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Reference Book
for
Statical calculations.

(Rapid Statics.)

**Force-diagrams for frameworks, tables,
instructions for statical calculations etc. for all classes
of building and engineering.**

Published for the practical use of boards of works,
architects and engineers, by

Francis Ruff,
Civil-Engineer, Frankfort on the Maine.
Membre titulare de la Société des Ingénieurs Civels de France.

With 160 illustrations.

Vol. I.

LONDON:
E. & F. N. Spon Ltd. 57 Haymarket.
NEW YORK:
Spon & Chamberlain, 123 Liberty Street.
1905.

The arrangement of this book — the order of the points of intersection s_0, s_1 etc. on the force diagrams in systematic connection with the text — is protected in Germany by a trade mark.

Printed by Oscar Brandstetter, Leipzig.

Motto:
Speed, Certainty and Ease.

Preface to the German Edition.

The following work is, so far as I am aware, the first attempt to present the applications of statics to the constructions most frequently appearing in technical work, in a comprehensive and easily graspable manner.

As a model, I have taken that excellent work "Lehmanns Hand-Atlanten für Medizin". I started with a clear recognition of the fact, that few practical men find it to their taste to gather the requirements of the moment from complicated and often obscure directions, such as occur in the present works of reference. This process is not only irksome to a degree, but it involves the expenditure of much valuable time.

I have accordingly made it the goal of my endeavours, to place in the hands of Engineers a practical book, which will enable them to carry out any desired statical calculation with ease and rapidity.

To the business man also is the work of great use, especially for estimates, designs, and rapid general supervision.

The compendium of tables at the end of the work has been added on purely practical grounds, and will certainly spare the possessor of the book an infinity of labour.

The "Reference Book" is not designed to teach, but to inform, and to give insight into the employment of force-diagrams in all ordinary cases.

It is to be hoped that the labour has not been in vain.

For my trouble there could be no greater reward than that the "Reference Book of Statical Calculations" should meet with a favourable reception by fellow-workers, fulfilling thus its earnest purpose of furthering individual and collective progress.

FRANKFORT on the Maine, 1. April 1903.

The Author.

Preface to the English Edition.

The universal approbation which the first edition of this book has met with in Germany and France, has convinced me of the fact that my work has not only aroused appreciative interest, but has also satisfied practical requirements.

I have therefore much pleasure in conforming to a widely expressed desire and presenting to my fellow workers in Great Britain and the British Colonies a translation of my work.

I hope that the "Rapid Statics" may prosper and widely extend itself in these lands also.

FRANKFORT on the Maine, 1. July 1905.

The Author.

Contents.

I. Bridge Trusses.

II. Strut Frames.

III. Roof Construction.

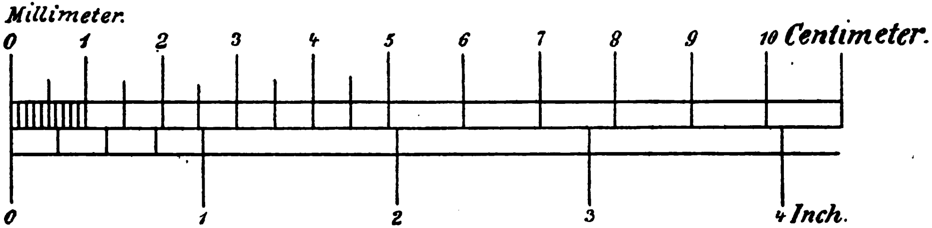
XIV. Pillars.

XV. Weights Tables, etc.

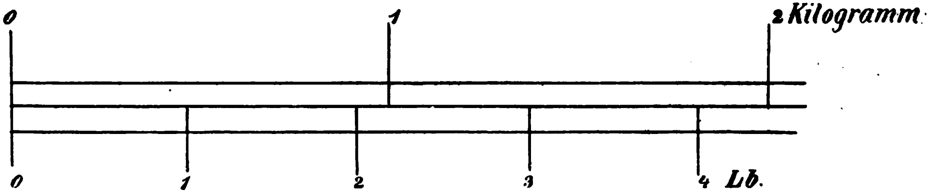
The author will be very glad to receive any corrections, or suggestions for the increase in scope and usefulness of the "Reference Book for Statical Calculations". Communications, which will be most gratefully received, should be addressed to

Francis Ruff
Civil Engineer
Frankfort o./M.

Comparison of Metrical Weights and Measures
to
English Weights and Measures.



5 cm. - 1 kg.

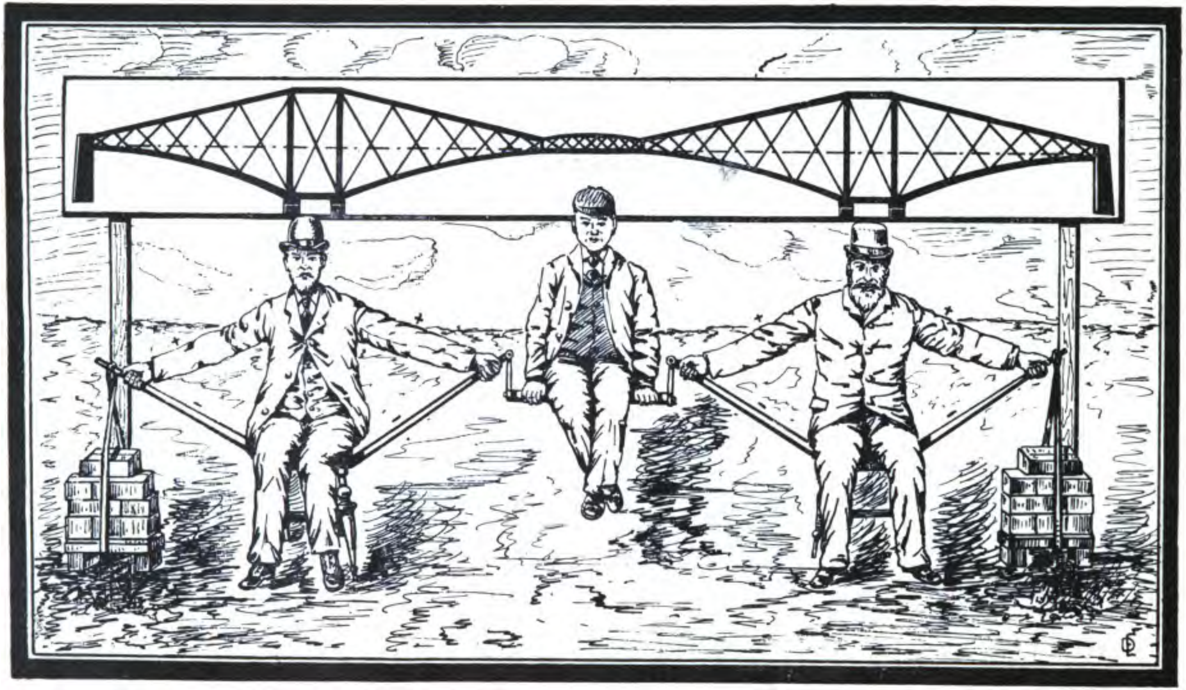


1 metre = 39,37 inch.
1 inch = 0,0254 metre.

1 kg. = 2,2046 Lb.
1 Lb. = 0,4536 kg.

Representation

of the Stresses in the Elements of the Forth Bridge, England.



Engineering News, 11. June 1887 by C. Clarke, C. E.

Hints to Readers.

The Statical Calculation for a bridge is always begun by the Calculation of the Roadway, and then, step by step, the weight of the Roadway platform and the longitudinal and cross beams and of the footways is ascertained before the examination of the chief beams is begun (for which see illustration pages 119 a, b).

In the Statical Calculation for a Roof the external forces acting on the joints are taken as a basis.

These forces are easily found from the supporting level, which is usually shown by the span of the trusses, and from the interval between the trusses.

The supporting capacity of the roof per square meter is to be found on page 121.

The advantage for readers consists in the fact that, in the force diagrams, the points of intersections of the series are designated by s_0, s_1, s_2 etc. **The starting point s_0** is, for the sake of clearness, marked ◎.

Hence it is at once perceived how any desired force-diagram may be easily and quickly constructed.

E. g. **The reader** desires to know what are the stresses in a parallel beam; he sees the required heading noted in the index under p. 66.

The scale of forces is 1 mm = 100 kg.

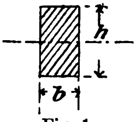
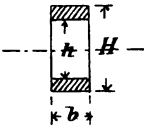
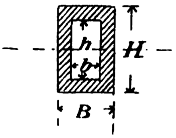
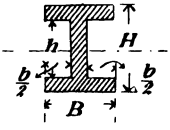
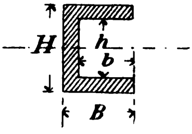
Thus the Diagonal stress $+D_1$ represented by the length $s_1\ s_2$ amounts to $36 \cdot 100 = 3600$ kg.

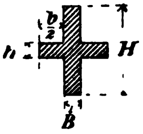
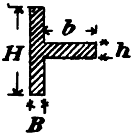
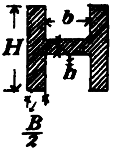
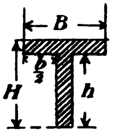
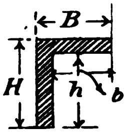
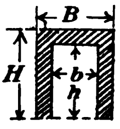
Tensions are denoted by the +, and thrusts by the — sign.

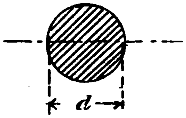
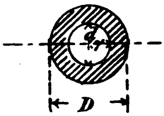
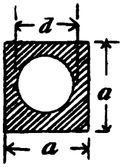
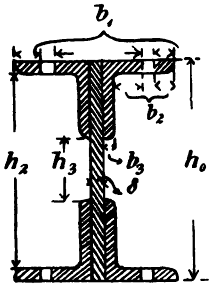
The list at the end of the book enables the reader to obtain at once exact estimates for more advanced calculations.

The Tables for Iron Sections correspond exactly to those in the "Deutsches Normal-Profil-Buch für Walzeisen" latest Edition 1897. JOS. LA RUELLE, Aachen.

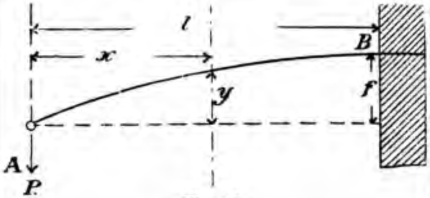
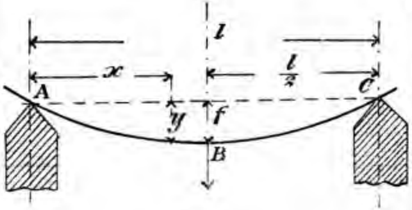
Table of moments of inertia and resistance of the cross sections most in use.

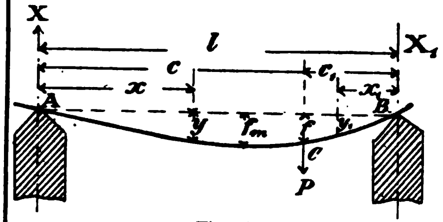
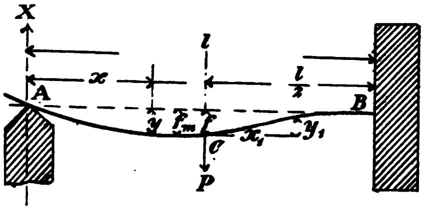
No.	Shape of section	Moment of inertia I	Moment of resistance $W=\frac{I}{e}$
1	 Fig. 1.	$\frac{b\,h^3}{12}$	$\frac{b\,h^2}{6}$
2	 Fig. 2.	$\frac{b}{12}(H^3-h^3)$	$\frac{b}{6\,H}(H^3-h^3)$
3	 Fig. 3.	$\frac{B\,H^3-b\,h^3}{12}$	$\frac{B\,H^3-b\,h^3}{6\,H}$
4	 Fig. 4.		
5	 Fig. 5.		

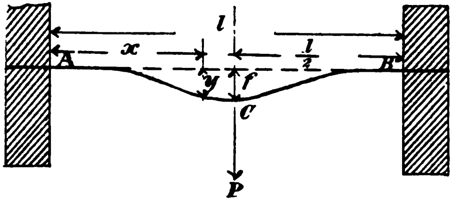
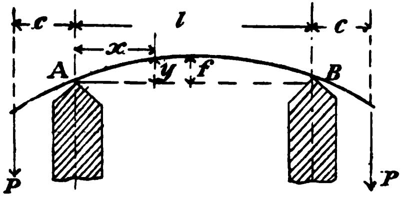
No.	**Shape of section**	**Moment of inertia** I	**Moment of resistance** $W=\frac{I}{e}$
6	 Fig. 6.	$\frac{BH^3+bh^3}{12}$	$\frac{BH^3+bh^3}{6H}$
7	 Fig. 7.		
8	 Fig. 8.		
9	 Fig. 9.	$I=\frac{(BH^2-bh^2)^2-4BHbh(H-h)^2}{6(BH-bh)}$	$W=\frac{(BH^2-bh^2)^2-4BHbh(H-h)^2}{6(BH^2-bh^2)}$
10	 Fig. 10.		
11	 Fig. 11.		

