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“An Experimental Determination of the Stresses in a Roof-Truss.”

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IN the design of roofs from theoretical considerations, it is usual to assume that the stresses actually existing in the members are in accordance with those determined on the assumption that the joints are hinged and frictionless, and that the distances between them remain constant. With a view to check the actual results obtained from these assumptions, an experimental roof-truss was constructed in the engineering laboratories of the University of Manchester, and tested to destruction.

Considerations other than those of strength influence to a great extent the scantlings adopted for small structures. In the case of a roof, it is not until a span of about 60 feet is reached that any considerable proportion of the bars can be designed with a view to economy of material alone. The determining factors for these small spans include appearance, durability, and facility for handling and erection. Hence, in order to obtain useful results, it was desirable to select a truss of not less than 60 feet span.

Questions of cost and space rendered the construction and loading of a truss of these dimensions impracticable, and a suitable scale for the experimental construction had to be selected. A small model of special material and workmanship cannot be expected to yield results applicable to the conditions of actual practice; for it is difficult exactly to imitate working conditions, and to obtain steel identical in quality and finish with that used in structural engineering work. Half scale therefore, or a span of 30 feet, was chosen as representing the largest structure that could be conveniently tested.

The data from which the roof was designed were as follows :—

Span, 60 feet; spacing of trusses, 15 feet; pitch, 30°; and wind load, 26 lbs. per square foot normal, corresponding by the usual con-

vention with a force of 40 lbs. on a surface perpendicular to the direction of the wind. The dead load was taken as 17·25 lbs. per square foot of roof surface, allowing for slates at 10 lbs., boarding 4 lbs., and purlins 3·25 lbs. per square foot. The weight of the full-sized truss was assumed for purposes of design to be 3,000 lbs., although this, of course, was an overestimate.

The steel used was carefully tested for extensions below the elastic limit and for breaking-load, etc., the following mean results being obtained:—

Stress at yield point	21·4 tons per square inch.
Stress at maximum load	27·4 " " "
Ratio of yield to maximum load	0·782
Elongation at maximum load, on 8 inches	21·5 per cent.
Elongation at rupture, on 2 inches	39·0 "
" " 8 " "	25·8 "
Contraction of area	51·8 "
Modulus of elasticity	28,000,000 lbs. per square inch.

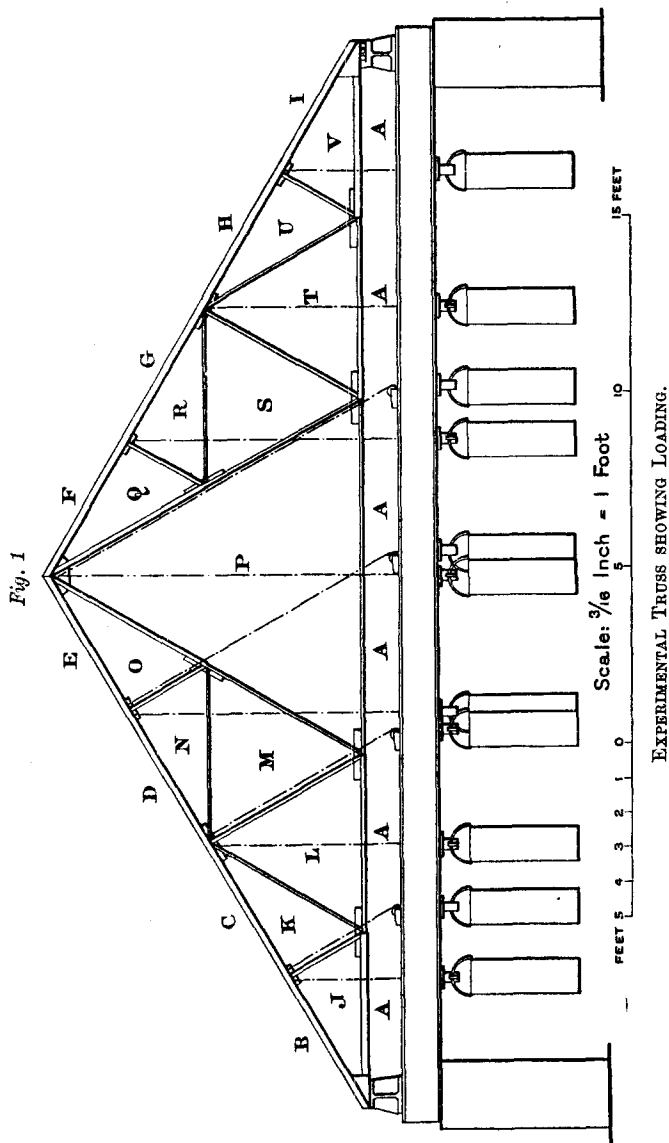
The general design of the truss followed the lines usually adopted in practice for a 30° roof of the foregoing dimensions. It was of the French type, the main backs being continuous and braced at three intermediate points, as shown in the outline diagram, *Fig. 1*. The sizes of the various members are given in the following Table:—

