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Experimental Investigation of a 33 ft. Span Lattice Girder

Experimentelle Untersuchung eines Fachwerkträgers von 10 m Spannweite

Recherche expérimentale concernant une poutre en treillis de 10 m de portée

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Introduction

If failure of a plane triangulated steel truss is brought about by buckling of one of its compression members, the actual force required to produce this buckling is dependent on the geometrical properties of the member, the conditions of end fixity and the effective length of the member. Where the joints have a certain degree of rigidity the end fixity is a function of the stiffness of the adjacent members and to a lesser extent, of the other more remote members; and hence, accurate information on the behaviour of the individual members can only be ascertained from tests carried out on complete trusses.

The tests described in the following paper were carried out on a single span girder manufactured from mild steel conforming to B.S. specification No. 15. The girder was manufactured and fabricated by Messrs. Dorman Long & Co., Middlesbrough, in accordance with their normal procedure for structural steelwork of this type. The gusset plates were riveted to the top and bottom chord members, the connections to the web members being made with M.S. black bolts.

The main object of the investigation was to obtain information on the ultimate carrying capacity of light latticed girders, and the behaviour of their compression members under practical working conditions. Since strain readings were also made on several tension members the authors feel that publication of the results obtained may be of interest to those engaged on structural steelwork of this nature.

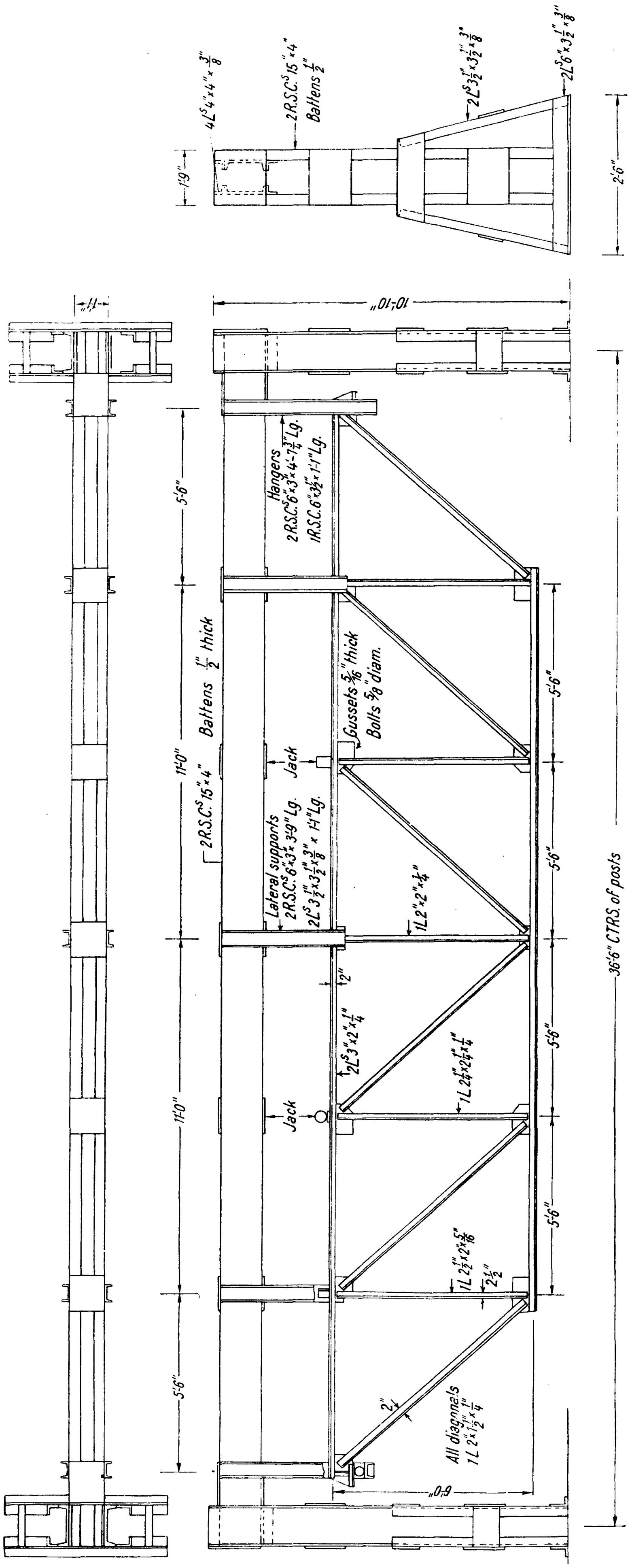
Hangers
Lateral supports
Jack
Jack
Battens 1/2" thick
Gussets 5/16" thick
Bolts 5/8" diam.
36'6" CTRS. of posts
All diagonals

Fig. 1

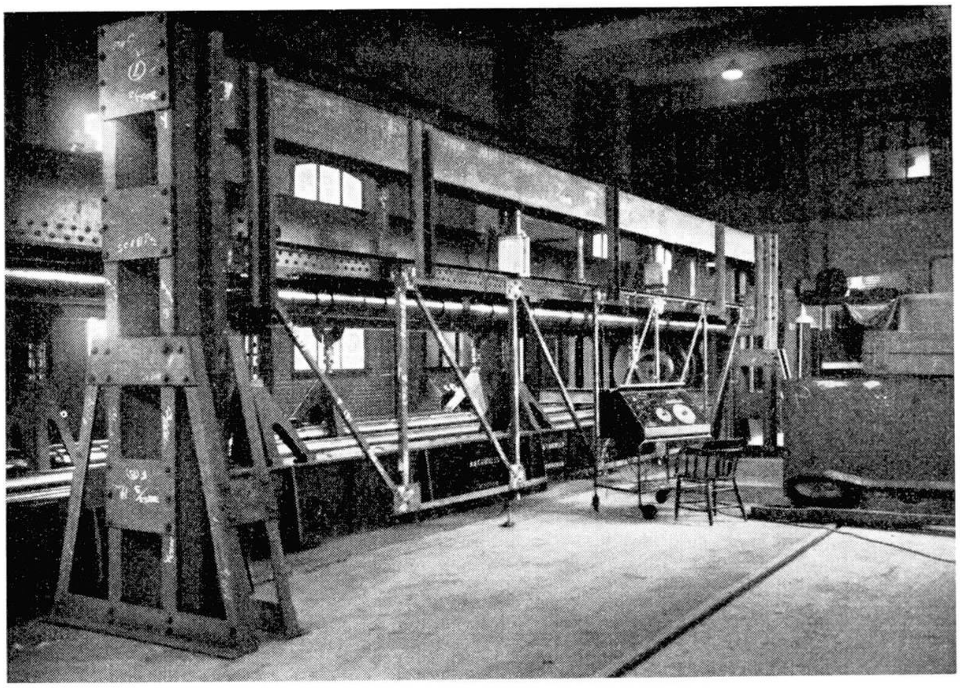

Plate 1

Description of Girder and General Programme of Tests

The general features of the test girder are shown in Fig. 1 and Plate 1. The panels measured 6′ 0″ high by 5′ 6″ long, giving a total girder length of 33′ 0″ centre to centre of bearings. To reduce the danger of premature failure due to lateral instability of the top chord the supports were raised as shown in the figure, thereby eliminating the need for end verticals. At the time of testing the girder was too large to be conveniently set up in any of the available test machines at the University of Leeds: a loading frame was therefore designed and constructed to accommodate the girder, the load being applied to the upper chord at panel points C and E by means of calibrated hydraulic jacks, each having a capacity of 12 tons. The general features of the loading frame are given in Fig. 1.