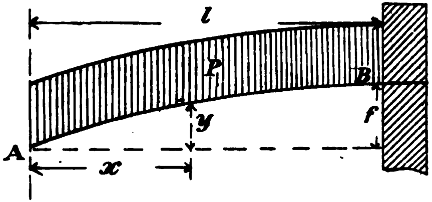
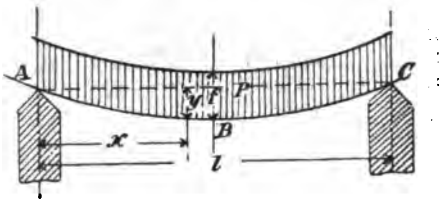
No.	Shape of section	Moment of inertia I	Moment of resistance $W=\frac{I}{e}$
12	 Fig. 12.	$\frac{\pi d^4}{64}$	$\frac{\pi d^3}{32}$
13	 Fig. 13.	$\frac{\pi}{64}(D^4-d^4)$	$\frac{\pi}{32}\cdot\frac{D^4-d^4}{D}$
14	 Fig. 14.	$\frac{1}{12}\left(a^4-\frac{3\pi}{16}d^4\right)$	$\frac{1}{6a}\cdot\left(a^4-\frac{3\pi}{16}d^4\right)$
15	 Fig. 15.	$T=\frac{1}{12}\left(b_1\cdot h_1^3-2b_2h_2^3-2b_3h_3^3\right)$	$\frac{2T}{h_0}$

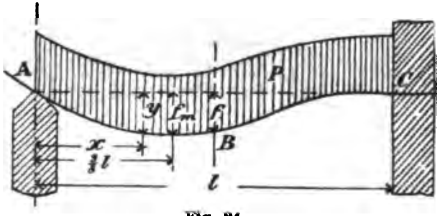
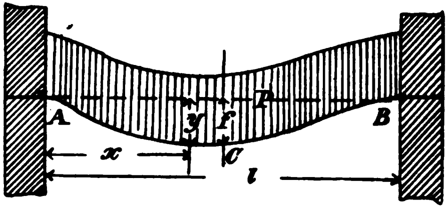
Formula for bending most in use.

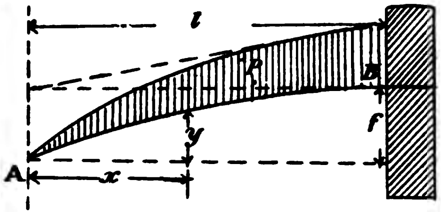
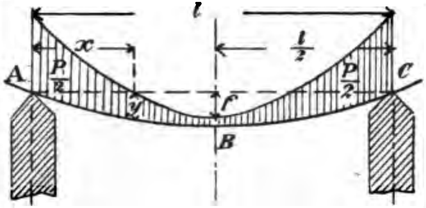
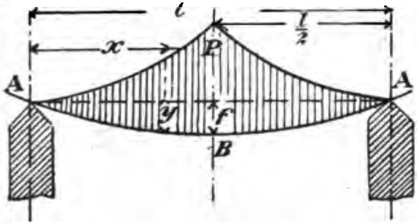
No.	Nature of Strain	Bending Moment M	Deflection f	Remarks
1	 Fig. 16.	$M = P \cdot x$ $M_{\max} = P \cdot l$	$f = \frac{P}{ET} \cdot \frac{l^3}{3}$	Cantilever. Greatest breaking strain at B
2	 Fig. 17.	$M = \frac{P \cdot x}{2}$ $M_{\max} = \frac{P \cdot l}{4}$	$f = \frac{P}{ET} \cdot \frac{l^3}{48}$	Girder freely supported at its ends. Greatest breaking strain in the middle

3	 Fig. 18.	for AC: $M=\frac{Pc_1\cdot x}{l}$ for BC: $M=\frac{Pc\cdot x_1}{l}$ $M_{\max}\frac{Pcc_1}{l}$	$f=\frac{P}{ET}\frac{l^3}{3}\frac{c^2}{l^2}\frac{c_1^2}{l^2}$ $f_{\max}$ for: $x=c\sqrt{\frac{1}{3}+\frac{2c_1}{3c}}$	Greatest breaking strain at C $X=P\frac{c_1}{l}$ $X_1=P\frac{c}{l}$
4	 Fig. 19.	for AC: $M=\frac{5}{16}P\cdot x$ for BC: $M=Pl\left(\frac{5}{32}-\frac{11}{16}\frac{x_1}{l}\right)$ $M_{\max}=\frac{3Pl}{16}$	$f=\frac{P}{ET}\cdot\frac{7\cdot l^3}{768}$ $f_{\max}=\sqrt{\frac{1}{5}}\cdot\frac{P\cdot l^3}{48\cdot E\cdot T}$ for $x=l\sqrt{\frac{1}{5}}$	Girder with one end fixed. Greatest breaking strain at B $X=\frac{5}{16}P$

No.	Nature of strain	Bending Moment M	Deflection f	Remarks
5	 Fig. 20.	$M = \frac{Pl}{2}\left(\frac{x}{l} - \frac{1}{4}\right)$ $M_{\max} = \frac{Pl}{8}$	$f = \frac{P}{ET} \cdot \frac{l^3}{192}$	Girder with both ends fixed. Greatest breaking strain at B, C & A
6	 Fig. 21.	for AB: $M = P \cdot c$	$f = \frac{P}{ET} \cdot \frac{l^3}{8} \frac{c}{l}$	Greatest breaking strain at an unknown position between A & B

7	 Fig. 22.	$M=\frac{Px^2}{2l}$ $M_{\max}=\frac{Pl}{2}$	$f=\frac{P}{ET}\cdot\frac{l^3}{8}$	Cantilever cross section at B
8	 Fig. 23.	$M=\frac{Px}{2}\left(1-\frac{x}{l}\right)$ $M_{\max}=\frac{Pl}{8}$	$f=\frac{P}{ET}\cdot\frac{5l^3}{384}$	Girder supported only at its ends. Greatest breaking strain in the middle

No.	Nature of strain	Bending Moment M	Deflection f	Remarks
9	 Fig. 24.	$M=\frac{Px}{2}\left(\frac{3}{4}-\frac{x}{l}\right)$ $M_{\max}=\frac{Pl}{8}$	$f=\frac{P}{ET}\cdot\frac{l^3}{192}$	Greatest breaking strain at C. Greatest depression at $x=\frac{1}{16}\left(1+\sqrt{33}\right)$ $A=\frac{3}{8}P.$ Bending point at $x=\frac{3}{8}l$
10	 Fig. 25.	$M=\frac{Pl}{2}\left(\frac{1}{6}-\frac{x}{l}+\frac{x^2}{l^2}\right)$ $M_{\max}=\frac{Pl}{12}$	$f=\frac{P}{ET}\cdot\frac{l^3}{384}$	Dangerous diagonal section at A & B. Bending point at $x=\frac{l}{2}\left(1-\sqrt{\frac{1}{3}}\right)$

11	 Fig. 26.	$M = \frac{Px^2}{3l^3}$ $M_{\max} = \frac{Pl}{3}$	$f = \frac{P}{ET} \frac{l^3}{15}$	Cantilever. Greatest breaking strain by B
12	 Fig. 27.	$M = Px\left(\frac{1}{2} - \frac{x}{l} + \frac{2x^2}{3l^2}\right)$ $M_{\max} = \frac{Pl}{12}$	$f = \frac{P}{ET} \cdot \frac{3l^3}{320}$	Greatest breaking strain in the middle
13	 Fig. 28.	$M = Px\left(\frac{1}{2} - \frac{2x^2}{3l^2}\right)$ $M_{\max} = \frac{Pl}{6}$	$f = \frac{P}{ET} \frac{l^3}{60}$	Greatest breaking strain in the middle

Diagonal forces and moments of application for the simple girders (beams).

1. Constant direct load working by single forces.

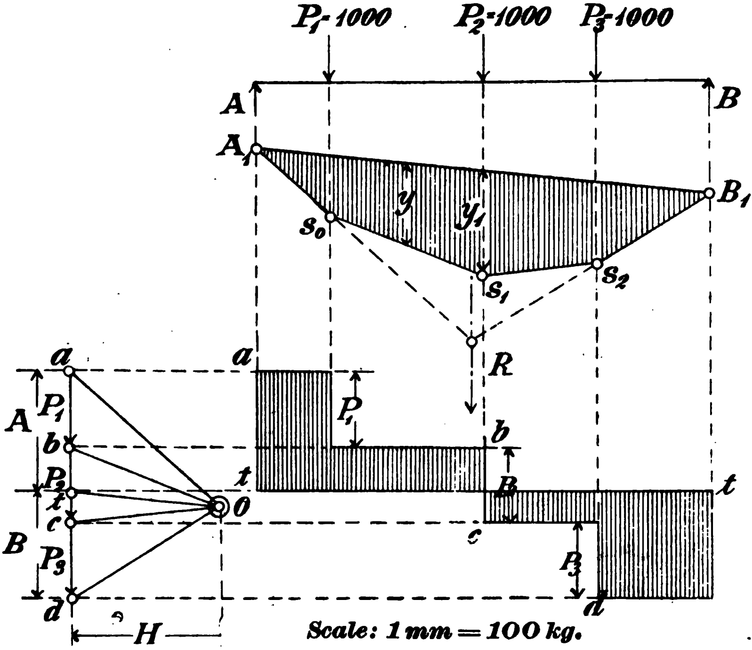


Fig. 29—31.

External forces:

The single forces P_1, P_2 and P_3, of 1000 kg each.

Construction of the polygon of forces and funicular polygon:

Set off the forces P_1, P_2, P_3 in the force diagram, take the Pole O at any point, and construct the funicular polygon s_0, s_1, s_2 with one angle at the point A_1. Close the polygon with the line $A_1 B_1$.

From O in the force diagram draw to its point of intersection t a line parallel to $A_1 B_1$, then the distances at and td will represent the reactions at A and B.

The moment for any desired distance is $M = H \cdot y$.

In this H is the polar distance, and y the perpendicular height of the funicular line for the cross section in question.

The **diagonal forces** are represented by the distances of the points a, b, c, d from t.

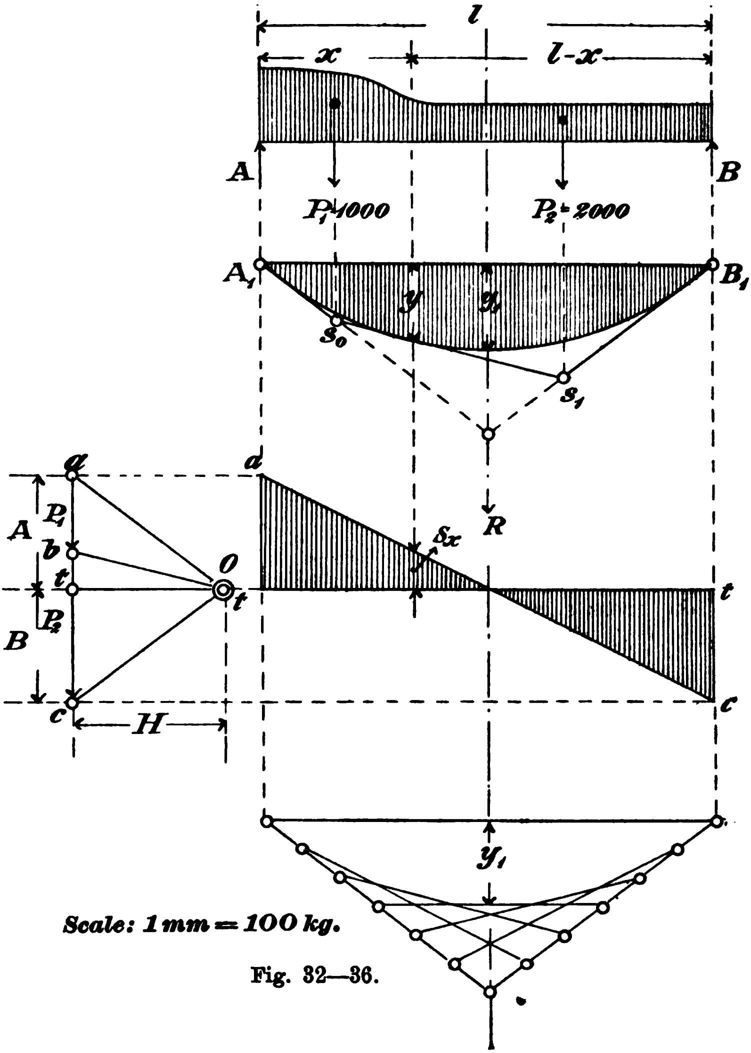
The diagonal force at each point of support is equal to the reaction of the support.

