \) T
\( DJ = 5100 \) T
\( IK = 1760 \) T
\( AK = KH = 5750 \) T
\( JL = 1734 \) T

Fig 2

Scale: 1 inch = 1000 pounds.

\( BC = PG = 620 + 810 = 1432 \) T
\( CD = FF = 1250 + 1620 + 454 = 3324 \) T
\( DE = 1250 + 1620 + 454 = 3324 \) T

Total Strain on
\( JL = 1734 + 1620 = 3354 \) T
Effect of wind

Fig III

Scale: inch = 1000 pounds.

CB: ED = 1015
ED = 2030.
BA: AE :: AR: Fig I: BR: Fig I
Therefore:

CK = 2950. C.
KJ = 2320. C.
DT = 1750. C.
TL = 1200. T.
AR = 3375. T.
EI = 2400. C.

Total strain on CK = TH = 6760 + 2950 = 9710. C.
" " DS = ET = 5140 + 2400 = 7500. C.
" " JR = TH = 1750 + 2320 = 4070. C.
" " TL = 3375 + 1200 = 4565. T.
" " AR = KH = 5860 + 3375 = 9225. T.
Sketch showing dimensions of the roof truss over L.
The partition between the baggage lines and the ballastway is in the upper line of the diagram, which is common at Fig. III.

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The beams, which are as follows.

El is wood, 2" x 2" in compression.
C. O. is wood, 2" x 3.4  
B. O. is wood, 2" x 3.4  
B. O. is wood, 2" x 3.4  
A. O. is wood, 4" x 3.4  
S. O. iron, 4.2" in diameter, in tension.
S. F.  
S. F.

The same formulas have been used to obtain the above values, that were taken where the only beam was under consideration.

As the two series D F and F X O are supported by the rubbing at pins between their ends, the distance between these intermediate points has been taken as the value of L in the formulas.

The other unsupported portions in the building are strengthened by a similar system of beams.
Fig VIII

Scale 1" = 2000 lbs

Total weight of partition = 12960

Load at BA = 2046
  AL = 3410
  LE = 4433 + 3/40 = 7573
  ED = 3069

Strain on BII =
  AN = 2200 C
  DP = 800 T
  EP = 3570 C
  CG = CL = 1700 + 500 = 2500
  FG = EL = 7573
  HG = AL = 3410