The load was applied in incremental stages, complete sets of strain and deflection readings being recorded at each stage. To differentiate between elastic deformation and plastic deformation or bolt-slip, the loading was carried out in cycles, zero load readings being taken before and after each cycle.

Strain measurements were taken on the various truss members with the aid of $100\,\Omega$ electric resistance strain gauges. Since not more than two 50-way strain recorders were employed at any one time, it was necessary to take two or more loading runs to read all the gauges. Huggenberger extensometers were also used to obtain additional and check readings and so verify the shape of the stress-distribution diagrams for the individual members.

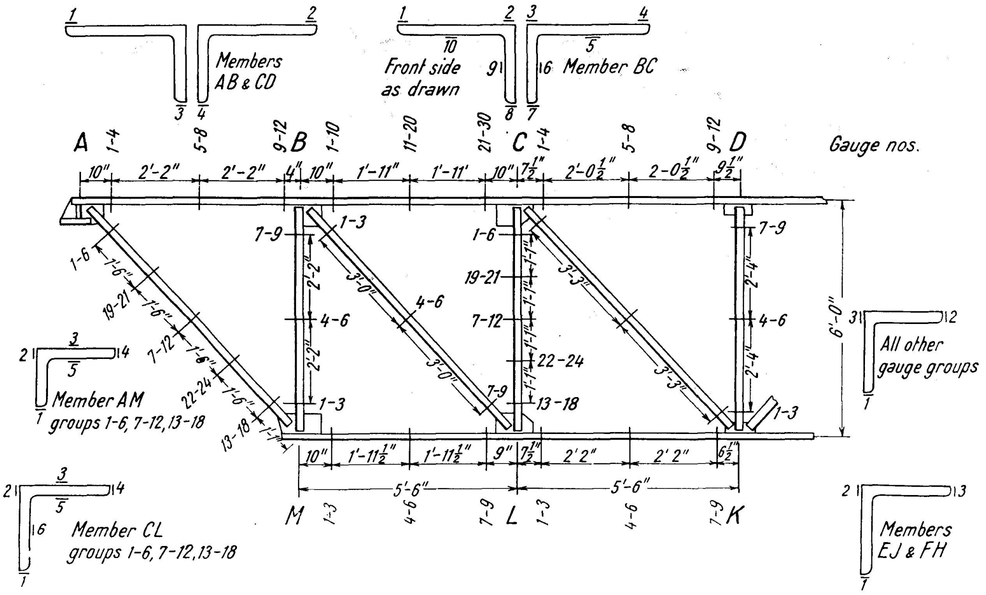

Fig. 2

For ease of reference the truss members are denoted by the letters corresponding to the joints at their extremities, thus $A\,B$ denotes the member connecting joints A and B. The strain gauges on each member have been numbered consecutively from 1 upwards, the disposition of the gauge groups being shown in general form on Fig. 2.

Test Results

The girder was first tested in a fully-bolted condition (Series I tests) and the forces in the members computed from the recorded strain gauge readings. Load was applied in incremental stages of 1.0 ton up to a maximum limit of 6.0 tons to avoid premature failure of the girder. After a sufficient number of readings had been taken, a second run of load was carried out with single bolted connections to all web members (Series II tests). When these tests were concluded the girder was loaded with a progressively increasing load until its ultimate carrying capacity was reached. The discussion of the test results described in the following sections is primarily concerned with the performance of the girder within its elastic range. For information on its behaviour up to the ultimate load the reader is referred to the remarks on the "Crippling Run".

Deflection

Vertical deflection readings were taken at the lower panel points J and L for each load increment, allowance for bolt slip being made by repeatedly loading and unloading the girder. Since the load-deflection graph for any one load cycle did not form a smooth curve due to erratic rotation of the joints,

Fig. 3 has been prepared by taking the mean readings from a number of cycles. Fig. 3a compares the actual and theoretical deflections (obtained from the Williot diagram) at the third panel points of the bottom chord for the Series II tests. The actual deflection curve does not include the deflection due to bolt slip. Fig. 3b shows a similar comparison for the fully-bolted girder except that in this case the graph has been extended to include the readings taken during the "Crippling Run".

It is evident from the figure that up to a total girder load of 2.0 tons in the case of the singly-bolted connections or 3.0 tons where the web-members are fully bolted, the measured vertical deflections agree very closely with the values obtained from the Williot diagram, being slightly less than the latter in the Series I tests. Thereafter the ratio of experimental to computed deflection progressively increases with increasing load until at the limiting loads of 6.0 tons in the case of Series I tests and 5.0 tons for the Series II tests it has values of 1.064 and 1.114 respectively. The comparative value for the Series I tests at 5.0 tons applied load is 1.055. The reduction in the ratio of experimental deflection/computed deflection from 1.114 to 1.055 is a measure of the increased joint rigidity obtained by using fully-bolted connections.

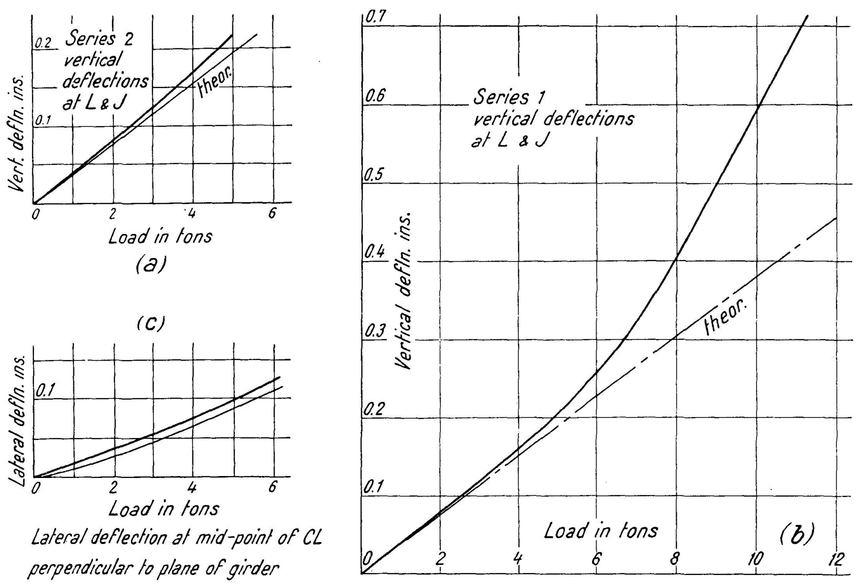

Fig. 3

In Fig. 3c the deflections of the mid-point of member CL perpendicular to the plane of the truss, are given for both Series I and II tests. Inspection of these graphs shows that up to a girder load of approximately one ton the load-deflection curves diverge and thereafter remain parallel as the girder load is increased. The vertical intercept between the parallel portions of the curves is a measure of the reduction in vertical deflection obtained by introducing fully-bolted joints instead of using single bolts. This reduction is governed by the least of three factors, viz.: the moment of resistance of the member itself, the bolted joint, or the gusset plate perpendicular to its plane. In this case bending of the gusset obviously forms the ruling factor and full resistance is developed at a girder load of one ton. Deflection readings of the midpoint of $C\,L$ in the plane of the truss were also recorded. Due to the greater influence of the end fixity in this plane the readings were not strictly correct and hence have not been given in the paper. Comparison of the results obtained however, definitely indicated a very marked reduction in deflection for the Series I tests.