The **moment of resistance** of the girder is:

$$W = \frac{M}{k}.$$

See page 28.

2. Distributed load.



Scale: 1 mm = 100 kg.

Fig. 32—36.

External forces:

An unevenly distributed load (Fig. 32).

Construction of the polygon of forces and funicular polygon:

Suppose the load acts as P_1 and P_2, describe on them the polygon of forces and funicular polygon.

The pressures at A and B are determined by the closing line and its parallel in the diagram of forces.

For an **equally distributed load** $p \cdot l$ we obtain as the funicular line is a parabola; if $H = 1$,
$y_1 = \frac{p \cdot l^2}{8}$.

The **surface of the diagonal force** is bounded by a straight line ac (Fig. 34).

Introductory Note to the force diagrams.

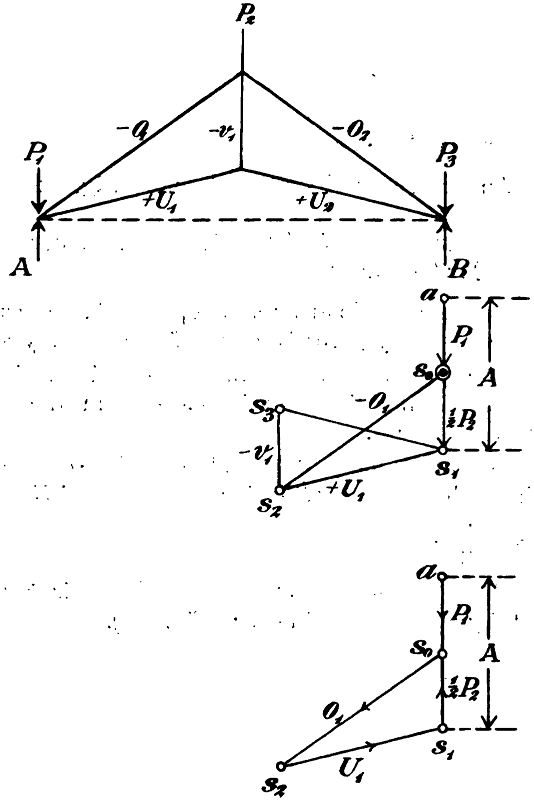


Fig. 37—39.

The **chief principles** are as follows:

1. The external and internal forces at **each joint** must be in equilibrium, and will therefore be geometrically represented by a **closed polygon.**
2. In this polygon the **direction** of the forces, i. e. the **sign, is found** when we consider the same to be always going in **one** sense.

Take as **example** the simple truss pp. **26 and 42.**

At the **joint** A, A P_1 O_1 and U_1 must be in equilibrium, the corresponding polygon being closed, i. e. the polygon $s_1\, a\, s_0\, s_2\, s_1$ ($a\, s_0$ falls on $s_1\, a$ as only vertical forces are taken). In going through this polygon, we must begin with a force of which we know the direction, say A, which acts upwards. Then we must proceed in a **fixed order,** according as the forces act on the joint; first A, then P_1, then O_1 and U_1 (not for ex. A, O_1, P_1, U_1 etc.).

If we therefore continue with A, beginning from s_1 outwards to a, then from a to s_0 (P_1), from s_0 to s_2 (O_1), and from s_2 back to s_1, we obtain the following **directions** (shown by arrows) i. e. the force O_1 acts towards the **joint,** and to it there corresponds a thrust (—); the force U_1 acts **away from the joint,** and is therefore a tension (+) etc.

The Calculations of Dimensions for single Bars etc.

The materials which are admissible and may be safcly used are to be found in the table p. 124.

The **load** P which a body of the cross section F can support with safety either as a tension or a thrust, is

$$P = F \cdot k, \qquad F = \frac{P}{k},$$

where k represents the load admissible for the unit of surface. In order that a body under a bending strain should resist the load sufficiently, the **moment of flexion** is

$$M = W \cdot k \text{ or } W = \frac{M}{k}.$$

The value $W = \frac{T}{e}$ is called the **moment of resistance**, T the equatorial **moment of inertia** of the cross section, e the distance of the furthest thread, which is in a state of tension or compression.

Owing to the smallness of its powers of resistance to tension, **cast iron** should not be used for this purpose. It is however very serviceable for supporting constant loads, and is often used for pedestals, columns etc.

The single members of **frameworks** should only be submitted to either tensile, compressive, or buckling strains. The **breaking point** of a bar under a buckling strain, varies according to the mode in which its ends are fastened.

For **frameworks, girders, columns** etc. the assumption generally holds, that both ends are free, and this corresponds to Euler's formula

$$P = \pi^2 \cdot \frac{E \cdot T}{l^2 \cdot n}$$

in which:

E is the modulus of elasticity of the material (p. 125),

T is the polar moment of inertia of the cross section,

l the length of the bar.

n the factor of security.

For **approximate** calculations we can make use of a well known empirical formula.

For wood $T = 80 \cdot l^2 \cdot P$ (about 10 fold security)

For cast iron $T = 6 \cdot l^2 \cdot P$ (about sextuple security),

For wrought iron $T = 3 \cdot l^2 \cdot P$ (about sextuple security).

It must always be **taken into consideration** whether a bar secure against buckling will suffice for the compression.

The shearing force is $P = k_s \cdot F$ in which $k_s = {}^4/_5$ the greater the value of k is for traction or pressure.

I. Bridge Trusses.

Simple Truss.

(Trussed girder.)

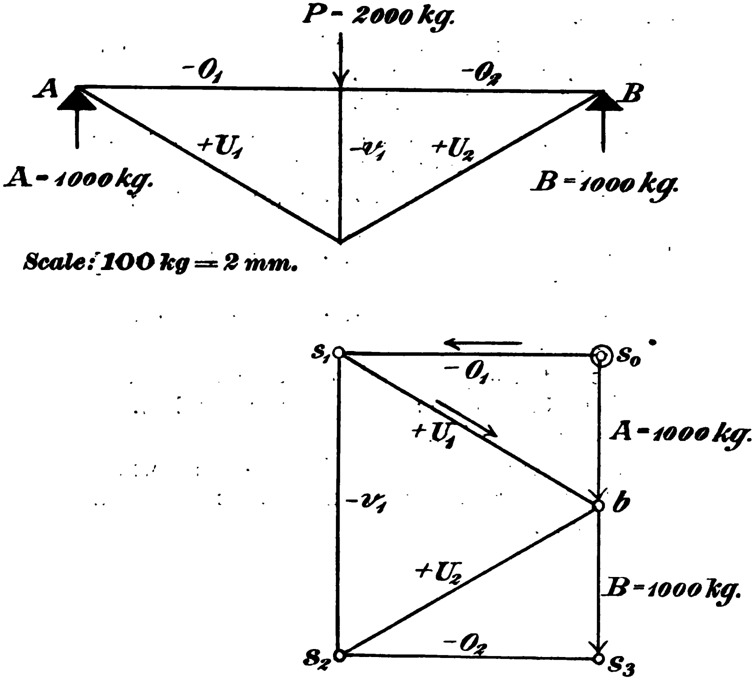


Fig. 40—41.

External forces:

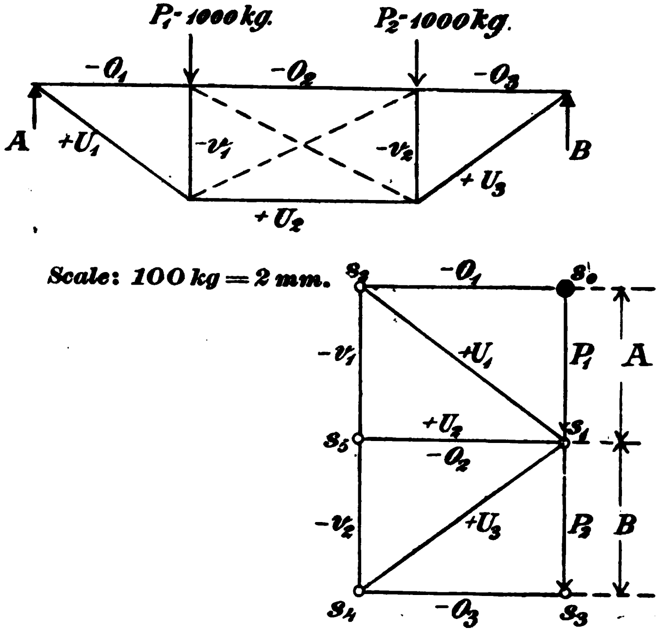
Single force $P = 2000$ kg acting at the middle point of the **upper bar.**

Determination of the stresses:

Set off the pressures A and B each $= \frac{P}{2}$ in the diagram of forces, then draw from the starting point s_0 parallel to the top chord O_1, and from the point b a parallel to the lower chord U, and the point of intersection s will be found. Then draw from b a parallel to U_2 and from s_3, a parallel to O_2, and then the point of intersection s_2 is obtained. The line joining the points of section s_1 and s_2 forms the vertical span V_1.

For the **calculation of the dimensions** of the single bars see p. 28.

Double Truss.



Scale: 100 kg = 2 mm.

Fig. 42—43.

External forces:

Two equal forces, P_1 and P_2 1000 kg each acting on the **upper bar.**

Determination of the stresses:

Set off the forces P_1 and P_2, then draw from s_0 a parallel to O_1 and from s_8 a parallel to U_1, and the point of intersection s_2 is obtained.

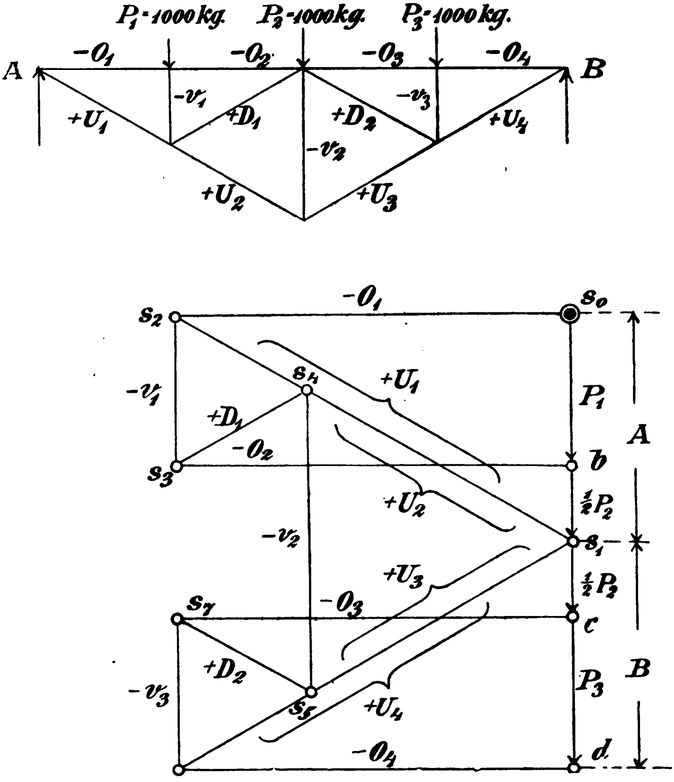
By drawing from s_1 a parallel to V_8 and from s_8 a parallel to O_8, we obtain the point of intersection s_4.

Joining s_2 and s_5 we get the vertical force V_1, V_2 is given by the length s_5 s_4.

Under **equal forces** the diagonals are free from the stress.

For the **calculation of the dimensions** of the single bars see p. 28.

Triple Truss.



Scale: 100 kg = 2 mm.

Fig. 44—45.