Member.	Description.	Size for Roof of 60-Foot Span.	Size in Model.
Rafters (BJ to IV)	Two angles	Inches. 4 × 4 × $\frac{1}{2}$	Inches. 2 × 2 × $\frac{1}{4}$
LM, ST	One angle	3 × 3 × $\frac{1}{2}$	1½ × 1½ × $\frac{1}{4}$
JK, NO, QR, UV	"	2 × 2 × $\frac{1}{4}$	1 × 1 × $\frac{3}{8}$
JA, VA	One flat bar	3½ × $\frac{1}{2}$	1½ × $\frac{1}{4}$
LA, TA	"	3½ × $\frac{1}{2}$	1½ × $\frac{1}{4}$
PA, OP, PQ	"	2½ × $\frac{1}{2}$	1½ × $\frac{1}{4}$
MP, SP	"	1½ × $\frac{1}{2}$	$\frac{7}{8}$ × $\frac{1}{4}$
KL, MN, RS, TU	"	1½ × $\frac{3}{8}$	$\frac{5}{8}$ × $\frac{3}{16}$

Since in the model the sectional area of each bar was one-quarter that of the corresponding member of the full-sized roof, the safe loads for the model were assumed as one-quarter of those calculated from the data taken.

This method obviated the difficulty which arises from the fact that the relative values of dead and wind loads differ in trusses of different spans. If *L* represent the linear dimensions, the total wind load varies as *L*², whilst the dead load due to covering varies

as L^n , where n has a value lying between 2 and 3. The weights of



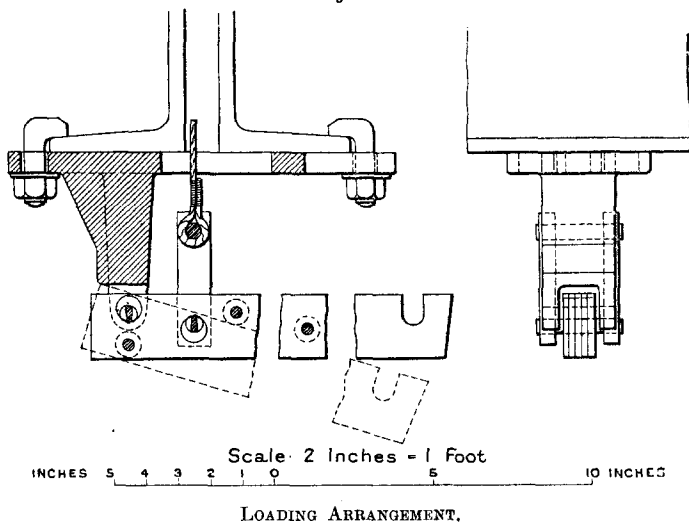
the structures themselves (assumed exactly similar) vary directly as L^3 .

It will be seen from the Table of scantlings that the principal compression and tension members correspond with sizes such as would be found in an actual truss of 60 feet span. Thus rafters B J and I V would consist of two angles, each 4 inches by 4 inches by $\frac{1}{2}$ inch—a practical size for a roof of this type.

For the purpose in view, however, it was essential to design the roof so as to obtain as far as practicable equal stress in all tension members, and therefore some of the bars may appear small.

In a roof of the type under test $\frac{3}{4}$ -inch rivets would probably be used, and it would appear therefore that in order to imitate practical conditions as closely as possible, $\frac{3}{8}$ -inch rivets should be used for

Figs. 2.