Stress Distribution in Members

Diagrams of stress-distribution taken at cross-sections at the centre and near the ends of typical members are shown in Figs. 4, 5 and 6.

In the web-compression member $C\,L$, the stress variation from heel to toe of the connected leg is due to secondary bending in the plane of the truss, caused by joint rigidity and eccentricity of the end connections to the member. The variation across the outstanding leg results solely from the eccentric end

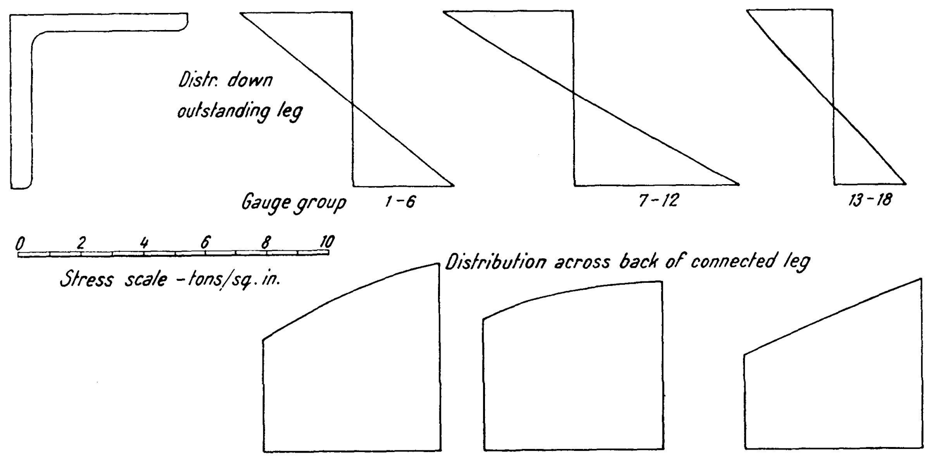

Fig. 4

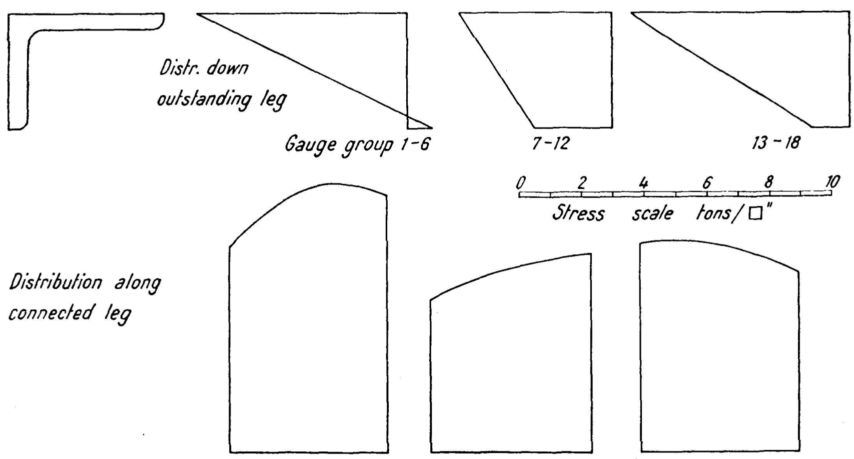

Fig. 5

connections except for the minor influence of the gusset plate rigidity perpendicular to its plane. The bending stress in the outstanding leg increases as the mid-length of the member is approached. This is due to the added eccentricity of loading arising from bending of the member in single-curvature perpendicular to the plane of the truss. In the connected leg the reversal of bending stress occurred at the central section at approximately 9.5 tons girder load, indicating that the stresses on that section at lower loads were considerably influenced by initial imperfections in the member.

Fig. 5 shows the stress-distribution over three cross-sections of a typical tension member, in this case, member $A\,M$. There is an obvious tendency to even-out the fibre stresses due to bending in both planes, towards the mid-length of the member. Bending of $A\,M$ in the plane of the truss occurs in double-curvature as shown by the reversal of bending stress along the connected leg. The high bending stresses in this leg close to joint A confirm the relatively higher bending moments due to eccentricity of the forces in the members, which occur at this joint, in comparison with joint M.

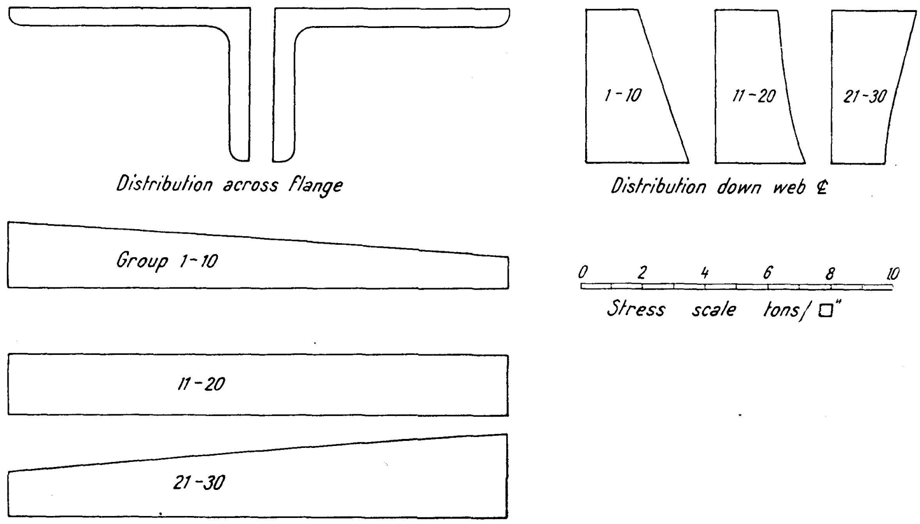

Fig. 6

The stress distribution in the double-angle compression chord is shown in Fig. 6. Here, as with the member $A\,M$, bending occurred in double-curvature in the plane of the truss. The stress variation across the flange of the chord indicates horizontal bending in the member. Reversal in the direction of the bending fibre stresses from B to C further denotes that this bending occurred in double-curvature between the joints. It is largely due to the forces in the diagonal web members acting at an eccentricity $e = (v + \frac{t}{2})$ from the plane of the truss, where v is the distance from the face of the connected leg to the centroid of the angle and t is the thickness of the gusset plate. The effect of this eccentricity can be seen from the horizontal elastic line for the top chord shown on Fig. 7. Without the central lateral supports the chord would bend laterally under the given loading conditions over a length at least equal to L' on the figure. The restraining effect of the supports at D became noticeable at an early stage in the test due to initial eccentricity of the chord which made it bear hard against the front supporting guide from the commencement of the test. In this connection it should be noted that the horizontal bending moment in this member at the central lateral support increased with increasing load up to a total girder load of 4.0 tons, and thereafter remained constant. Due to their length, the loading jacks were comparatively flexible and could offer no serious lateral restraint at the points of application of load.

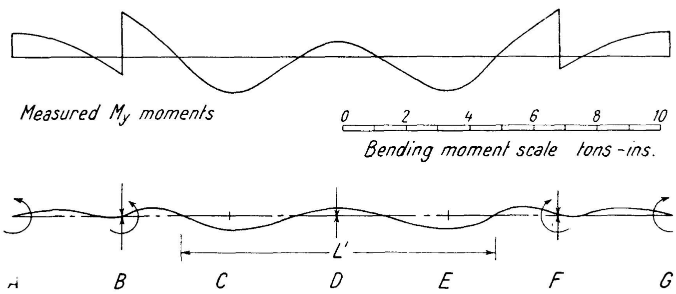

Fig. 7

Fig. 7 shows the influence of the lateral joint-moments in reducing the effective length of this member for loads up to the safe working load for the truss and so increasing its stability against lateral buckling. In view of the influence of initial imperfections at low loads and buckling of the chord under strut action at higher loads, it would be unwise to make use of this local stiffening effect for design purposes.