External forces:

Three equal forces P_1 P_2 P_3 each 1000 kg acting on the **upper bar.**

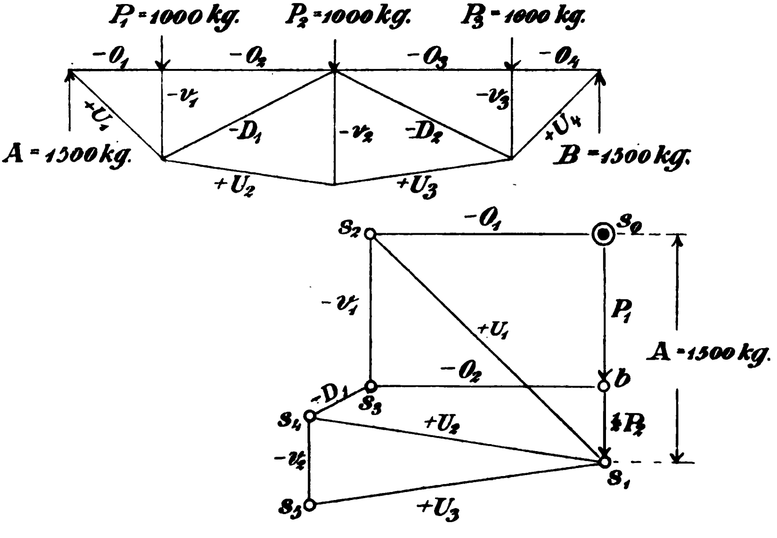
Determination of the stresses.

Set off the forces P_1 P_2 P_3 and the pressures A and B at the point of intersection s_1, draw from s_0 a parallel to O_1, from s_1 a parallel to U_1, and the point of intersection s_2 is obtained. Draw a perpendicular from s_2 to O_2, the point of intersection s_3 is obtained; by drawing from s_3 a parallel to D_1, the point of intersection s_4 (upon U_1) is found.

Continuing, draw from s_1 a parallel to U_4 and from d a parallel to O_4, then from the resulting point of intersection s_6 the parallel V_3, from c a parallel to O_3 and from the point of intersection s_7, a parallel to D_2, the point of intersection s_5 (upon U_4) is found. $s_4 s_5$ gives the stress V_2 which is a pressure.

For the calculation of the dimensions of the single bars see p. 28.

Triple truss, the lower members on each side not being in the same straight line.



Scale: 100 kg = 2 mm.

Fig. 46—47.

External forces:

Three equal forces P_1, P_2, P_3, each 1000 kg acting on the **upper bar.**

Determination of the stresses:

Set off the forces P_1 and $^1/_2 P_2$, draw from s_0 a parallel to O_1, and from s_1 a parallel to U_1, thus obtaining s_2.

The parallels from s_2 to V_1 and b to O_2 intersect at s_3.

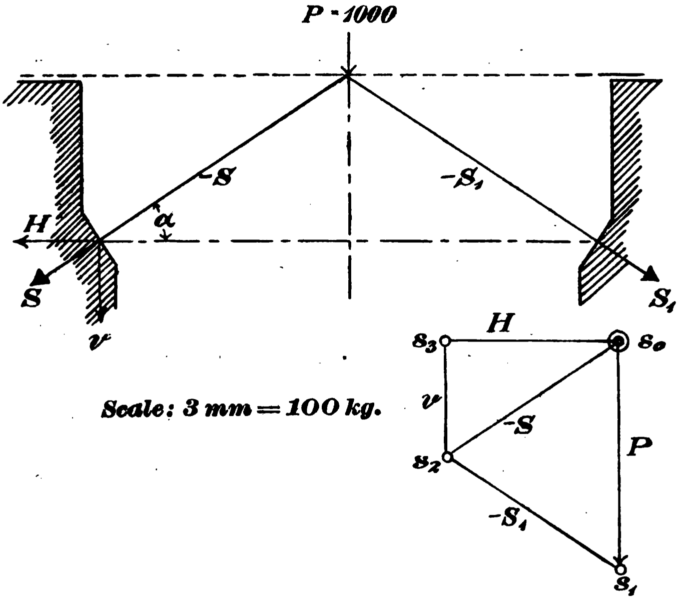
If from s_3 a parallel to D_1, and from s_1 a parallel to U_2 is drawn, the point of intersection s_4 will be found.

The lines from s_4 parallel to V_2, and from s_1 parallel to U_3, intersect at s_5.

For the **calculation of the dimensions** of the single bars see p. 28.

II. Strut frames.

The simple strut frame.



Scale: 3 mm = 100 kg.

Fig. 48—49.

External forces:

A single force $P = 1000$ kg, acting at the **middle point.**

Determination of the stresses:

Set off the force P in the force diagram, draw from s_0 a parallel to S, and from s_1' a parallel to S_1, and the point of intersection s_2 is obtained.

If we draw from s_0 a horizontal and from s_2 a vertical line, the point of intersection s_3 is obtained.

The straight lines $s_0\ s_3$ and $s_2\ s_3$ represent respectively the horizontal force H and vertical force V.

The thrust in the strut is:

$$S = \frac{P}{2 \cdot \sin \alpha}.$$

For the **calculation of the dimensions** of the single bar forces see p. 28.

The double strut frame.

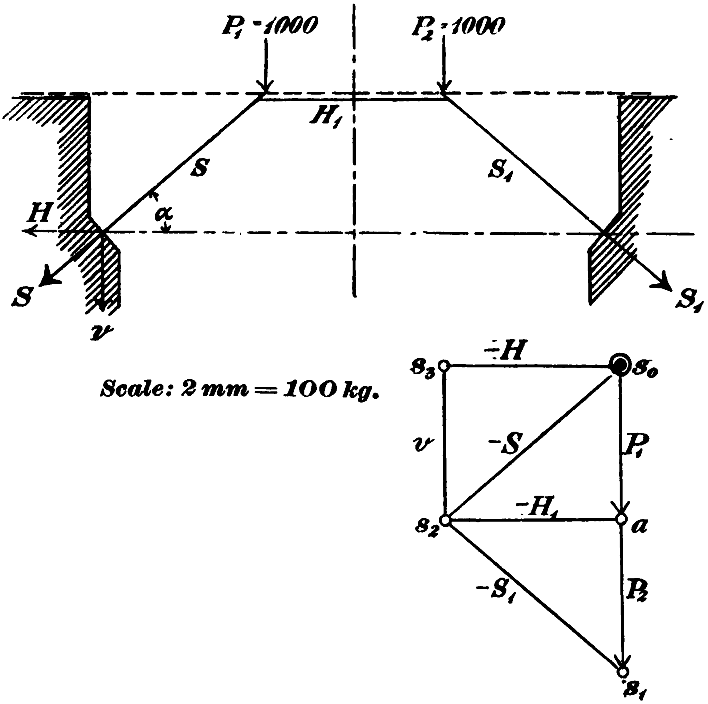


Fig. 50—51.

External forces:

Single force $P_1 = 1000$ kg, $P_2 = 1000$ kg.

Determination of the stresses:

Set off the forces P_1 and P_2 in the diagram of forces, draw from s_0 a parallel to S, and from s_1 a parallel to S_1, then the point of intersection s_2 will be obtained.

A straight line from s_2 through a gives us the thrust H_1.

If we draw from s_0 a horizontal and from s_2 a vertical line, we obtain the point of intersection s_3.

The straigth lines $s_0\ s_3$ and $s_2\ s_3$ represent the horizontal and vertical forces H and V.

The **thrust** in the **struts** and **straining beam** is

$$S = \frac{P_1}{\sin a} \text{ et } H_1 = \frac{P_1}{\operatorname{tg} a}.$$

For the **calculation of the dimensions** of the single bars see p. 28.

III. Roof Construction.

Roof Trusses with the principal rafters unbraced.

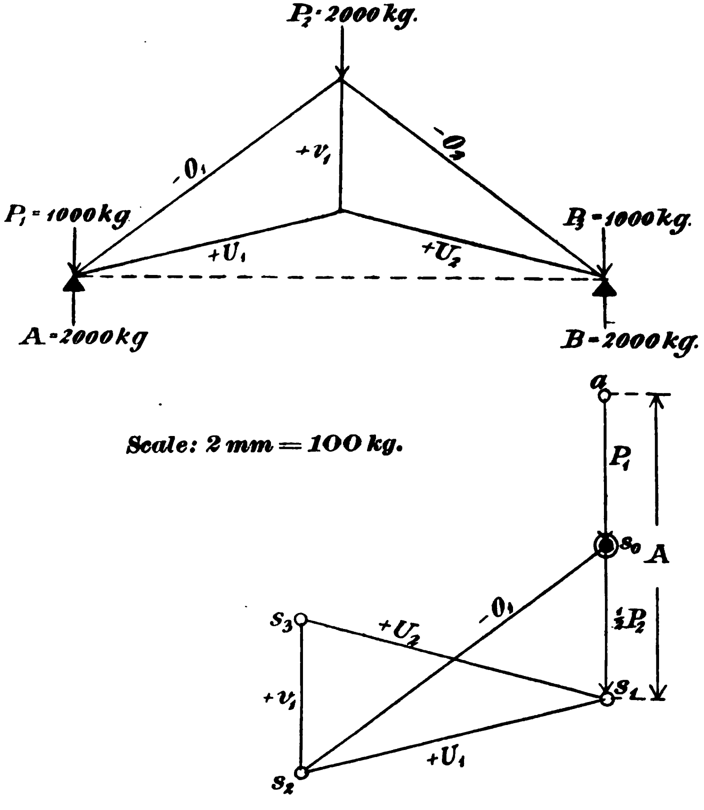


Fig. 52—53.

External forces:

$P_1 = 1000$ kg, $P_2 = 2000$ kg, $P_3 = 1000$ kg acting on the **upper bar.**

Determination of the stresses:

Set off the forces P_1 and $^1/_2 P_2$ in the force diagram, draw from s_0 a parallel to O_1, and from s_1 a parallel to U_1, thus obtaining the point of intersection s_2.

If we now draw a parallel from s_2 to V_1, and from s_1 to U_2, we obtain the point of intersection s_3.

Joining s_2 and s_3 we get the **vertical force** $+V_1$.

For the calculation of the dimensions of the single bars see p. 28.

Trusses with the principal rafters unbraced,

Wind pressure being taken into account.

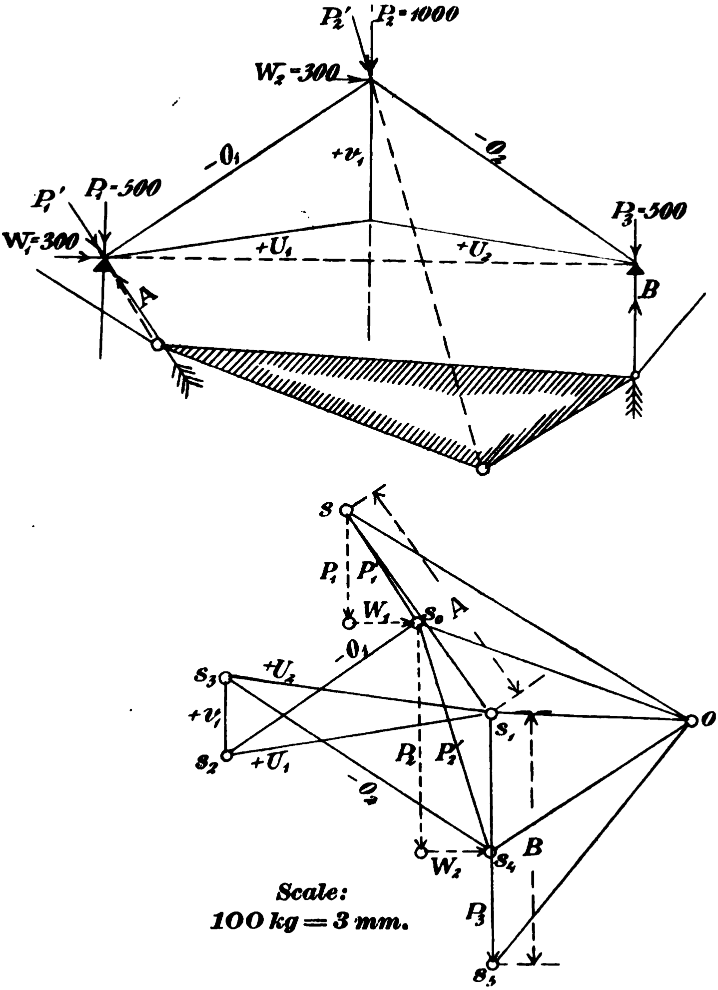


Fig. 54—55.

External forces:

Due to their **dead-weight** P_1 and P_3 are each of 500 kg, $P_2 = 1000$ kg; due to the **wind pressure** W_1 and W_2 are each of 300 kg.