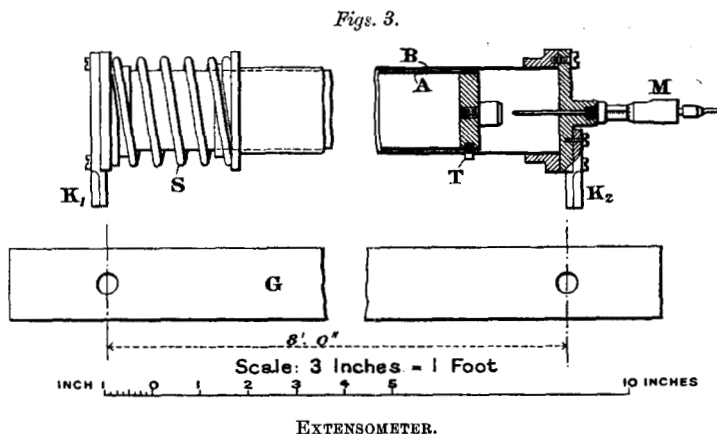
the model. Since, however, the object of the experiment was to determine the stresses in the main members, and not to determine the strength of the joints, bolts of $\frac{3}{8}$ -inch diameter were used instead of rivets, and their total cross sectional area was made sufficiently large to preclude any chance of failure from shear.

It will therefore be seen that the structure used was, with the exception of the riveting, an exact half-scale model of the 60-foot truss, both with regard to construction and loading.

The experimental apparatus (*Fig. 1*) consisted of two 12 by $3\frac{1}{2}$ -inch channels placed back to back, 1 inch apart, and supported at their ends by two brick piers, the channels being about 5 feet above

the ground. Two cast-iron pedestals were bolted to the upper side of the channels, the truss being fixed to the left-hand pedestal and carried on rollers on the right-hand pedestal. Lateral movement of the structure was prevented by means of A-shaped frames striding the roof and placed one at each joint of the main rafter, these frames being arranged so as not to interfere with vertical motion. The frames therefore gave to the model the support which in practice it would have received from adjacent parts of the roof.

In the system of loading adopted, a shackle was attached to each of the gusset plates on the main back, and these were connected by means of wire rope and rigging-screws to a set of 20-to-1 levers (*Figs. 2*) carried by the channels. Large buckets were suspended



from the free ends of the levers, and the load was varied by pouring in the required weight of water. The range of motion of each lever was restricted by stops, so that failure of any one bar would automatically remove the load and thus prevent damage to the other members of the frame. Each lever with its corresponding bucket was carefully calibrated, the resulting loads being determined by means of the laboratory testing-machine.

The stresses in the bars were inferred from the strains, as measured by the extensometer shown in *Figs. 3*. In this instrument the two concentric brass tubes A and B are an easy fit one in the other, and each carries a projecting knife-edge K which can be fitted into $\frac{3}{8}$ -inch gauge-holes. These holes were drilled 3 feet apart in the member to be tested. The tube B is pressed to the right by the

spring S, and when the instrument is not in use the tongue T rests against the inner stop. The outer end of B carries a micrometer gauge M, by which any variation of distance between K_1 and K_2 can be read off in ten-thousandths of an inch.

Owing to the fact that the work was done in the open, and extended over several days, the effect of thermal expansion was considerable. This was eliminated by taking readings before and after each experiment on a gauge G, which was placed near the truss and constructed of the same material.

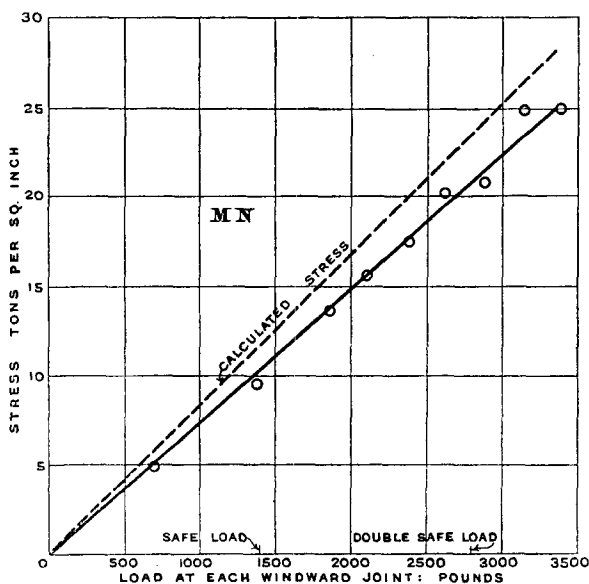
Some of the tension members were initially slightly bent, but of course straightened out when loaded. For this reason no readings were taken below one-fifth of the calculated ultimate load. The loads on the truss were increased in steps, the ratio between dead and wind loads being kept constant.