Effect of Increasing Load on Moments and Stresses in Members

The effect of increasing load on the bending moments in the members has been investigated for $C L$ and $A M$. The results of this investigation for M_x moments in the plane of the truss and M_y moments perpendicular to that plane are shown in Figs. 8 and 9.

Rigidity of the gussets in the plane of the truss at C and L was sufficient to prevent $C L$ from buckling about the oblique axis giving the least radius of gyration. Ultimate failure occurred by outward buckling of the member about the rectangular axis as indicated in the M_y diagrams.

The slight reversal of curvature due to the restraining influence of the gusset plates against bending perpendicular to their plane is most noticeable at the higher loads. The extent of this restraint is, however, insufficient to justify any reduction in the effective length of the member when considering buckling at right angles to the plane of the truss. For this criterion of failure the effective length should be taken as the actual length between top and

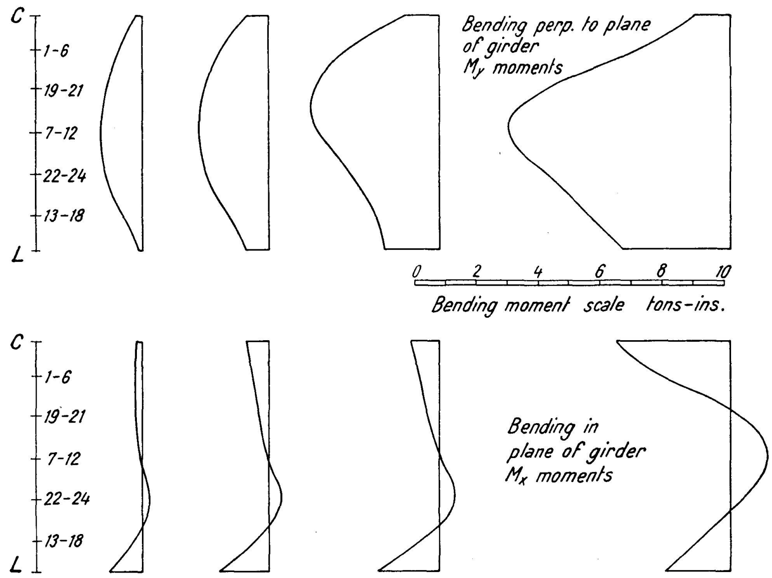

Fig. 8

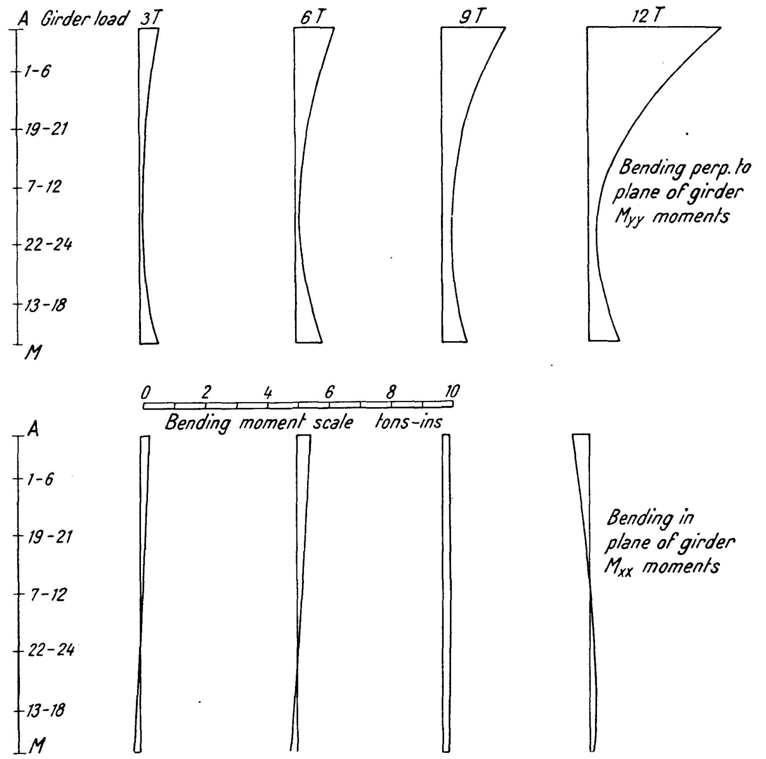

Fig. 9

bottom chord intersections measured along the axis of the member and any eccentricity of loading due to the nature of the end connections should be taken into account in design. The effect of attachment by one leg only is shown by the presence of M_y moments at C and L in the strut examined.

Joint rigidity has a marked influence on bending of $C\,L$ in the plane of the truss as witnessed by the shape of the M_x diagrams particularly at the higher loads. With increasing girder loads both the central and terminal B.M.'s in the member increase. As the latter have been extrapolated from the slope of the moment diagrams at the nearest strain gauge sections the actual moments may differ appreciably from the values shown on the figures. The maximum values of the actual end moments will be governed by the least of the following factors: (1) the applied moment Fe where F is the force in the member and e is its eccentricity measured from the centroid of the bolts, in the plane of the truss; (2) the moment of resistance of the connecting bolts against rotation in this plane, or (3) the moment of resistance of the gusset plates themselves. However, with the adoption of normal practice in joint design, that is, at least two bolts at each end, it appears that sufficient joint restraint can be relied upon to justify taking the effective length of the web compression members as half the actual length when considering bending in the plane of the truss. At loads near the ultimate the effect of initial imperfections becomes insignificant.

In the case of member $A\,M$ the high rigidity of the bearing connection perpendicular to the truss resulted in an applied M_y moment in the member at end A greater than the total external moment due to the eccentricity of pull in this plane. The M_y terminal moment at end M was much smaller in magnitude due to the greater flexibility of this joint. A rapid decrease in moment perpendicular to the truss-plane occurred along the length of $A\,M$ from both ends towards the centre, the value of the central moment being approximately one-fifth the mean value of the end moments at a girder load of 12 tons.

Considering bending in the plane of the truss, comparison of the (M_x) diagrams for $A\,M$ and $C\,L$ shows the difference between tension and compression members in resisting secondary moments due to joint rigidity. Since the joint stiffness in a lattice truss is a function of the stiffness of the several members intersecting at the joint, any alteration in the stiffness of these members will affect that of the joint. As the truss loading increases the stiffness of the individual members will decrease or increase depending on whether they are in compression or tension respectively. Referring to member $A\,M$ this is equivalent to stating that as the girder load is increased the stiffness of the adjacent joints relative to $A\,M$ decreases thus preventing any increase in the terminal moments due to distortion of the truss. This can be readily seen in Fig. 10 and leads to the general conclusion that for light trusses designed and constructed in accordance with normally accepted practice, the influence of the M_x moments is not sufficient to warrant consideration in the design of tension members.

Fig. 5 shows evidence of high stress concentration in the connected leg of member $A\,M$ close to joint A and slight concentration near joint M. Along the central portion of the member more equitable distribution of the stress is obtained. As the girder load was increased up to its limit, no major difference in the proportions of load carried by the connected and outstanding legs of member $A\,M$ was observed. The stress distribution at any intermediate load can therefore be taken as representative of the stress conditions existing up to the point of failure.