The **wind** acts on **one** side only (the left). The forces on the left side are oblique, (P'_1 and P'_2 are the results of the deadweight P_1 and P_2 and the windpressure W_1 and W_2).

Determination of the stresses:

Set off the forces P_1 and W_1 in the diagram of forces proceeding from the point s; we obtain thus the resulting force P'_1 in direction and magnitude. Then set off the forces P_2 and W_2 from s_0, and from s_4 P_3. From the points s, s_0, s_1, s_4 and s_5 we draw radii to **any pole** O, thus determining the **reactions** A and B by means of the funicular polygon. Having found A and B, we proceed as in the case of **vertical forces only** (see Fig. 52, 53).

For the **calculation of the dimensions** of the single bars see p. 28.

Sheds.

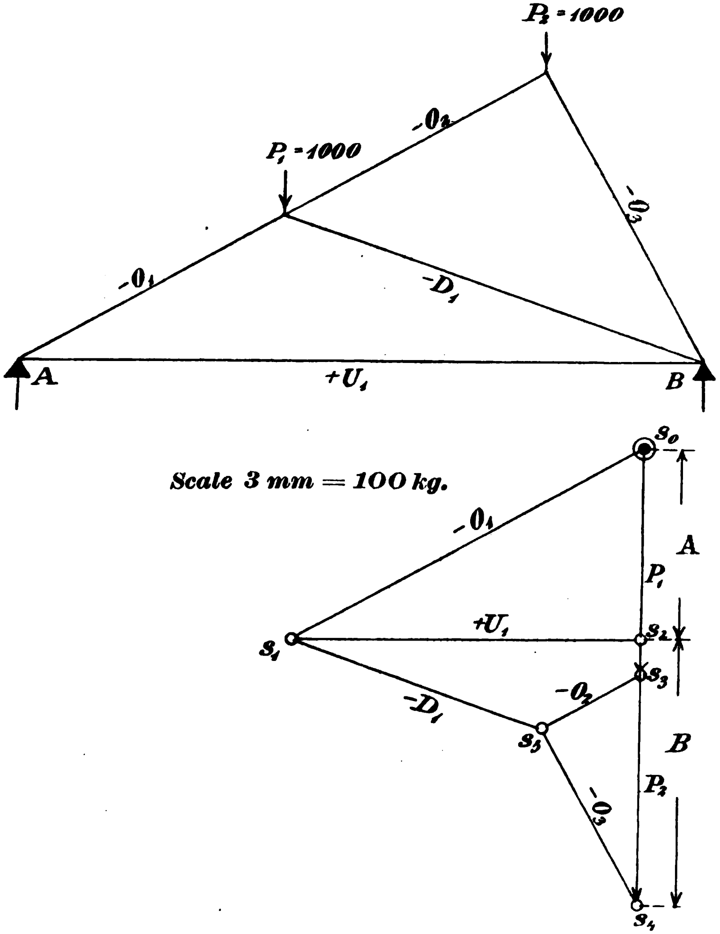


Fig. 56—57.

External forces:

The single forces P_1 and P_2 1000 kg each, acting on the **upper bar.**

Determination of the stresses:

Set off the forces P_1 and P_2 in the force diagram, draw from s_0 a parallel to O_1, from s_3 a parallel to O_2, and from s_4 a parallel to O_3, obtaining the point of intersection s_5; from s_5 the parallel to D_1 gives us s_1, and from s_1 the parallel U_1 gives us s_2. The lengths $s_0\,s_2$ and $s_2\,s_4$ are the two **reactions** A and B.

For the **calculation of the dimensions** of the single bars see p. 28.

Trusses. Simple System "Polonçeau".

(length of span about 15 m.)

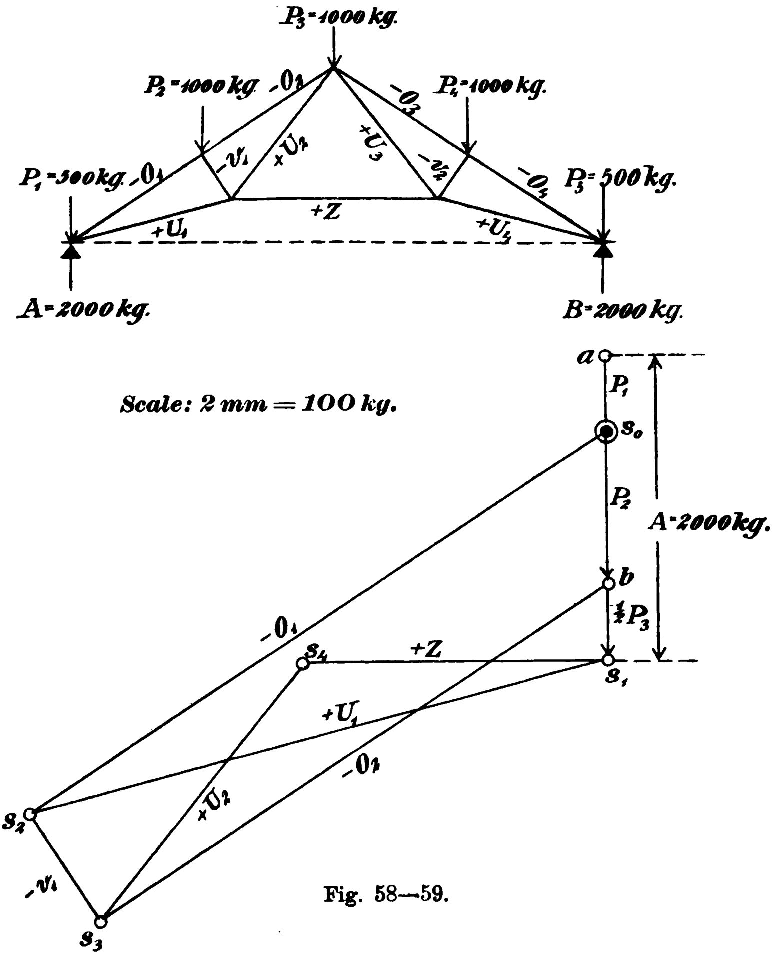


Fig. 58—59.

External forces:

Three equal forces P_2, P_3 and P_4 1000 kg each, P_1 and P_5 each, 500 kg acting on the **upper bar.**

Determinaton of the stresses:

Set off the forces P_1, P_2 and $^1/_2P_3$, draw from s_0 a parallel to O_1 and from s_1 a parallel to U_1, thus obtaining the point of intersection s_2.

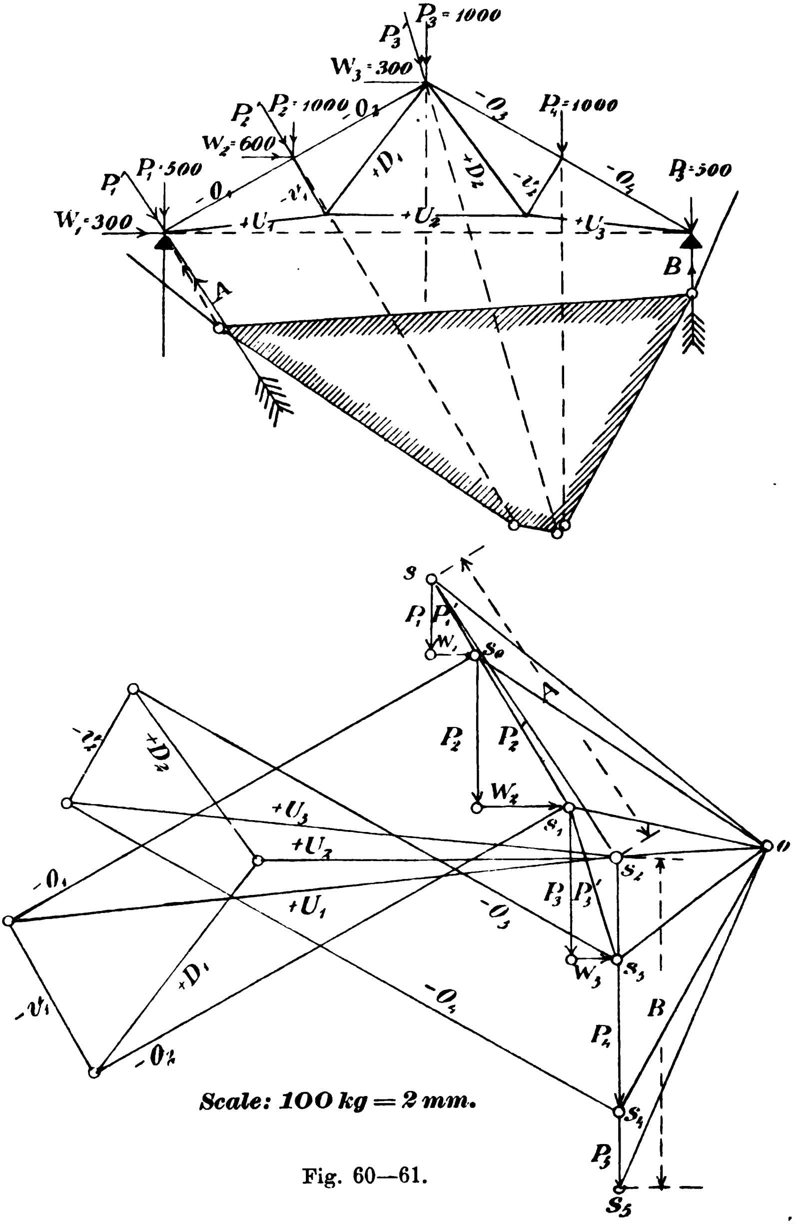
If we now draw from b a parallel to O_2, and from s_2 a parallel to V_1, we get the point of intersection s_3.

By drawing from s_1 and s_3 parallels to Z and U_2 respectively, we obtain the point of intersection s_4.

For the **calculation of the dimensions** of the single forces see p. 28.

Trusses. Simple System Polonceau

as on p. 48, but taking account of wind pressure.



Scale: 100 kg = 2 mm.

Fig. 60—61.

External forces:

P_1 and P_5 500 kg each by their **deadweight,** P_2, P_3 and P_4 each 1000 kg; W_1 and W_3 each 300 kg due to the **wind pressure,** $W_2 = 600$ kg.

The wind acts on **one** side only (the left). Thus the forces on the left are oblique.

(P'_1, P'_2 and P'_3 are the resultants of the deadweight of P_1, P_2 and P_3 and of the wind pressure W_1, W_2, W_3).

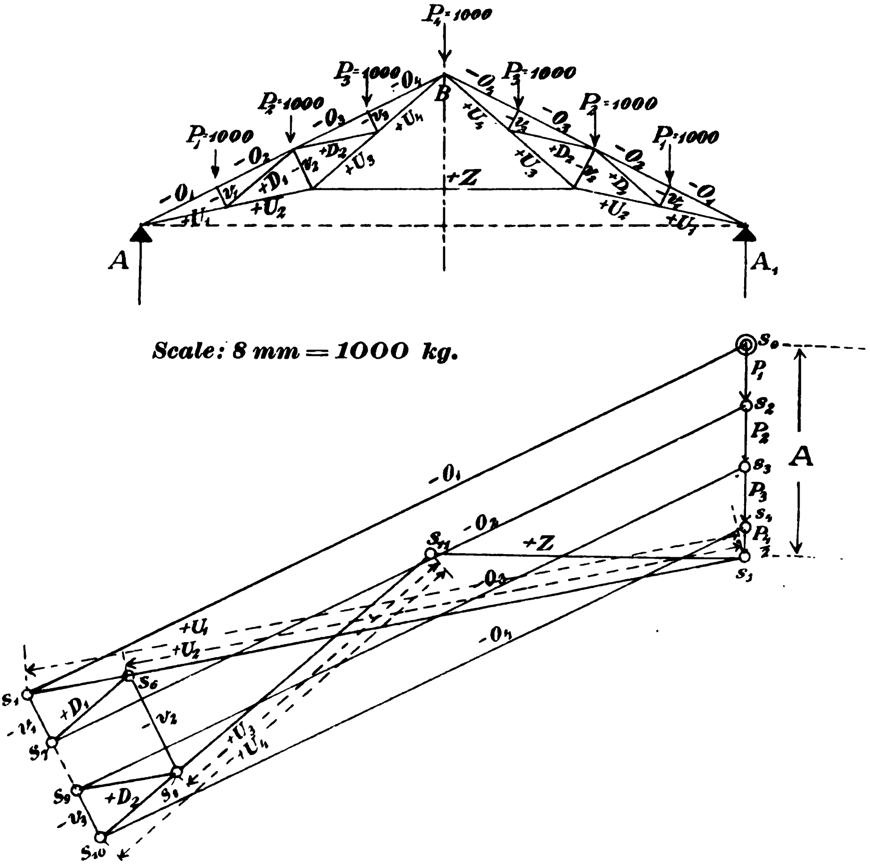
Determination of the stresses:

The method of calculation is the same as that of the **Trusses with strutless principal rafters** (see Fig. 54—55, p. 44).

