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RESULTS OF TESTS WITH ROLLED BEAMS REINFORCED BY THE AID OF WELDING.

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I.—Tests on Ribbed Girders.

Bending moments in rolled steel beams are generally calculated according to the equation :

$$\sigma = \frac{M}{W} \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

where σ is the normal stress on the edge of the cross-section of the flange, while W is the section modulus. σ max. is taken to be the permissible stress k, this being $1/n$ of the ultimate stress ($n = 3$ approx.), or the yield point ($n = 2$ approx.). The application of the above formula (1) is justified if it is certain that by increasing the moment M the limit of resistance of the beam will be attained, *i.e.*, failure will occur in the plane of action of the load. This is the case when treating a long and shallow beam, suitably protected against deformation in the horizontal direction (buckling). The section modulus W is here really an indicator of the resistance of the beam. By increasing W the moment M, which the beam can safely bear, is also increased ; yet this rule is of real value up to a certain limit only. When the beam is relatively short and high, normal stresses in a horizontal section through the web at the points of concentrated loading grow more important, and can easily become more dangerous than the normal stresses. Increasing the bending moment M in such a case finally results in the crushing of the flange, directly below the acting load, and of the web, thus causing failure of the beam, consequent on the sudden diminution of the section modulus.

The danger of crushing can be delayed, if not avoided, by means of stiffeners welded to the **I**-beams, similar to the stiffeners in the plate-girders. Those ribs allow one to apply formula (1) even to relatively high and short beams, such as are often seen in practice (*e.g.*, stringers and floor beams in bridges and girders).

Tests were carried out on two series of beams. The first consisted of 16 **I**-beams, Nos. 16, 20, 24 and 30, and the second comprised 6 beams Nos. 32 and 34. The number of a beam refers to its depth in cm. All these beams, of span L equal to 2 m., were submitted to bend tests, a concentrated load being applied to the centre of the

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beam by means of a 200-ton Amsler machine. Three types of beam were investigated (see Fig. 1).

(1) Beams without ribs.
(2) Beams with three ribs, one in the centre and one at each support.
(3) Beams with five ribs, 50 cm. apart.

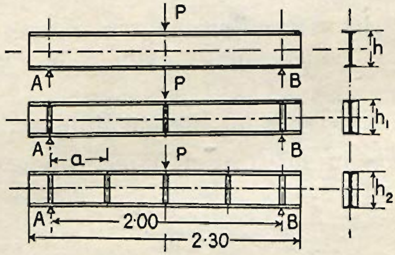


Fig. 1.

Table I.—*The Maximum Load* (*R*) *borne by Each Beam.*

Load in tons.

No. of Beam. (Depth in cm.)	Number of Ribs.		
	Nil.	3	5
16	8·6	7·425	7·6
20	15·4	13·75	15·8
24	22·9	23·85	26·3
30	39·9	48·45	48·3
32	46·0	58·5	59·5
34	51·0	69·5	72·5

Table I. represents the maximal values of the loads borne by the tested beams.

In Table II. the value for R from Table I. is denoted by R_0 for beams without ribs, R_3 for beams with three ribs, and R_5 for beams with five ribs. The figures in the table give the differences between these values in tons, and as a percentage of the subtracted figure.

The values (R_3–R_0) in column 2 show that the deeper the **I**-beam the greater is the increase of its resistance obtained by the addition of three stiffeners (placed under the concentrated load and over the supports). No increase was obtained in **I**-beams Nos. 16 and 20.

TABLE II.—*Comparison of Ribbed and Unribbed Beams.*

No. of Beam. (Depth in cm.)	$R_3 - R_0$	$\frac{R_3 - R_0}{R_0}$	$R_5 - R_3$	$\frac{R_5 - R_3}{R_3}$	$R_5 - R_0$	$\frac{R_5 - R_0}{R_0}$
	Tons.	%	Tons.	%	Tons.	%
16	— 1·175	— 13·7	0·175	2·36	— 1·0	— 11·6
20	— 1·75	— 11·3	2·05	14·9	0·4	2·6
24	0·95	4·15	2·45	10·27	3·4	14·8
30	8·55	21·4	— 0·15	— 0·31	8·4	21·0
32	12·5	27·2	1·0	1·71	13·5	29·4
34	18·5	36·3	3·0	4·6	12·5	42·2

The addition of more ribs between the acting forces generally increases the resistance (except No. 30), but in a much less distinct manner. The last column gives the increase of resistance which is obtained by the use of five ribs (except No. 16). The percentage gain in strength increases with the depth of the beam.