The effect of increasing load on the stress distribution across member $A\,M$ at a section XX close to end A and represented by gauges 1—6 is shown in Fig. 10. Increasing the girder load from 3 tons to 9 tons raises the mean stress in the connected leg at this section from 3.3 to 11.7 tons per sq. in. The central peaks on the distribution diagrams for this leg are due to stress concentration

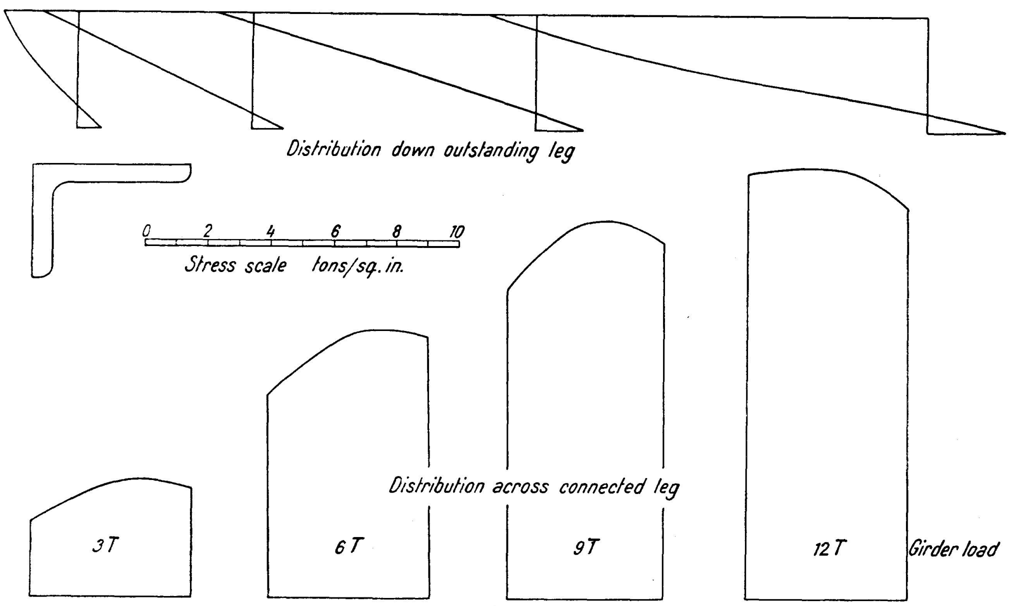

Fig. 10

from the bolts at end A. The stress due to bending in the plane of the truss is measured by the difference between the fibre stresses at the toe and heel of the connected leg. The more even distribution of stress across this leg at the 12 ton girder load must be due to yielding of the member at the reduced section $Y\,Y$ through the lower bolt hole which would automatically re-distribute the bending stresses.

The effective area of $A\,M$ for design purposes is given by B.S.S. 449, 1948 as 0.48 sq. in., the corresponding safe load of the member being 4.3 tons. From tensile tests carried out on specimens cut from the central portion of $A\,M$ the modulus of elasticity (E) for the material was found to be 13,000 tons per sq. in. and the yield stress 16.8 tons per sq. in. Using the latter value of the yield stress and the nett cross-sectional area as specified in B.S.S. 449 the yield load of the member has been evaluated at 8.05 tons. As given above, the mean stress in the connected leg at section $X\,X$ for a girder load of 9 tons was 11.7 tons per sq. in. representing a direct load in the member of 6.95 tons. This stress represented a stress of 15.1 tons per sq. in. at section YY through the lower bolt hole. For the 12 ton girder load the measured direct force has been evaluated at 8.80 tons, the corresponding mean stresses in the connected leg being 13.90 and 18.80 tons per sq. in. at sections XX and YY respectively.

From this it must be inferred that partial yielding of the member at section YY occurred before the 12 ton load was reached. Inspection of the stress distribution graphs at intermediate loading stages revealed marked evidence of this when the measured force in the member reached 8.03 tons. The close agreement between the theoretical and measured values of the yield load for AM justifies the method adopted in B.S.S. 449 for estimating the strength of tension members of this type.

Direct Forces and Moments in Truss Members

The experimental and theoretical forces and moments in the individual members of the girder are compared in Tables I and II. Table I, which refers

Table I

Member	Joint	Theor. Load	Exptl. Load	Ratio (4) : (3)	Theor. M_{xx}	Exptl. M_{xx}	Theor. M_{yy}	Theor. M_{yy}	Exptl. M_{yy}
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
AB	A	−2.75	−2.95	1.07	−2.17	−3.00	−1.39		−0.70
	B				−1.27	−0.65	−0.68		−0.36
BC	B	−5.50	−5.60	1.02	+0.11	−0.65	−0.68		−1.20
	C				+0.22	−0.42	0		−1.30
CD	C	−5.50	−5.70	1.03	+1.06	+1.90	0		+1.30
	D				+0.53	+0.70	0		+0.40
AM	A	+4.06	+4.06	1.00	−0.67	−0.23	+2.30		+0.70
	M				−0.59	−0.38	−2.30		−0.80
BL	B	+4.06	+3.89	0.96	−0.01	−0.02	+2.30		+0.80
	L				−0.39	−0.26	−2.30		−0.70
CK	C	0	0		+0.32	−0.01	0		0
	K				+0.14	0	0		0
KD	K	0	−0.07		0	+0.02	0		−0.10
	D				0	+0.08	0		+0.05
ML	M	+2.75	+2.42	0.88	−0.93	−0.16	+2.02		+0.60
	L				−1.03	−0.17	−1.01		−0.40
LK	L	+5.50	+5.46	0.99	−0.46	−0.20	+1.01		+0.20
	K				−0.29	−0.61	0		+0.40
MB	M	−3.0	−2.76	0.92	−0.46	−0.40	+2.12	+1.28	+0.60
	B				+0.05	+0.30	−2.12	−1.28	−0.40
FH	F	−3.0	−2.76	0.92	−0.05	−0.85	−2.12	−1.28	−0.45
	H				+0.46	+0.30	+2.12	+1.28	+1.10
LC	L	−3.0	−3.02	1.01	−0.20	−0.90	+2.17	+1.55	+0.70
	C				+0.23	+1.00	−2.17	−1.55	−0.90
EJ	E	−3.0	−2.96	0.99	−0.23	+0.50	−2.17	−1.55	−0.70
	J				+0.20	+0.25	+2.17	+1.55	+1.10

to the Series I tests, is drawn up for an applied girder load of 6.0 tons whereas in the case of Table II, referring to the Series II tests for the singly-bolted condition, the corresponding girder load is 5.0 tons.