From elastic tests on material taken from various members of the frame, the value of Young's Modulus was found to lie between 27,500,000 and 28,500,000 lbs. per square inch. The variation therefore was not greater than 4 per cent., and a value of 28,000,000 was taken as the mean.

The extensions observed between the first two loadings give the increment of stress due to one-fifth of the calculated ultimate load. It was assumed that if the bars had been initially perfectly straight, the same extension would have been obtained between zero load and the first applied load; e.g., in J A the extension observed between the first and second loadings was 0.0078 inch, and the whole extension from zero to the second loading would therefore be 0.0156 inch. This indicated a stress of 5.45 tons per square inch on the full section of the bar between the gauge points, or a stress of 6.8 tons per square inch across the minimum or designed section, and so on throughout the range of the tests. In the Table on p. 327 the stress thus obtained is given for each tension member, together with that calculated from the stress diagram. This was drawn initially from the data given at the bottom of p. 320, and the calculation required for successive loads consisted merely in an alteration of its scale. Rupture occurred when dead and live loads reached values slightly in excess of that calculated from the known strength of the material, and took place across one of the gauge-holes in the bar M N. The observations taken by means of the extensometer indicated a stress of 9.9 tons per square inch upon the full section between the holes, namely, 0.117 square inch. The total load on the bar therefore amounted to 1.16 ton, which would produce a stress of 24.8 tons per square inch on the minimum

or designed section of the bar, that is, across the rivet-holes. The theoretical and actual stresses in this bar are plotted in *Fig. 4*. It will be seen that even at rupture the stress existing in the bar, except in the immediate neighbourhood of the gauge and rivet-holes, is well below the elastic limit; for the portion, therefore, on which the extensions were measured the stress would be proportional to the strain. The local yielding at the gauge-holes which would take place prior to rupture would be of very small extent, but would introduce a small error in the readings taken at or about the

Fig. 4.



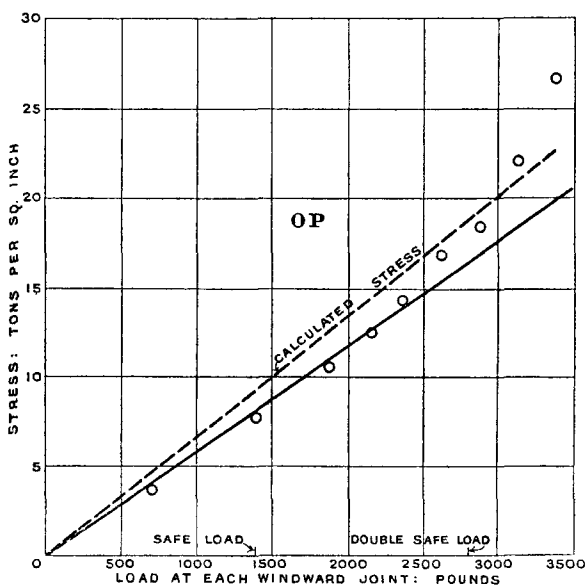
STRESSES FOR BAR M N WHICH FAILED.

maximum load. This would also be true for bar O P, which, as will be seen from *Fig. 5*, shows some departure from a straight line law. A similar curve was obtained for bar A P.

A new bar of slightly greater effective cross section was then substituted for the broken one, and the principal was again loaded up, when a general failure occurred with the same loading as previously. The truss suddenly failed and the free end jumped off the support. No bar was broken, the failure probably being caused by the temporary buckling of the main rafter.

It was hoped that it would be possible to obtain from the extensometer readings the stresses in both compression and tension members. The fact that the extensometer holes on the compression members could not conveniently be placed exactly on the axis, however, would cause any bending to alter the actual length of the material between the two gauge-holes, and therefore vitiate the readings. The bending would also be accentuated by the compressive forces, and it was evident from the commencement that no reliable observations could be made on these members of the truss

Fig. 5.



STRESSES FOR BAR O.P.

as constructed. On the other hand, in the tension members the gauge-holes were central, and bending actions would produce no alteration in length beyond the difference between the chord and the arc, which for the curvatures observed would be infinitesimal. The action of tensile forces also tends to reduce the curvature remaining after the small initial load, and makes the error introduced negligible.