For the **calculation of the dimensions** of the single bars see p. 28.

Double System Polonceau

width of span about 30 metres.



Scale: 8 mm = 1000 kg.

Fig. 62—63.

External forces:

The **single forces** P_1 to P_8 1000 kg each, $\frac{P_4}{2} = 500$ kg, acting on each of the **upper bars.**

Determination of the stresses:

Set off the forces P_1 to P_8 and $\frac{P_4}{2}$ in the force diagram, draw from s_0 a parallel to O_1, and from s_5 a parallel to U_1, thus obtaining the point of intersection s_1.

By drawing from s_1 a parallel to V_1 and from s_2 to O_2, we obtain the point of intersection s_7. We draw from s_7 a parallel to D_1, to meet U_1, and obtain the point of intersection s_6. From s_3 and s_4 we draw parallels to O_3 and O_4 to meet the line s_1 s_7 produced; this gives us the points of intersection s_9 and s_{10}.

Lines from s_{10} and s_5 parallels to U_3 and Z respectively intersect in s_{11}. From s_9 the parallel to D_2 gives us s_8, and by drawing from s_6 to s_8 a parallel to V_2 we obtain the stress V_2.

For the **calculation of the dimensions** of the single bars see p. 28.

English Roofing.

Width of span about 20 m.

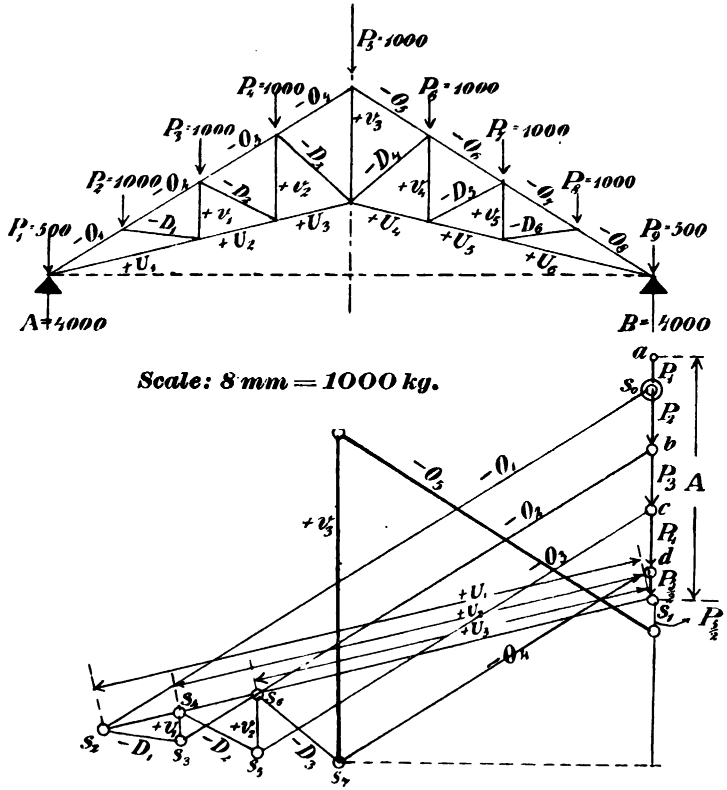


Fig. 64—65.

External forces:

The single forces P_1 and P_9 500 kg each, P_2 to P_8 1000 kg each, acting on the **upper bar.**

Determination of the stresses:

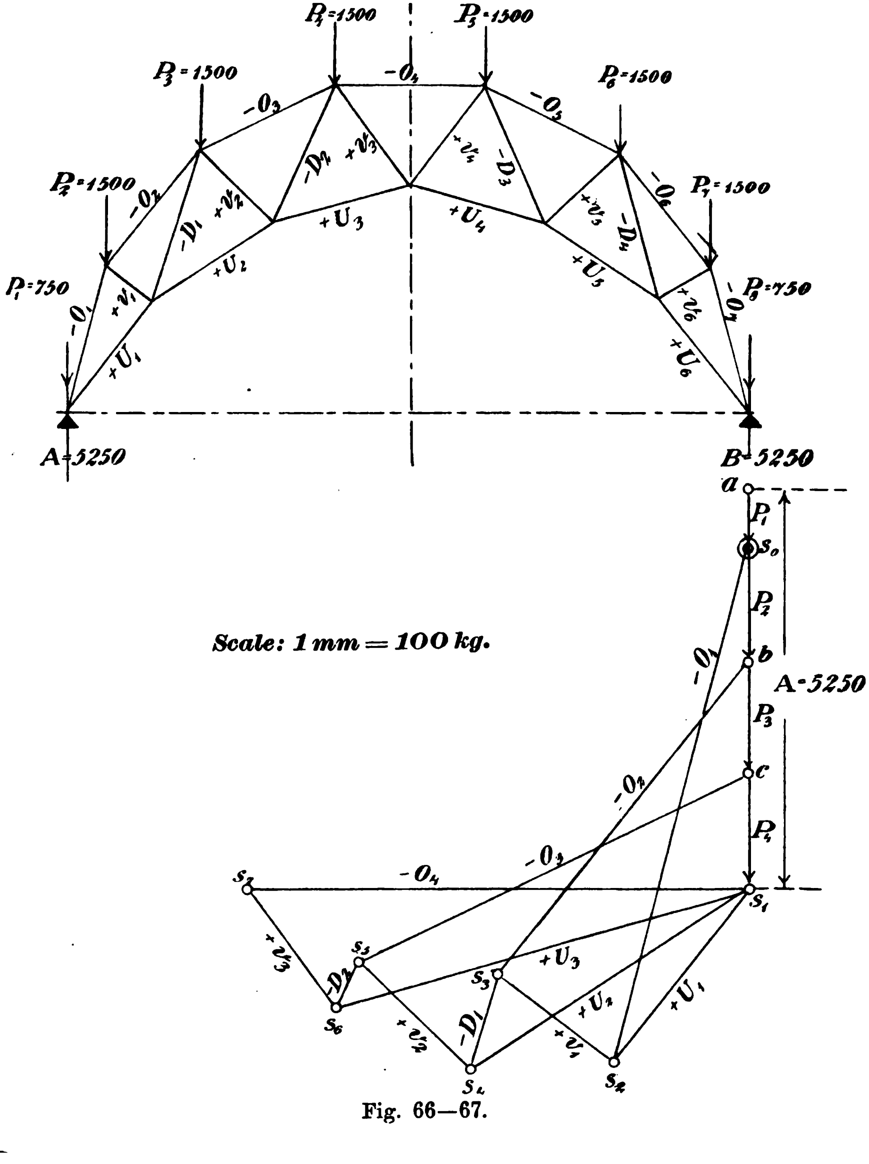
Set off the forces P_1, P_2, P_8, P_4 and $\frac{P_5}{2}$ in the force diagram; drawing from s_0 a parallel to O_1 and from s_1 a parallel to the lower bar, the point of intersection s_2 is obtained.

The parallels from s_2 to D_1 and from b to O_2 give us the point of intersection s_8.

By now drawing a parallel from s_3 to V_1, we obtain s_4 upon U_1. Proceeding in this manner the polygon is completed.

For the **calculation of the dimensions** of the single bars see p. 28.

Crossbeams (with any arch form).



Scale: 1 mm = 100 kg.

Fig. 66—67.

External forces:

$P_1 = 750$ kg, $P_8 = 750$ kg, P_2 to P_7 1500 kg, each acting on the **upper bar.**

Determination of the stresses:

Set off the forces P_1, P_2, P_3 and P_4 in the force diagram; drawing from the starting point s_0 a parallel to O_1, and from s_1 a parallel line to U_1, the point of intersection s_2 is obtained.

If we now draw from s_2 a parallel to V_1, and from b a parallel to O_2, we obtain the point of intersection s_3.

Proceeding in this manner the polygon of forces closes of itself.

For ex. the length s_1 s_7 represents the **thrust in the bar** O_4.

For the **calculation of the dimensions** of the single bars see p. 28.

Domes.

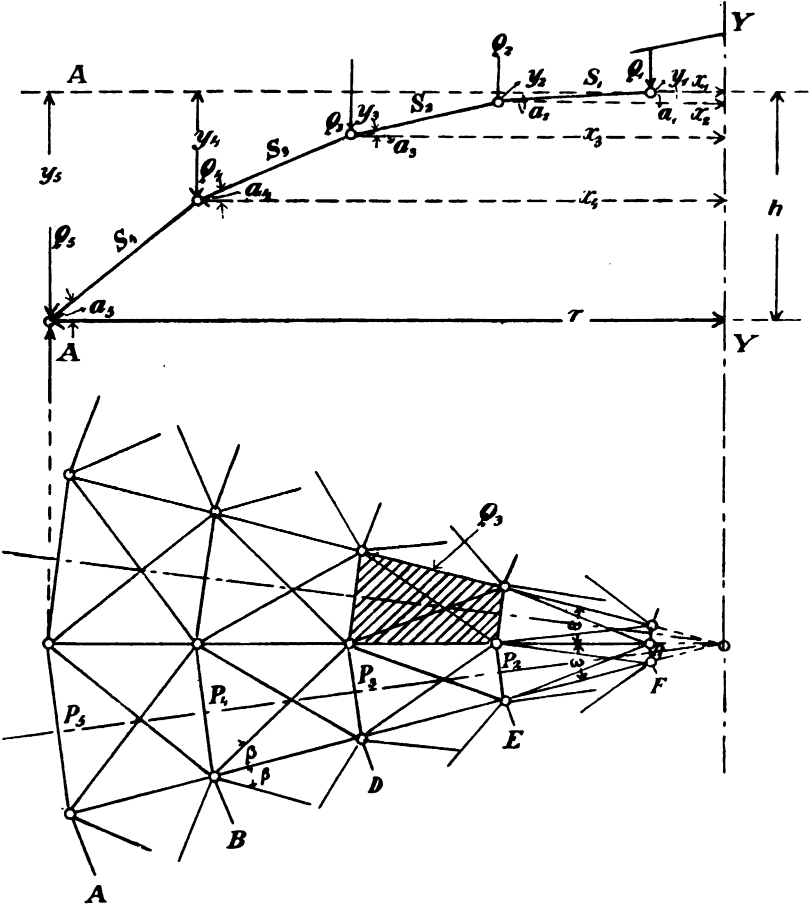


Fig. 68—69.

In this figure Q_1, Q_2 etc. represent the weights of the zones, for ex Q_3 intercalates between two fully loaded rafters.

For domes with a minimum of material we usually choose for the upper curve limit of the rafters a **cubic parabola** according to the formula:

$$y = \frac{h \cdot x^3}{r^3}; \quad r = \text{the radius of the dome}$$

taking the summit as the origin, and

$$y = \frac{h(r^3 - x^3)}{r^3}$$

if the origin of coordinates be taken at depth h below the summit.

h the height of the parabola is usually taken as $^1/_6$ to $^1/_8$ of the spanwidth $2\,r$.

Determination of the stresses:

The **rafter pressure** is resolved as follows:

$$S_1 = \frac{Q_1}{\sin \alpha_2} \quad S_2 = \frac{Q_1 + Q_2}{\sin \alpha_3} \quad S_3 = \frac{Q_1 + Q_2 + Q_3}{\sin \alpha_4}$$

$$S_4 = \frac{Q_1 + Q_2 + Q_3 + Q_4}{\sin \alpha_5}.$$

A load Q_5 on the curve of the wall A has no influence on the members, these being directly supported by the wall.

The **spans of the curve** $\boldsymbol{P}$ are obtained from the general equation:

$$S \cos \alpha = 2\,P \sin \frac{w}{2}$$

in which $w = 2\frac{\pi}{n}$; n = number of the rafters.