Table II

Member	Joint	Theor. Load	Exptl. Load	Ratio (4) : (3)	Theor. M_{xx}	Exptl. M_{xx}	Theor. M_{yy}	Theor. M_{yy}	Exptl. M_{yy}
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
AB	*A*	−2.29	−2.70	1.18	−1.81	−2.40	−1.16		−0.75
	B				−1.06	−1.70	−0.58		−0.20
BC	*B*	−4.58	−4.74	1.03	+0.09	−0.02	−0.58		−1.10
	C				+0.18	0	0		−0.30
CD	*C*	−4.58	−4.65	1.01	+0.88	+1.45	0		+0.30
	D				+0.44	+0.80	0		−0.25
AM	*A*	+3.38	+3.44	1.02	−0.56	−0.30	+1.92		+0.75
	M				−0.49	−0.40	−1.92		−0.75
BL	*B*	+3.38	+3.52	1.04	−0.10	−0.08	+1.92		+0.60
	L				−0.33	−0.16	−1.92		−0.70
CK	*C*	0	+0.71		+0.27	−0.05	0		0
	K				+0.17	−0.02	0		0
KD	*K*	0	+0.12		0	+0.20	0		+0.05
	D				0	+0.40	0		+0.10
ML	*M*	+2.29	+2.08	0.91	−0.73	−0.16	+1.68		+0.70
	L				−0.86	−0.48	−0.84		−0.10
LK	*L*	+4.58	+4.23	0.93	−0.39	−0.20	+0.84		+0.80
	K				−0.24	−0.20	0		−0.10
MB	*M*	−2.50	−2.26	0.91	−0.38	−0.40	+1.72	+1.05	+0.65
	B				+0.04	+0.09	−1.72	−1.05	−0.60
FH	*F*	−2.50	−2.52	1.01	−0.04	−0.50	−1.72	−1.05	−0.65
	H				+0.38	+0.60	+1.72	+1.05	+0.60
LC	*L*	−2.50	−2.25	0.90	−0.17	−1.00	+1.81	+1.29	+0.70
	C				+0.19	+0.30	−1.81	−1.29	−1.10
EJ	*E*	−2.50	−2.42	0.97	−0.19	−0.40	−1.81	−1.29	−0.40
	J				+0.17	+0.20	+1.81	+1.29	+1.00

Forces

In both tables, column (3) shows the theoretical forces calculated on the assumption that all joints are frictionless pins. The corresponding observed forces are given in column (4) and the ratio $\frac{\text{observed force}}{\text{calculated force}}$ for each member is shown in column (5). Considering the comparatively small forces involved, the agreement between the computed and measured forces is reasonably good. The mean error for the stressed members in Table I is 2%, that for Table II being 3%. The corresponding maximum errors are 12% and 18% respectively. In spite of the rather large discrepancies in a few individual cases the agreement on the whole confirms the usual method of analysis of lattice girders of this type.

Moments in the Plane of the Truss (M_x moments)

On the assumption of fully rigid joints, terminal moments due to the combined effects of joint rigidity and eccentricity of connections in the plane of the truss, were computed by Manderla's method. The results are shown in column (6) and compared with the measured moments given in column (7). The latter values were deduced by extrapolation from the moment curves obtained from the cross-sections examined on the various members. Although agreement between the experimental and theoretical values is poor, the moments do compare, indicating that the girder is acting with partially stiff joints. It follows from this that even with light lattice girders having comparatively slender members, an accurate forecast of the manner in which buckling of the individual members in the plane of the girder will occur, is possible from computation of the theoretical secondary moments in that plane.

The absence of any major difference in the end moments between Series I and Series II tests is due to frictional bond developed between the members and their end gusset plates due to the tightness of the nuts on the single-connecting bolts in Series II. With increasing load a stage would eventually be reached when no further increase in bond could be developed by single bolts and slipping at the joints must ensue. Thereafter marked differences would occur between the two sets of moments, provided that such stage occurred before ultimate capacity of the girder was reached.

The excess of the experimental joint moments over the corresponding theoretical values at A and C is probably due to the external loads being eccentrically applied to the panel points. Calculations based on this assumption revealed that load eccentricities of 0.13 and 0.11 in. for joint A in the Series I and II tests respectively would account for the discrepancies. The corresponding figures for joint C were 0.22 and 0.06 in. respectively. All these figures are within the limits of experimental accuracy in dealing with girders of this type and size. However, they are based on the assumption of fully-rigid joints and should be slightly increased if only partial fixity is attained at the girder joints. The discrepancies between observed and computed M_x end moments for the other members may be due to any of the following causes:

1. Errors due to extrapolation from the moment curves for the main lengths of the members.
2. Over-estimation of the degree of fixity attained at the various girder joints.
3. Discrepancies between the actual stiffness factors for the individual truss members and those assumed in the calculations.
4. Variation in the actual E.R.S. gauge factor from that adopted in the basic stress conversion computations.

The determination of the relative importance of the above factors in causing the discrepancies is both unreliable and difficult and certainly not warranted in the present instance.

Moments perpendicular to the Plane of the Truss: (M_y moments)

The calculated M_y moments for full eccentricity $\left(e = v + \frac{t}{2}\right)$ are given in column 8 of Tables I and II. The corresponding experimental values, obtained by extrapolation from measured values are given in column 10.

Considering all members except the compression chord which has already been considered, it is found that the measured values are below those given in column 8. From this it is evident that some restraint is developed by the gusset plates against bending perpendicular to the plane of the girder. B.S.S. 449 makes allowance for this by adopting a reduced value given by $\left(v - \frac{t}{2}\right)$ for the eccentricity in single angle struts connected by one leg. The theoretical M_y moments calculated on this assumption are given for the discontinuous strut members in column 9 and afford closer agreement with the measured values.

Crippling Run of Load

From observations taken during the preliminary loading cycles it was anticipated that failure of the girder would result from buckling of member CL or possibly yielding of AM. Accordingly attention was concentrated on these two members during the final application of load. Readings were also taken on the other four vertical members.

General Behaviour of Girder up to Collapse

As the final loading was applied the deflection readings for the bottom chords lay on a smooth curve up to the previously attained maximum girder load. Thereafter irregular bolt slip occurred rendering the plots erratic. Allowance for this has been made in Fig. 3b by taking the mean readings for panel points L and J and smoothing out any obvious bolt slip.

When the total load reached a value of 13 tons a fair amount of bowing perpendicular to the plane of the truss was visible in the vertical struts. At this stage a slight dropping off of load on the jacks became noticeable indicating yield at some point in the girder. A small increase in the applied load then caused sudden failure of member CL by outward buckling as shown in Plate 2. Readings taken of the movement of the mid-point of CL relative to its ends were invalidated at the higher loads by the erratic movement of the girder, causing displacement of the dial gauge points. The errors were accentuated by a slight twisting of the loading frame cross beam. Visual inspection of the member immediately prior to and after buckling revealed a slight tendency to buckle about that axis giving the least radius of gyration. Since the actual plane of buckling is almost at right angles to that of the girder, this tendency can be safely ignored in formulæ for the crippling load of the member.

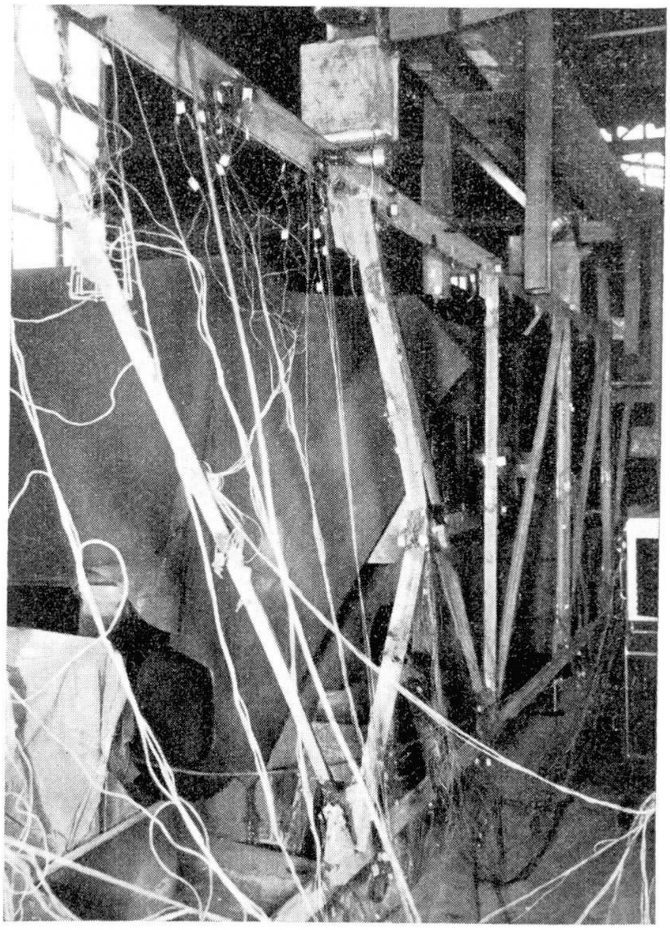

Plate 2

Comparison of Experimentally deduced Crippling loads with those given by standard Strut Formulæ