In equation (1) let it be assumed $k = 1200$ kg. per sq. cm. $L = 200$ cm.,

$$\text{now} \quad M = \frac{P\,L}{4} \quad . \quad . \quad . \quad . \quad . \quad (2)$$

hence the maximum safe load, P_b, is given by

$$P_b = \frac{4\ W}{L} = \frac{4 \times 1200}{200} \times W = 24\ W$$

The factor of safety n $\left(n = \dfrac{R}{p}\right)$ or the relation of the greatest load R to the maximum safe load P_b, is given by Table III.

TABLE III.—*Factors of Safety for Ribbed and Unribbed Beams.*

Series.	No. of Beam. (Depth in cm.)	W. Cm.	P_b. Tons.	Factor of Safety. Plain Girder. n_0	Factor of Safety. 3-ribbed Girder. n_3	Factor of Safety. 5-ribbed Girder. n_5
1	16	117	2·81	3·06	2·98	3·05
	20	214	5·14	3·0	2·68	3·08
	24	354	8·50	2·7	2·80	3·10
	30	653	15·67	2·55	3·09	3·08
2	32	782	18·75	2·45	3·12	3·16
	34	923	22·32	2·28	3·12	3·25

From this table it is seen that the addition of stiffeners to the **I**-beams increases the factor of safety, especially in the case of the three stiffeners below the concentrated loads.

By substituting the corresponding load R from Table I. and the section modulus W from Table III. it was possible to work out Table IV., which eliminates to some extent the influence of the variation in depth of the **I**-beams, *i.e.*, the influence of section moduli, and enables the influence of other factors upon the bending of beams to be estimated.

TABLE IV.—*Stresses in the Beams.*

No. of Beam. (Depth in cm.)	Number of Stiffeners.	Stresses in the Beams. Kg. per sq. mm.
	0	36·8
16	3	31·7
	5	32·4
	0	36
20	3	32·2
	5	36·9
	0	32·4
24	3	33·8
	5	37·2
	0	30·6
30	3	37
	5	37
	0	29·4
32	3	37·4
	5	38·0
	0	27·7
34	3	37·7
	5	39·3

Table IV. is illustrated by Figs. 2 to 4. The depth of the beams on the horizontal axes is measured in cm., and the stresses on the vertical axes are measured in kg. per sq. cm. Fig. 2 relates to beams without ribs, Fig. 3 to beams with three ribs, while Fig. 4 refers to beams with five ribs. If the conditions of the tests had been perfect and the material of the beams absolutely uniform, excluding all possibility of lateral buckling, and furthermore, if the formula (1) had been strictly applicable, then the lines would have been ideally horizontal.

It will be noticed that the curve in Fig. 2 droops, while the curve for the stiffened girders (Figs. 3 and 4) slope upwards.

The first result is not unexpected, as all known tests show that the resistance to bending of deep beams is smaller than that of shallow ones.

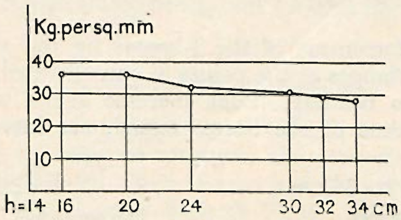


FIG. 2.

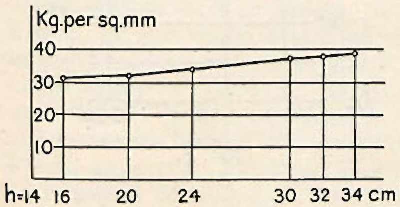


FIG. 3.

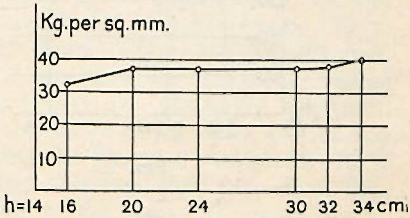


FIG. 4.

The second result signifies not only that this drop in strength can be avoided, but that there is an increase of resistance for deep beams when using ribs. This increase is the greater the deeper the beams.

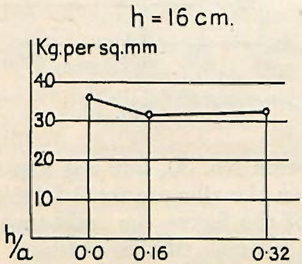


FIG. 5.