The fact that the general failure took place very nearly at the calculated maximum load is a confirmation of the reliability of the

TABLE SHOWING THE OBSERVED AND CALCULATED STRESS ON THE
DESIGNED SECTION OF EACH MEMBER OF THE FRAME.

Number . .	1	2	3	4	5	6	7	8	9	Observed load Ratio Calculated load for safe load in Col. No. 2	
Load applied : percentage of calculated breaking load	21	42	58	66	74	81	89	97	104		
Member. (See Fig. 1.)	Stress : Tons per Square Inch.										
JA {	Obs.	3.4	6.8	9.1	10.3	10.8	12.6	13.6	15.9	16.6	0.75
	Calc.	4.5	9.0	12.2	13.7	15.4	17.0	18.6	20.3	21.9	
LA {	Obs.	3.8	7.6	10.1	11.5	12.8	14.5	15.7	18.2	19.8	0.85
	Calc.	4.5	8.9	12.1	13.7	15.3	16.9	18.5	20.1	21.8	
KL {	Obs.	2.5	5.0	7.5	8.0	8.4	9.1	10.7	10.1	10.5	0.43
	Calc.	5.8	11.7	15.9	18.0	20.2	22.3	24.4	26.5	28.6	
MN {	Obs.	4.8	9.6	13.7	15.7	17.3	20.2	20.7	24.8	24.8	0.82
	Calc.	5.8	11.7	15.9	18.0	20.2	22.3	24.4	26.5	28.6	
MP {	Obs.	4.4	8.9	10.5	13.2	14.3	15.7	16.9	18.8	19.3	0.94
	Calc.	4.7	9.4	12.8	14.5	16.2	17.9	19.6	21.3	22.9	
OP {	Obs.	3.8	7.7	10.6	12.2	14.3	16.8	18.5	22.2	26.8	0.82
	Calc.	4.7	9.4	12.8	14.4	16.1	17.8	19.5	21.2	22.8	
QP {	Obs.	1.8	3.6	4.3	4.9	5.9	6.0	5.9	7.6	7.6	1.06
	Calc.	1.7	3.4	4.7	5.4	5.9	6.5	7.1	7.8	8.4	
RS {	Obs.	2.2	4.5	6.1	7.2	7.0	7.4	10.0	7.4	8.8	1.03
	Calc.	2.2	4.4	5.9	6.8	7.4	8.2	9.0	9.8	10.6	
SP {	Obs.	1.7	3.5	3.7	4.8	4.9	5.4	6.6	6.5	6.4	1.03
	Calc.	1.7	3.4	4.7	5.4	5.9	6.5	7.1	7.8	8.4	
TU {	Obs.	No load in this member throughout test.									
	Calc.	2.2	4.4	5.9	6.8	7.4	8.2	9.0	9.8	10.6	
VA {	Obs.	2.1	4.3	5.4	6.3	7.4	8.8	9.5	10.1	10.7	0.72
	Calc.	3.0	6.0	8.2	9.2	10.3	11.4	12.7	13.6	14.7	
TA {	Obs.	3.1	6.0	8.3	9.3	9.5	11.2	11.9	13.4	13.3	0.92
	Calc.	3.3	6.6	8.8	10.1	11.2	12.4	13.4	14.7	15.9	
AP {	Obs.	3.9	7.9	10.1	12.7	13.5	15.6	17.4	21.4	24.9	0.92
	Calc.	4.3	8.6	11.7	13.2	14.8	16.3	17.9	19.4	21.0	

The Table gives for each member the stress as derived from the extensometer measurement and the stress taken from the stress diagram or reciprocal figure, the former stress being marked "observed" and the latter "calculated." The values for the safe load are given in column No. 2 and their ratio in the last column.

The safe load was taken as 560 lbs. dead load on each joint of both rafters, together with 840 lbs. wind load on each joint of one rafter.

usual methods of analysis as applied to a complete structure. It will however be observed that in the case of some individual members, the discrepancy between the observed and calculated stress is considerable. The difference, as might be expected, is least in the main tension members. It is important to notice that in almost every case the observed tensions lie below the calculated values; the amount corresponds doubtless with the bending moment taken up by the main rafter, which in theory is assumed to be pin-jointed, and in practice is continuous.

The Paper is accompanied by four drawings, from which the Figures in the text have been prepared.