Values of the crippling stress in the vertical compression members have been calculated by the Ayrton-Perry formula for eccentrically loaded struts as given below.

$$p = \frac{P_y + (\eta_1 + 1)\, P_e}{2} - \sqrt{\frac{P_y + (\eta_1 + 1)\, P_e^2}{2} - P_y\, P_e}$$

where $$\eta_1 = 0.003\,\frac{l}{k} + \frac{6}{5}\,\frac{a \cdot e_0}{k^2}$$

Eccentricity of loading (e_0) has been allowed for by assuming the end loads to be applied at the mid-point thickness of the attached leg. A similar procedure to that given in B.S.S. 449, of neglecting the effect of bending in the plane of the truss has been adopted.

Two cases have been considered for each member; in one, the effective length l has been taken as equal to the actual member length ($L = 72$ in.) whereas in the other a value of $l = 0.8\,L$ has been chosen. In all cases the figures for the yield stress have been determined from tensile tests carried out on pieces cut from the actual members at the conclusion of the tests. The values obtained are compared with the actual or estimated crippling loads derived from the test results in Table III.

It appears from inspection of this table that the actual crippling load of an eccentrically loaded angle strut forming part of a lattice girder lies between the values given by adopting the above formula and taking values of $l = 0.8\,L$ and $1.0\,L$. It should be noted that the ultimate loads have been computed by considering failure about the axis of eccentricity and not about the axis giving the least radius of gyration as recommended in B.S.S. 449. The safe load values derived from table 8 in B.S.S. 449 give load factors of 2.7 and 2.4 for members CL and MB respectively. Since table 8 referred to above has been derived by adopting a load factor 2 it follows that the B.S.S. methods for estimating the crippling load must be conservative. Whether this is universally true or not can only be determined from further tests carried out on a wider range of angle sizes and lengths.

Conclusions

A summary of the main conclusions derived from the investigation, in so far as they affect the practical design of light lattice girders, is set out below:

1. The assumption that the stress distribution is planar across truss members having a slenderness ratio of about 130 in accordance with B.S.S. 449, 1948, is justifiable at all sections except those close to the end connections where stress concentrations due to the connecting rivets or bolts may seriously affect the distribution in the connected leg and to a lesser extent that in the outstanding leg.

2. With light type, bolted girders having the end supports close to the compression flange, the actual deflections, excluding the amount due to bolt slip, agree fairly closely with those computed by the standard method, provided that the members are connected to the gussets with at least two bolts at each end. In such cases the measured value is likely to be about 5 per cent in excess of the theoretical deflection when the applied load is equal to the calculated safe load for the girder.

3. Good agreement between the measured forces in the truss members and those calculated on the assumption of pin-jointed connections confirms the reliability of this method for the analysis of forces in plane triangulated frames.

4. In the case of tension members consisting of single angles connected through one leg, the test results confirm the method of determining the effective area as set out in B.S.S. 449, clause 39a.

The effect of eccentricity of the direct load at right angles to the plane of the gusset is a maximum close to the end connections and diminishes as the mid-length of the member is approached. Where this has to be included in design, the allowance to be made will depend upon the rigidity of the gussets at right angles to their plane. With rigid-stiffened gusset plates full eccentricity of load should be allowed for in calculating the bending fibre stresses close to the ends of the member. Where the gusset plates are of similar size and rigidity to those commonly employed in light Pratt trusses with two bolt connections, rotation of the gussets perpendicular to the plane of the truss reduces the moment due to eccentricity of load in this plane down to about half full value. High concentrations of direct stress exist in the material close to the bolt or rivet holes. If normal practice in regard to size and spacing of bolt holes is adopted, the effect of these concentrations need not be considered in practical design since local yielding of the material under such concentrations gives a more equitable distribution of the stress over the cross-section of the material as soon as the total load reaches a sufficiently high value.

5. The gusset plates used in riveted or bolted lattice girders offer considerable rigidity against bending of the web members in the plane of the girder. Due notice should be taken of this factor in calculating the safe loads for the compression web members. When bending in this plane is the criterion for design, the ratio of the effective length to the intersection length may have a range of values from about 0.75 down to 0.50 for connections of the type commonly employed in this class of work.

In parallel-chord trusses with continuous top and bottom chord members the test results show that where the web-compression members have at least two-bolt connections the effective length in the plane of the truss should be taken as one half the intersection length.

6. Where bending of the members perpendicular to the plane of the truss is considered the rigidity of the gussets at right angles to their plane reduces the effective length slightly below the intersection length at low loads. As the load in the member is increased the effective length increases until at the ultimate load of the member it should be taken as the intersection length for design purposes.

7. For consideration of bending about the oblique axes the effective length of single-angle compression members should be given an intermediate value between the limiting cases given in conclusions 5 and 6 above. Where the connected leg is not less than the outstanding leg the value taken should be closer to that given by (5) than to the intersection length. Further tests on a variety of different size angle compression members are necessary to obtain more exact information on this subject.

8. Close inspection of the behaviour of the experimental truss under load shows that for continuous compression chord members the effects of lateral buckling can be assessed on the assumption of bending in single-curvature between effective lateral supports. The force required to give lateral support at any point along the length of the chord is not severe and can be easily developed with light lateral bracing or tie members.

9. From observations on a limited number of single-angle compression members only one of which was loaded up to its ultimate capacity, the permissible stress values given in table 8 of B.S.S. 449, 1948, appear to be conservative.

Failure of the experimental truss was brought about by crippling of one compression member, and in the case of a second member its crippling load could be estimated within close limits by extrapolation from the strain gauge readings. Two values of the ultimate load for compression members were therefore obtained for comparison with the safe loads for these members computed on the basis of the permissible stresses given in B.S.S. table 8. In these cases the load factors obtained were respectively 2.7 and 2.4.

In view of the above results the crippling loads for the members in question were calculated in accordance with the method suggested by AYRTON and PERRY for struts with eccentric loading based on the conditions laid down in B.S.S. 449 Appendix *D* for the derivation of table 8 and considering buckling about the axis of applied eccentricity. For comparison purposes the experimental loads and safe loads obtained from table 8 have been included with these values in Table III below.

Table III

Section	Slenderness ratio $\frac{0 \cdot 8L}{r_{min}}$	B.S.S. 449 Safe Load (tons)	Ultimate Load in Tons		
			Ayrton-Perry ($l = 1.0\ L$)	Ayrton-Perry ($l = 0.8\ L$)	Estimated or measured
$2\frac{1}{4} \times 2\frac{1}{4} \times \frac{1}{4}$	131	2.49	5.94	6.90	6.66
$2\frac{1}{2} \times 2 \times \frac{5}{16}$	137	2.89	6.02	7.06	6.90

Inspection of this table shows good agreement between the experimental loads and those computed by the AYRTON-PERRY method whereas the crippling loads derived from table 8 on the assumption of a load factor 2 are much lower than those obtained experimentally. It would appear from this that some alteration in the method of computing safe loads for compression members is justified but further work is necessary before definite conclusions can be drawn concerning this point in design.