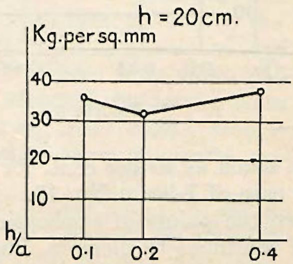


FIG. 6.

In the case of the deepest beams tested (No. 34), it was more than 40 per cent.

The most important cause of this phenomenon is that the ribs prevent the crushing of the chord immediately under the acting load.

Figs. 5–10 represent the increase of R for each group of beams according to h/a (the ratio of the depth of the beams to the distance between the ribs).

The conclusions which follow from these tests may be formulated thus:

(1) The reinforcement of the **I**-beams by the aid of ribs welded to the web and flanges at the points of concentrated loading increases the resistance to bending. Such increase grows with the depth of beam. In the case of the beams tested, the increase of resistance

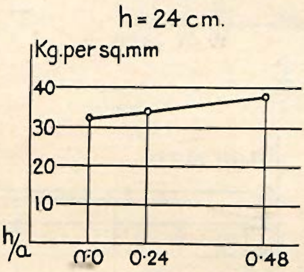


Fig. 7.

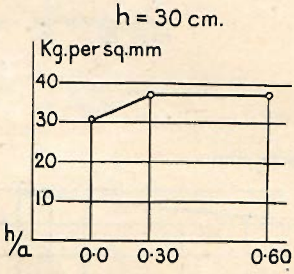


Fig. 8.

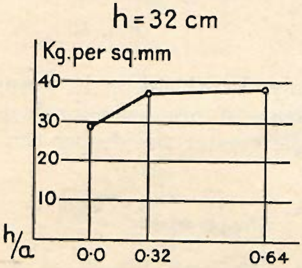


Fig. 9.

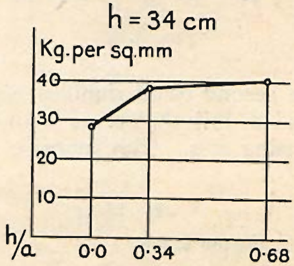


Fig. 10.

was as much as 40 per cent. for **I**-beam No. 30, but was non-existent in the case of **I**-beam No. 16. When the ribs are fixed to the beams between the points of application of the forces the resistance of the beams continues to increase, but to a much smaller extent.

(2) When the depth of the beams increases, the resistance rises more slowly than the section modulus. The allowable stresses obtained from the formula $\sigma = M/W$ decrease with the growing depth of the beams. This formula should not be used for determining the resistance of high beams subjected to the action of concentrated forces, as the beams are not destroyed by direct fracture but by crushing. But if ribs are welded at the points of concentrated loading the danger of crushing is eliminated and the above formula can be used safely.

For shallow beams, which are not subject to crushing, but only to bending, the rôle played by the stiffeners is a minor one, while for very shallow beams the stiffeners are without appreciable effect.

II.—Tests on Plate Girders.

The second series of tests was carried out (together with Mr. Chmielowiec) in order to ascertain the strength of riveted and welded plate girders. All these beams had the same length as the ones used in the first series of tests (2·30 m.), with supports placed at a distance of 2 m. The beams were tested in two groups. The first one comprised nine tests (Nos. 41881–41889), made on five different types of beams. Types *A*, *B* and *C* consist of **I**-beams (NP 30) with plates welded to both flanges. Type *A* (Fig. 11) had plates 140 × 8 mm. on the whole length of the beam. Type *B* (Fig. 12) had, moreover, additional plates 150 × 8 mm. and 600 mm. long welded in the middle of the span. In type *C* (Fig. 13), plates 145 × 16 mm. and 600 mm. long were welded in the centre of the span, one on top and one at the bottom, the beam being also provided with plates 140 × 8 mm. as in type *A*. Type *C* differs therefore from type *B* in this, that instead of having in the middle of the span two plates 8 mm. thick welded to each flange, there is only one plate, but that one twice as thick. The junction of the 8-mm. and 16-mm. plates was made as shown in Fig. 14. The types *D* and *E* consisted of plate girders, with a web 300 × 10 mm., and four angle irons 75 × 75 × 10 mm. attached with rivets 20 mm. in diam., and reinforced over the supports and at the point of application of the load by ribs made of angle iron, 65 × 65 × 10 mm., and rolled plate 65 × 10 mm. and 150 mm. long. In the type *D* there were plates 180 × 10 mm. riveted to the beams (Fig. 15), and in the type *E* plates 140 × 10 mm. were welded on by means of intermittent welds (Fig. 16). Three tests were made with this type of beam, two with plates attached by intermittent welds and one with plates attached by a continuous weld. One test was made with each of the types *B* and *C*, and two tests with each type *D* and *E*.