According to this the **curve of the wall** is pulled by a force:

$$P_5 = +\frac{S_4 \cos \alpha_5}{2 \sin \frac{w}{2}}.$$

The **lantern ring** *F* is pressed upon by:

$$P_1 = \frac{S_1 \cos \alpha_2}{2 \sin \frac{w}{2}}.$$

Further we obtain:

$$P_2' = +\frac{S_1 \cos \alpha_2}{2 \sin \frac{w}{2}} \text{ (traction)} \quad P_2'' = -\frac{S_2 \cos \alpha_3}{2 \sin \frac{w}{2}} \text{ (pressure).}$$

These last two give us the resultant stress:

$$P_2 = \frac{S_1 \cos \alpha_2 - S_2 \cos \alpha_3}{2 \sin \frac{w}{2}} \text{ together.}$$

For $P_3' = +\frac{S_2 \cos \alpha_3}{2 \sin \frac{w}{2}}$ and $P_3'' = -\frac{S_3 \cos \alpha_4}{2 \sin \frac{w}{2}}$

we obtain $P_3 = \frac{S_2 \cos \alpha_3 - S_3 \cos \alpha_4}{2 \sin \frac{w}{2}}$;

then for

$$P_4' = +\frac{S_3 \cos \alpha_4}{2 \sin \frac{w}{2}} \text{ and } P_4'' = -\frac{S_4 \cos \alpha_5}{2 \sin \frac{w}{2}}$$

$$P_4 = \frac{S_3 \cos \alpha_4 - S_4 \cos \alpha_5}{2 \sin \frac{w}{2}}.$$

An exact calculation of the stresses in the **diagonals** may be usually dispensed with; domes with a span width of up to 45 m are considered strong enough if the cross section amounts to 3 sq. cm.

Owing to the arrangement of the purlins and roof covering the danger of a displacement is avoided; perhaps if the load of the dome is arranged too much on one side a few slight slips might occur, but that is all.

If we wish to determine the **diagonal stresses,** we must notice that in every pair of rafters to which a diagonal is fixed the one has S_{max}, and the other S_{min}.

If we assume that the difference $S_{max} - S_{min}$ is entirely taken by a diagonal, and if we represent the angle of the latter with the rafters by β, the greatest tension is:

$$D = \frac{S_{max} - S_{min}}{\cos \beta}.$$

In the case of the greatest possible strain, the following points may be mentioned:

1. When the dome is fully loaded, the thrusts in the bars are at a maximum.
2. The **maximum tension or pressure** of a circle occurs when the part of the dome situated inside the same is fully loaded, but the circle and its zone unloaded.
3. The maximum tension in the diagonals occurs when the **half** of the dome on one side of the diameter passing through the middle of the diagonals is fully loaded, and the other half unloaded.

For the **calculation of the dimensions** of the single bars see p. 28.

IV. Cantilevers.

Projecting Roof.

Free length about 6 m.

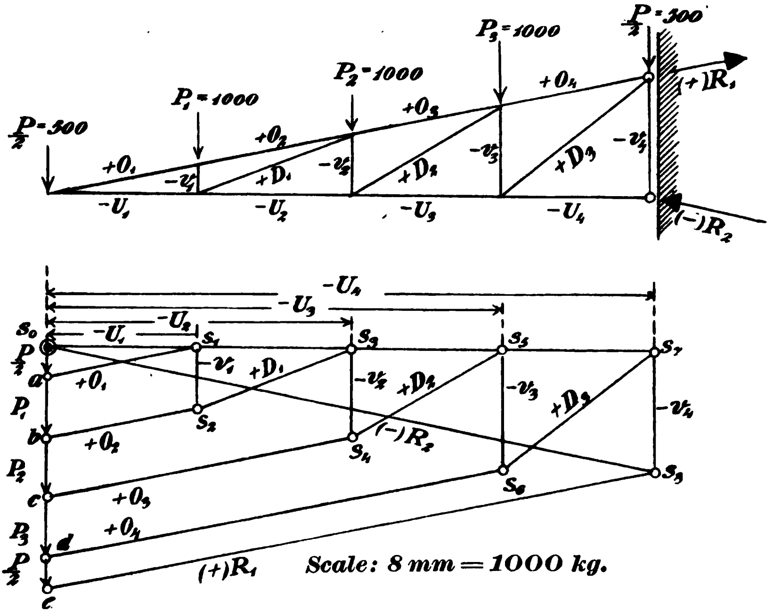


Fig. 70—71.

External forces:

The **single forces** $\frac{P}{2} = 500$ kg, P_1 to P_8 1000 kg each, $\frac{P}{2} = 500$ kg acting on the **upper bar.**

Determination of the stresses:

Set off the forces $\frac{P}{2}$, P_1 to P_8 and $\frac{P}{2}$ in the force diagram, draw from s_0 a parallel to U_1 and from a a parallel to O_1, and the point of intersection s_1 is obtained; from s_1 draw a parallel to V_1, and from b a parallel to O_2, and we obtain s_2. The parallel from s_2 to D_1 gives us s_3. Proceed in this manner, until the polygon is closed.

The lengths s_8 to e and s_8 to s_0 represent the **reactions** R_1 and R_2.

For the **calculation of the dimensions** of the single bar forces see p. 28.

Projecting Roof with tie rod.

Free length about 8 m.

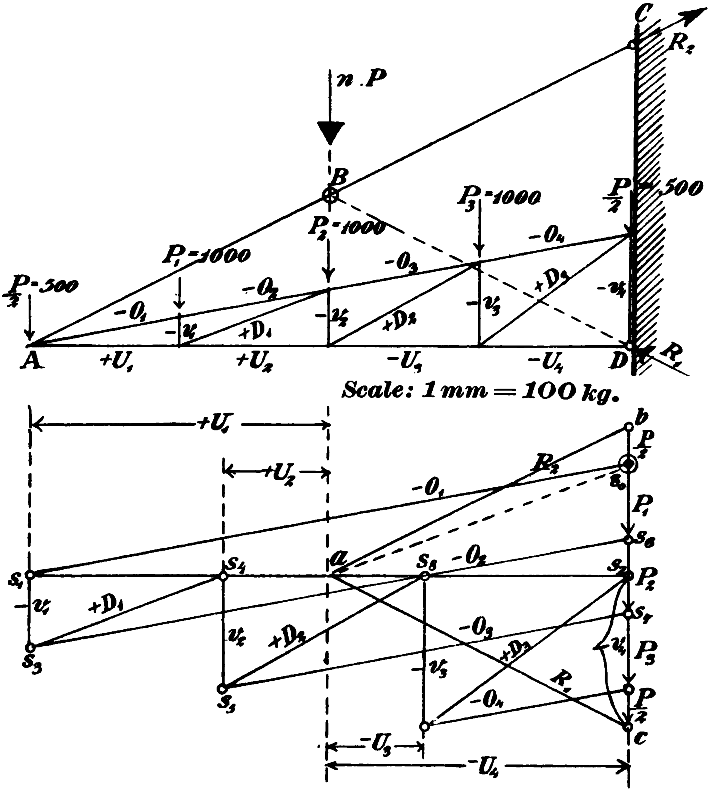


Fig. 72—73.

External forces:

The **single forces** P_1 to P_8 1000 kg each and $\frac{P}{2} = 500$ kg acting on the **upper bar.**

Determination of the stresses:

Set off the forces $\frac{P}{2}$, P_1 to P_8 and $\frac{P}{2}$ in the force diagram; draw from s_0 a parallel to O_1, and from s_2 a parallel to U_1, and the point of intersection s_1 is obtained; by drawing parallels from s_1 to V_1, and from s_6 to O_2, we obtain the point of intersection s_3. A parallel from s_3 to D_1 gives us s_4. Proceed in this manner, until the polygon is closed. By drawing from b a parallel to AC, and from c a parallel to BD the two reactions R_1 and R_2 are constructed; R_1 is a **thrust,** and R_2 a **tension.**

B is the point of intersection of the resultant $n \cdot P$ of the external forces P with AC, in our figure it happens to coincide with P_2.

For the **calculation of the dimensions** of the single bars see p. 28.

V. Open web girders.

Simple parallel beam.

Height of the beam about $^1/_{10}$ of the width of the span.

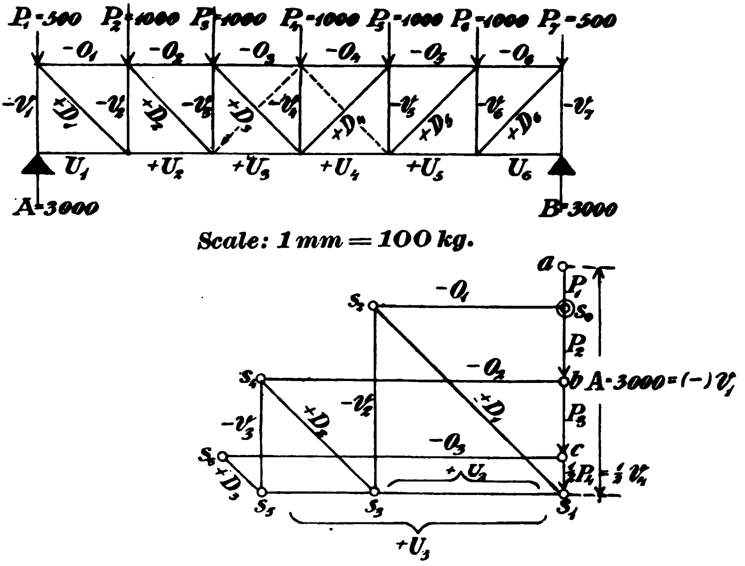


Fig. 74—75.

External forces:

P_1 and P_7 500 kg each P_2 to P_6 1000 kg each, acting on the **upper flange.**

Determination of the stresses:

Set off the forces P_1, P_2, P_3 and $^1/_2 P_4$ in the force diagram, draw from s_0 a parallel to O_1, and from s_1 a parallel to D_1, and the point of intersection s_2 is obtained.

By drawing parallels from s_2 to V_2 and from s_1 to the lower flange, the point of intersection s_3 is obtained.

Proceeding in this manner, the polygon of forces closes of itself. The **bars** U_1 and U_6 are **unstrained.**

If the forces are applied to the **lower flange,** the force diagram is similarly constructed.

In the **middle** fields **opposite diagonals** are purposely arranged.

For the **calculation of the dimensions** of single bars see p. 28.

Warren girders without vertical extremities.

Height of the beam about $^{1}/_{10}$ of the width of span.

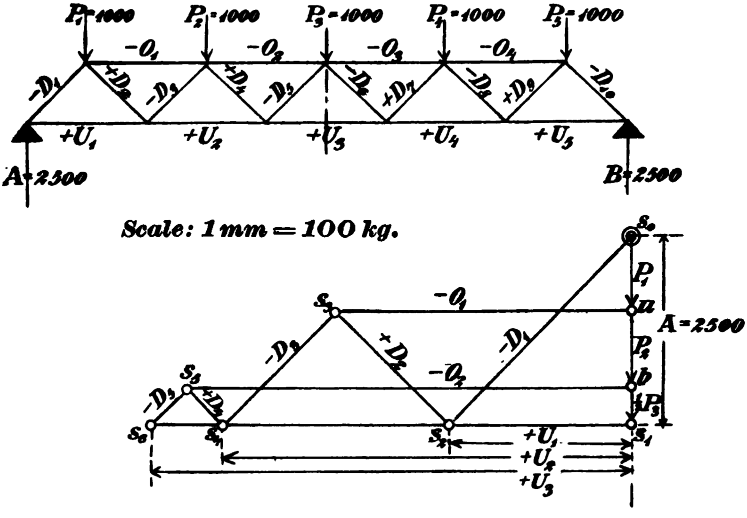


Fig. 76—77.

External forces:

The single forces P_1 to P_5 1000 kg each, acting on the **upper flange.**

Determination of the stresses:

Set off the forces P_1, P_2 and $^1/_2 P_3$ in the force diagram, draw from s_0 a parallel to D_1 and from s_1 a parallel to U_1, and the point of intersection s_2 is obtained.

By drawing from s_2 and a parallels to D_2 and O_1 respectively, the point of intersection s_3 is obtained.

By drawing from s_3 a parallel to D_3 in the direction of U_2, we obtain the point of intersection s_4.

If we now draw from s_3 towards O_2 a parallel to D_4, we obtain the point of intersection s_5.