Acknowledgements

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Summary

This paper deals with loading tests carried out on a 33 ft. span mild steel, bolted lattice girder. The loading which was symmetrically applied at two panel points of the upper chord, was raised in incremental stages until failure of the girder occurred by buckling of a vertical web member under one of the loading points. Strain gauge readings were obtained for all members in one-half of the truss at various stages of the loading for both fully-bolted and singly-bolted connections to the web members and from these recordings stress-distributions and secondary bending moments were computed for the members forming the girder. The strains used in the subsequent analysis were recorded by means of electric resistance gauges supplemented by Huggenberger extensometers. Readings were taken close to the ends and at the mid-lengths of all members examined, from which the variation of bending moments along the lengths of the members was obtained.

The experimental results were compared with those computed by the current methods for design of such girders and the following general inferences drawn.

1. Good agreement between the measured direct forces and those computed by assuming frictionless pin joints confirms the reliability of the latter method of computing the forces in girders of this type.

2. The method set out in British Standard Specification No. 449,1948, for the design of tension members connected by one angle leg is adequately confirmed by experiment.

3. Certain modifications appear to be desirable in the methods set out in the same specification for estimating safe loads for single angle compression members, particularly in regard to the effective lengths to be adopted and the axis about which failure is to be computed.

4. Due to effects of eccentricity of loading in the members arising from the end connections, only poor agreement was obtained between the experimental secondary moments and those computed by the Manderla method for secondary stress analysis.

5. Vertical deflections at the intermediate panel points of the lower chord agreed fairly well with the theoretical deflections in the case of the fully bolted girder. Where the web members were connected by single bolts the measured deflections appreciably exceeded the corresponding theoretical values.

Zusammenfassung

Der Aufsatz beschreibt Belastungsversuche, die an einem verbolzten Ständerfachwerk aus Flußstahl von 10 m Spannweite ausgeführt wurden. Die an zwei Knotenpunkten des Obergurtes symmetrisch angebrachte Belastung wurde in kleinen Intervallen vergrößert, bis das Versagen des Trägers durch Ausknicken des Pfostens unter einem der Lastangriffspunkte eintrat. Ablesungen der elektrischen Spannungsmesser wurden für alle Teile der einen Trägerhälfte bei verschiedenen Belastungsstufen vorgenommen, und zwar sowohl bei vollständig verbolzten als auch bei einfach verbolzten Anschlüssen der Füllungsglieder. Aus den Meßresultaten wurden die Haupt- und Nebenspannungen für alle Teile des Tragwerks berechnet. Neben den elektrischen Widerstands-Meßinstrumenten kamen Huggenberger-Spannungsmesser für ergänzende Untersuchungen zur Anwendung. Aus den Messungen, die unmittelbar an den Enden und in der Mitte der Stäbe vorgenommen wurden, ergab sich die Änderung der Biegemomente über die Stablänge.

Aus dem Vergleich der Versuchsergebnisse mit den Resultaten der üblichen Berechnungsmethoden für solche Träger ergeben sich folgende allgemeine Schlußfolgerungen:

1. Die gute Übereinstimmung zwischen den direkt gemessenen und den unter der Voraussetzung reibungsfreier verbolzter Anschlüsse berechneten Stabkräften bestätigt die Zweckmäßigkeit dieser Berechnungsannahme.

2. Die Richtigkeit der in den Britischen Normen Nr. 449, 1948, festgelegten Methode für die Berechnung von Zugstäben, die nur an einem Winkelschenkel angeschlossen sind, ist ebenfalls experimentell bestätigt.

3. Für die in denselben Normen festgelegten Methoden zur Bestimmung der zulässigen Last eines aus einem einzelnen Winkel bestehenden Druckstabes erscheinen gewisse Änderungen, vor allem im Hinblick auf die einzuführende Stablänge und die Wahl der für das Versagen maßgebenden Achse, wünschenswert.

4. In bezug auf den Einfluß der von den Anschlüssen herrührenden exzentrischen Stabbelastung zeigte sich nur eine schlechte Übereinstimmung zwischen den experimentell ermittelten und den nach der Berechnungsmethode Manderla für sekundäre Spannungen bestimmten Momenten zweiter Ordnung.

5. Die vertikalen Durchbiegungen der Zwischen-Knotenpunkte des Untergurtes stimmten für den Fall des vollständig verbolzten Trägers ziemlich gut mit den nach der Theorie zu erwartenden Werten überein. Bei Anschluß der Füllungsglieder durch Einzelbolzen überstiegen die gemessenen Durchbiegungen diese theoretischen Werte beträchtlich.

Résumé

L'auteur expose des essais de mise en charge qui ont été effectués sur un treillis avec montants verticaux en acier, assemblés par boulonnage et ayant une portée de 10 m. La charge appliquée symétriquement en deux points d'assemblage de la membrure supérieure a été accrue progressivement jusqu'à la mise en carence de la poutre par flambage du montant, sous l'un des points d'application de la charge. La lecture des extensomètres électriques a été effectuée, pour tous les éléments de l'une des moitiés de la poutre, sous différentes valeurs progressives de la charge, aussi bien avec boulonnage intégral des éléments du treillis qu'avec boulonnage simple. Les tensions principales et les tensions secondaires ont été calculées, pour toutes les parties de la poutre, à partir des résultats des mesures. On a utilisé non seulement des instruments de mesure de la résistance électrique, mais aussi des extensomètres Huggenberger qui ont permis d'étendre les investigations. A partir des mesures qui ont été effectuées directement aux extrémités et au milieu des barres, on a pu déterminer la variation des moments fléchissants sur toute leur longueur.

La comparaison des résultats fournis par ces essais avec ceux que donnent les méthodes courantes de calcul pour des poutres de cet ordre a permis de tirer les conclusions générales suivantes:

1. La bonne concordance entre les efforts dans les barres tels qu'ils ont été mesurés directement d'une part et d'autre part, calculés dans l'hypothèse de l'absence de frottement aux points d'assemblage boulonnés, confirme la légitimité de cette hypothèse de calcul.

2. L'exactitude de la méthode exposée dans les ,,Normes Britanniques'', n^0 449 de 1948, pour le calcul des éléments soumis à la traction et assemblés sur une aile de cornière seulement, est également vérifiée expérimentalement.

3. En ce qui concerne les méthodes figurant dans les mêmes normes en vue de la détermination de la charge admissible sur une barre sollicitée à la compression et constituée par une seule cornière, il apparaît opportun de prévoir certaines modifications, principalement au sujet de la longueur de barre à prévoir et du choix de l'axe de rupture.

4. Au sujet de l'influence de la charge excentrique due aux assemblages, on n'a constaté qu'une faible concordance entre les moments de deuxième ordre, déterminés d'une part expérimentalement et d'autre part d'après la méthode de calcul de Manderla.

5. Les fléchissements verticaux des points intermédiaires d'assemblage de la membrure inférieure, dans le cas de la poutre intégralement boulonnée, concordent assez bien avec les valeurs que l'on peut prévoir d'après la théorie. Dans le cas de l'assemblage des éléments de treillis par des boulons isolés, les fléchissements mesurés ont considérablement dépassé les valeurs théoriques.