The second series tested comprised 12 **I**-beams (No. 30) provided with reinforced ribs (as depicted in Fig. 1), and divided into three types ; type *A* (nine beams), type *B* (two beams), and type *C* (one beam). The stiffeners constitute the difference between the former group and those just described. Tests were made with **I**-beams of different depths (numbers), provided with plates and ribs, and without ribs. Considering also the plain **I**-beams (No. 30), and calling them type *AA*, the tests can be divided into three groups :

Group 1, types *A*, *B*, *C*, and *AA* of the first series.
Group 2, ,, *A*, *B*, *C*, and *AA* of the second series.
Group 3, ,, *D* and *E* of the first series.

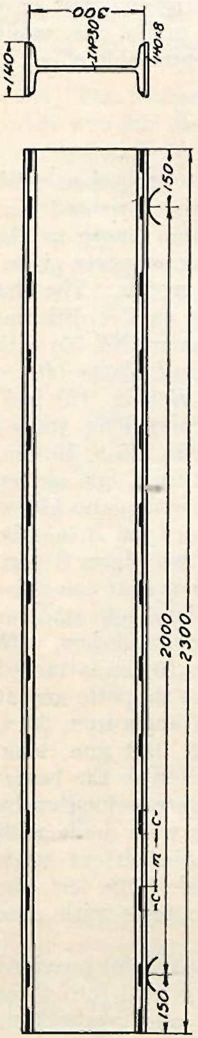
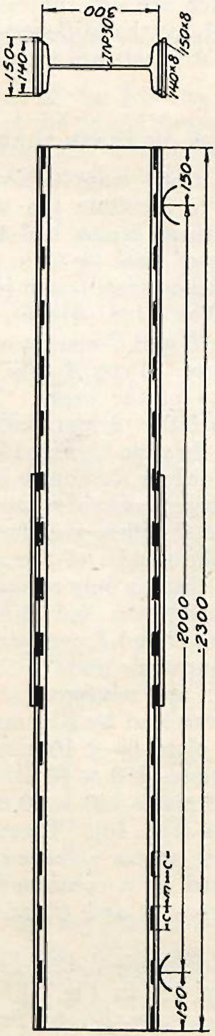


FIG. 11.



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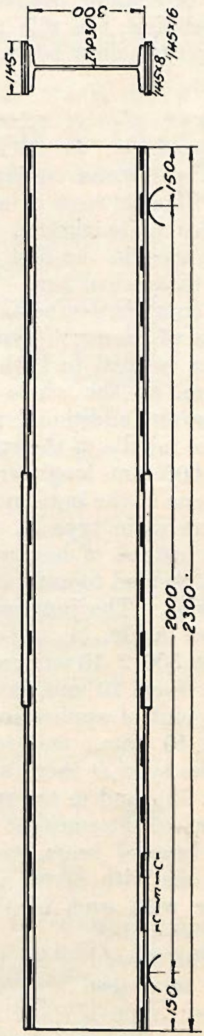


FIG. 13.

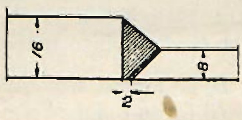
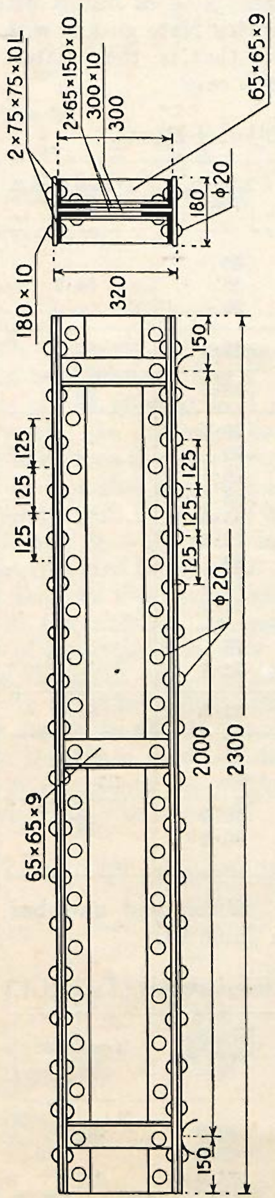


FIG. 14.


2×75×75×10L
2×65×150×10
300×10
300
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180×10
180
ϕ20
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65×65×9
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Fig. 15.

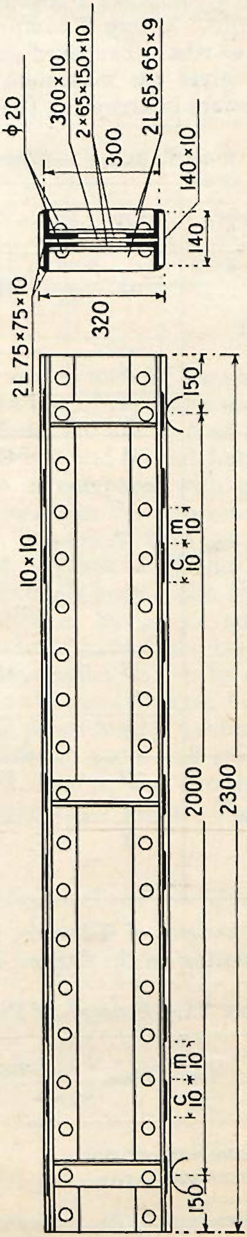
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Fig. 16.

Group 1 comprises **I**-beams type No. 30, provided with plates but without ribs. Group 2 comprises the same kind of beams with the addition of ribs. The third group deals with plate girders with ribs. Table V. gives the maximum stresses R, that is, the greatest load applied before interrupting the test, in each case.

TABLE V.—*Strength of Reinforced Beams.*

Group.	Type.	No.	Load R. Tons.	Average Load R. Tons.
1	*A*	41,881	55	54·7
		2	59	
		3	50	
	B	4	62·5	62·5
	C	5	68·5	68·5
2	*A*	42,162	79·5	71·3
		3	70	
		4	78·5	
		5	63	
		6	65·5	
		7	72·5	
		8	69	
		9	72·5	
		70	71·5	
	B	42,171	79	76·75
		2	74·5	
	C	3	84·5	84·5
3	*D*	41,886	74	74·9
		7	84	
	E	8	69·5	74·9
		9	80·3	

The strength of **I**-beams, type *AA*, ribbed and unribbed, but without plating on the flanges, is given in Table VI.

TABLE VI.—*Strength of Plain and Ribbed Beams (Type AA.)*

Type of Beam.	Yield Stress Q. Tons.	Maximum Stress R. Tons.	Average R. Tons.
Beams without ribs .	30·0	39·9	39·9
Beams with 3 ribs .	38·5	49·9	48·4
	38·0	47·0	
Beams with 5 ribs .	39·5	48·3	

TABLE VII.—*Comparative Strengths of Reinforced Beams.*
Maximum load R, in tons.

Type:	*A.*	*B.*	*C.*	*AA.*
Group. 1	54·7	62·5	68·5	39·9
2	71·3	76·75	84·5	48·4
Difference	16·6	14·25	16·0	8·5
Difference %	30·3	22·8	24·5	21·3
		22·5		

Table VII. gives the average greatest load in tons for the particular types in both groups 1 and 2. The third line of this table shows the increase of the greatest load resulting from the addition of ribs—in other words, the difference between the first and second lines. The fourth line gives the percentage increase as compared with group 1. It can be seen that the addition of ribs increases the greatest load by 30·3 per cent. in type A, 22·8 per cent. in type B, 23·4 per cent. in type C, and 21·3 per cent. in type AA. It will be further noticed that the ribs give the greatest efficiency in the I-beams with one plate, rather than in the beams without a plate or having an additional plate in the middle. As most of the tests have been made with beams of the type A, in view of their importance, it can be assumed that in the given case, the ribs increase the greatest load by 26 per cent. In order to estimate the economy presented by welding plates or ribs and to be able to compare the different types and groups, the specific resistance must be determined; that is, the quotient R/G, where G represents the weight of the particular beam. The figures in Table VIII. were obtained.

TABLE VIII.—*Specific Strengths of Stiffened and Unstiffened Beams.*

Type:	*AA.*	*A.*	*B.*	*C.*
Group. 1	320	330	354	388
2	362	409	414	455
Difference	42	79	60	67
Difference %	13	24	17	17·3

It is seen from the table that ribs increase the specific resistance by from 13 to 24 per cent.

If the different types of beams are compared in regard to their greatest resistance, the following conclusions are arrived at:

(1) The addition of the first plate to a No. 30 I-beam increased its resistance by 37·2 per cent. in the first group and by 47·4 per cent. in the second group.

(2) In passing from type *A* to type *B*, that is, by adding 600 mm. long plates in the middle part of the beam, the increase in resistance will be 14·25 per cent. in the case of beams without ribs, and 7·65 per cent. in the case of beams with ribs.

(3) In passing from type *A* to type C, that is, by replacing the two 8 mm. plates by one 16 mm. plate, an increase in resistance of 25·3 per cent. is obtained if the beams are provided with ribs, and of 18·5 per cent. if they are not.

(4) On replacing type *B* by type *C* by welding to the centre part of the beam a 16-mm. plate 600 mm. in length, instead of two 8-mm. plates of the same length, the resistance is increased by 10 per cent., irrespective of the presence or lack of ribs. This can be accounted for by the greater stiffness of the thicker 16 mm. rolled plate resisting the formation of a wave in the upper flange, which otherwise diminishes the modulus of the section. It must be said that with the lower flange, when there is no wave formation, there is in fact no difference between the types *B* and *C*. A beam without ribs is more influenced by the thickness of the centre plate than a beam provided with ribs, as the latter resist the crushing effect. Otherwise the supporting ribs do not play any important rôle.

Other results obtained in the tests lead to the following conclusions:

The addition of a plate to an I-beam increased its specific resistance by 3·12 per cent. in the first group, and by 13 per cent. in the second group, whereas the addition of further short plates increased it by 7·34 per cent. in the first group and by 1·5 per cent. in the second.

The type *C* is the most efficient. Compared with *AA* it gives an economy of 26 per cent. in both groups. The replacing of type *B* by type *C* gives an economy of 10 per cent.

The influence of ribs is considerable, as was stated in the first section of this paper. They distribute the load over both flanges and through the entire depth of the web, and so prevent the web from undergoing wave deformation where the load occurs. As a proof of their efficiency the following may be quoted: (1) The increase of the greatest load *R*. (2) a distinct breaking of the lower flange in the second and third groups which does not occur in the first group; (3) the bending of the ribs in Nos. 42170 and 42171. However, if the ribs were placed at other places than that of the load, their influence would be much reduced. The ribs should, therefore, be used at the places where the forces are transferred from the beams to the girders, and from the longitudinal beams to the diagonals. However, ribs cannot increase the modulus of the section, nor do they prevent the buckling of the web along the edge which is under load. But in fact such buckling never appeared. The effect of local crushing was prominent in all the tests.

The influence of the length and spacing of the welds was found to be without great importance, although it can be noticed that the resistance diminishes to a certain degree, not very considerable, with the increase of intervals between the welds.

In comparing the I-beams provided with ribs with plate girders the following conclusions are obtained :

(1) In regard to the greatest load R, for the plate girders type D, R is 79 tons ; for plate girders type E, R is 74·9 tons ; for I-beams type A, R averages 71·3 tons. The plate girders are stronger mainly by virtue of the reinforcing action of the vertical arms of the angle iron of the upper flange, which prevents the buckling of the web. The plate girder type D is stronger than the type E, as the plates with which it is provided are wider : 180 × 10 mm. instead of 140 × 10 mm.

Table IX.—*Relative Specific Strengths.*

Group.	Type.	Specific Strength. R/G	Differences.		Differences % on E.
2	AA	362	AA–E	53	17
	A	409	A–E	100	32·4
	B	414	B–E	105	34
	C	455	C–E	146	47
3	D	308			
	E	309			

(2) In regard to the specific resistance the plate girders are less efficient than the I-beams (see Table IX.). The use of the I-beam, with plates or without, in place of a plate girder gives an economy of from 17 to 47 per cent. This economy appears still greater if the cost of the work is considered ; it is much greater with plate girders than with I-beams. In riveted constructions plate girders are often used instead of I-beams, because :

(1) It is difficult to assemble I-beams and transverse girders.

(2) With a given height the plate girder has a greater absolute strength—at least according to calculations.

With welded constructions the joining of beams does not present any difficulty and the absolute strength may be increased by adding plates. In such constructions, therefore, the use of I-beams rather than plate girders is not only possible but desirable. This is one of the great advantages to be gained by welded constructions.

BIBLIOTEKA
POLITECHNIKI WARSZAWSKIEJ
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