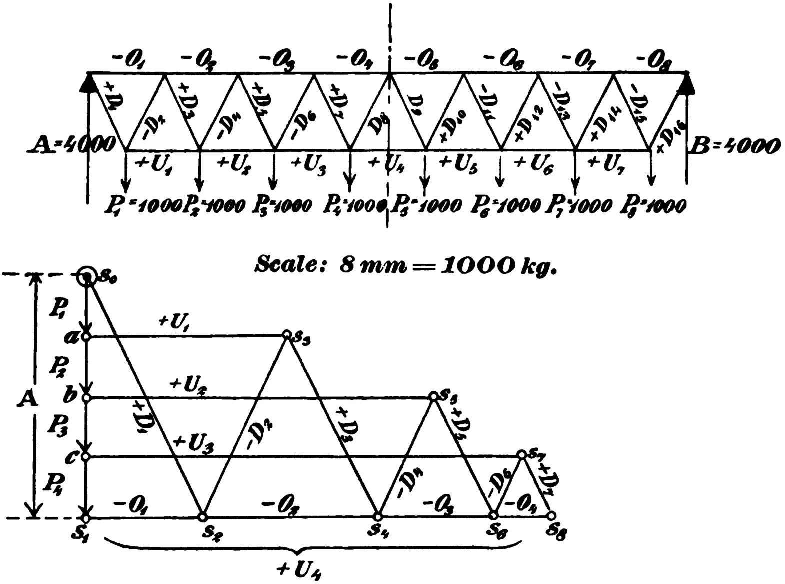
Now draw from s_5 a parallel to D_5, and the point of intersection s_6 will be obtained.

If we apply the forces at the **lower flange**, the diagram of forces is similarly constructed.

For the **calculation of the dimensions** of the single bars see p. 28.

Warren girders as suspension work.

Height of the beam usually $^1/_{10}$ of the span width.



Scale: 8 mm = 1000 kg.

Fig. 78—79.

External forces:

The single forces P_1 to P_8 1000 kg each at the **lower flange.**

Determination of the stresses:

Set off the forces P_1, P_2, P_3, P_4 in the force diagram, draw from s_0 a parallel to D_1, and from s_1 a parallel to the upper flange, and the point of intersection s_2 is obtained.

By drawing from s_2 and a parallels to D_2 and U_1 respectively, the point of intersection s_3 is obtained.

If we now draw from s_3 a parallel to D_3, we obtain the point of intersection s_4.

Pproceeding in this manner, the polygon closes of itself.

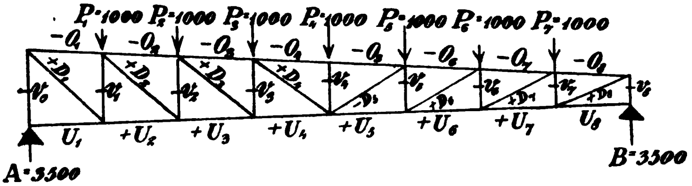
The line s_1 s_2 represents the force of the flange $s_1 s_4 = O_2$ etc.

The diagonals D_8 D_9 are unstrained.

If the forces act on the upper flange, the force diagram is similarly constructed.

For the **calculation of the dimensions** of the single bars see p. 28.

Warren girders with upper flange oblique.



Scale: 5 mm = 1000 kg.

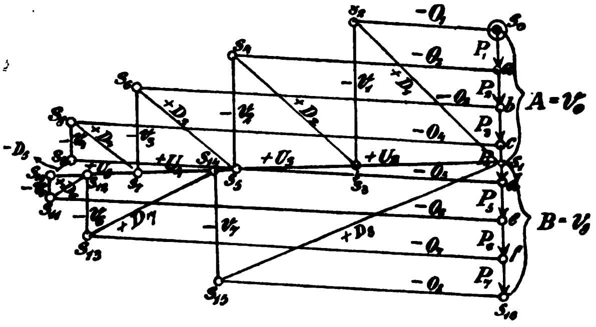


Fig. 80—81.

External forces:

The single forces P_1 to P_7 1000 kg each, acting on the **upper flange.**

Determination of the stresses:

Set off the forces P_1, P_2 to P_7 in the force diagram, and draw from the points of intersection s_0, a, b, c etc. parallels to the upper flange.

If we draw through the **middle** point of the forces a parallel to the lower flange and to D_1, we obtain the point of intersection s_2.

By drawing from s_2 a parallel to V_1 we obtain s_3; and then by drawing from s_3 a parallel to D_2 s_4 is found.

Proceeding in this manner, the polygon of forces closes.

For ex. the line $s_1\, s_3$ equals the stress $+ U_2$, $s_1\, s_5 = + U_3$, $s_1\, s_7 = + U_4$, $s_1\, s_{10} = + U_5$, $s_1\, s_{12} = + U_6$, $s_1\, s_{14} = + U_7$.

The **bars of the flange** U_1 and U_8 are unstrained.

The stress D_5 is a thrust (—); if however, the upper flange were horizontal, it would be a tension (+).

For the **calculation of the dimensions** of the single bars see p. 28.

Suspension work with lower member arched.

(Any shape of arch.)

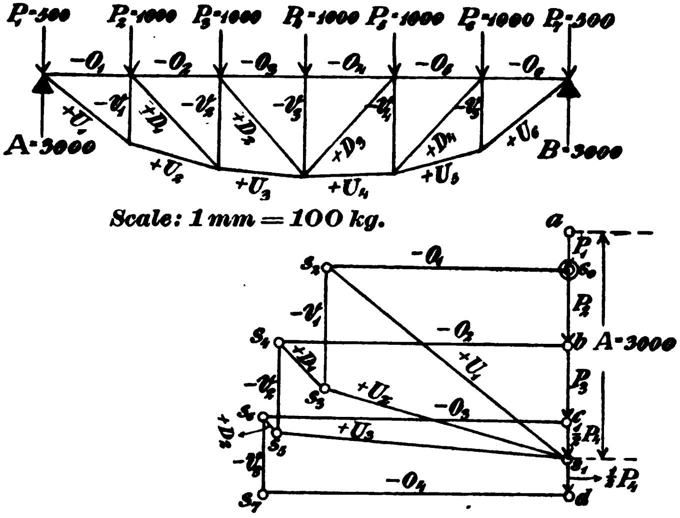


Fig. 82—83.

External forces:

The single forces P_1 and P_7 500 kg each, P_2 to P_6 1000 kg each, acting on the **upper bar.**

Determination of stresses:

Set off the forces P_1, P_2, P_3 and P_4 and the pressure A in the force diagram, draw from s_0 a parallel to O_1, and from s_1 a parallel to U_1, and the point of intersection s_2 will be obtained.

By drawing from s_2 and s_1 parallels to V_1 and U_2 respectively, the point of intersection s_3 is obtained.

If we now draw from s_3 and b parallels to D_1 and O_1 respectively, we obtain the point of intersection s_4.

Proceeding in this manner, the polygon of forces closes.

For the **calculation of the dimensions** of the single bars see p. 28.

Fish belly shaped beams.

(Arc shape: circular or parabolar.)

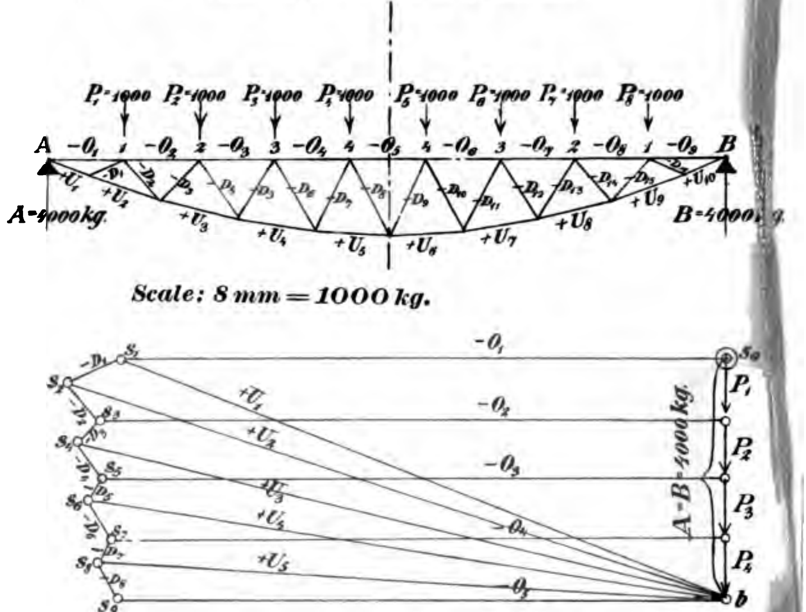


Fig. 84—85.

External forces:

The single forces P_1 to P_8 1000 kg each, acting on the **upper flange.**

Determination of the stresses:

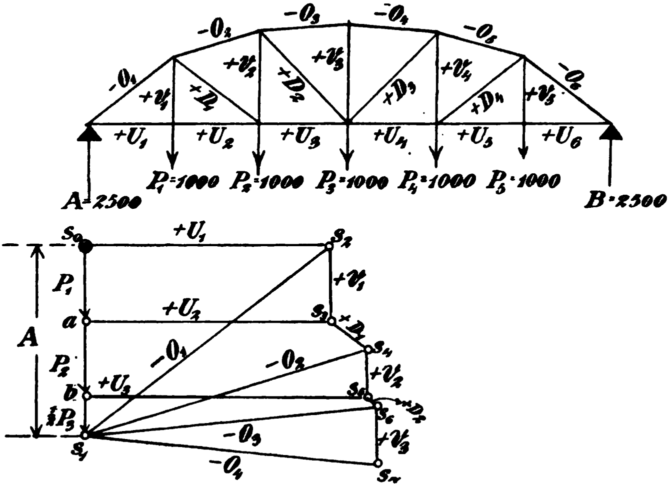
Set off the forces P_1 to P_4 in the force diagram, draw from s_0 and b parallels to O_1 and U_1 respectively, and the point of intersection s_1 will be obtained; by drawing from s_1 and b parallels to D_1 and U_2 respectively, the point of intersection s_2 is obtained.

Proceed in the manner shown until the polygon of forces is completely closed.

For the **calculation of the dimensions** of the single bars see p. 28.

Web girder with upper flange arched.

(Any shape of arc.)



Scale: 100 kg = 1 mm.

Fig. 86—87.

External forces:

The single forces P_1 to P_5, 1000 kg each, acting on the **lower flange.**

Determination of the stresses:

Set off the forces P_1, P_2 and $^1/_2 P_3$ in the force diagram, draw from s_0 and s_1 respectively parallels to U_1 and O_1, and the point of intersection s_2 will be obtained.

By drawing from s_2 and a parallels to V_1 and U_2 respectively, the point of intersection s_3 is obtained.

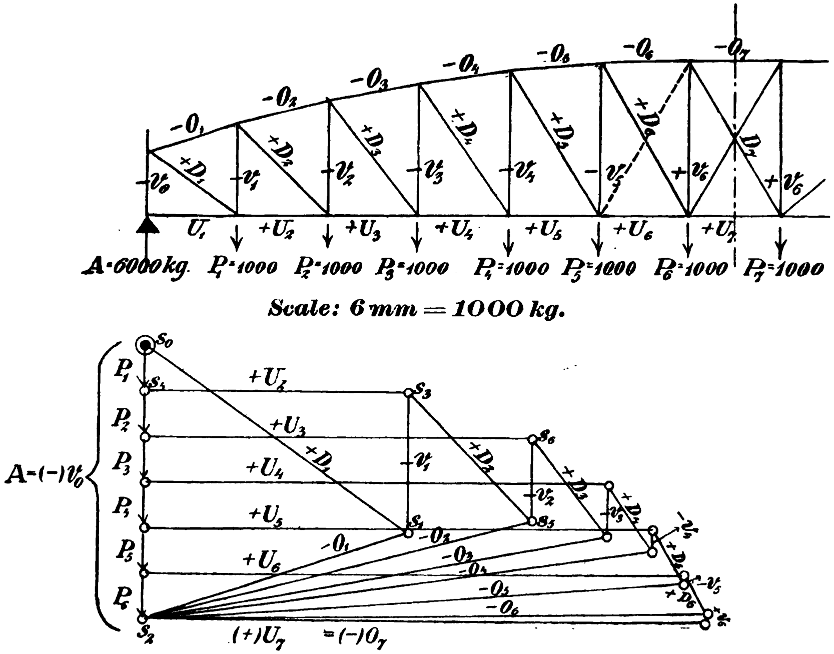
If we now draw from s_3 and s_1 parallels to D_1 and O_2 respectively, we obtain the point of intersection s_4.

Proceeding in this manner, the polygon of forces closes.

For the **calculation of the dimensions** of the single bars see p. 28.

Beams formed in a semi-parabola.

(Arc shape: parabola.)



Scale: 6 mm = 1000 kg.

Fig. 88—89.

External forces:

The single forces P_1 to P_{12}, 1000 kg each, acting on the **lower bar.**

Determination of the stresses:

Set off the forces P_1 to P_6 in the force diagram, draw from s_0 and s_2 parallels to D_1 and O_1 respectively, and the point of intersection s_1 will be obtained.

By drawing from s_1 and s_4 parallels to V_1 and U_2 respectively, the point of intersection s_3 is obtained.

Proceed in the manner shown till the polygon closes.

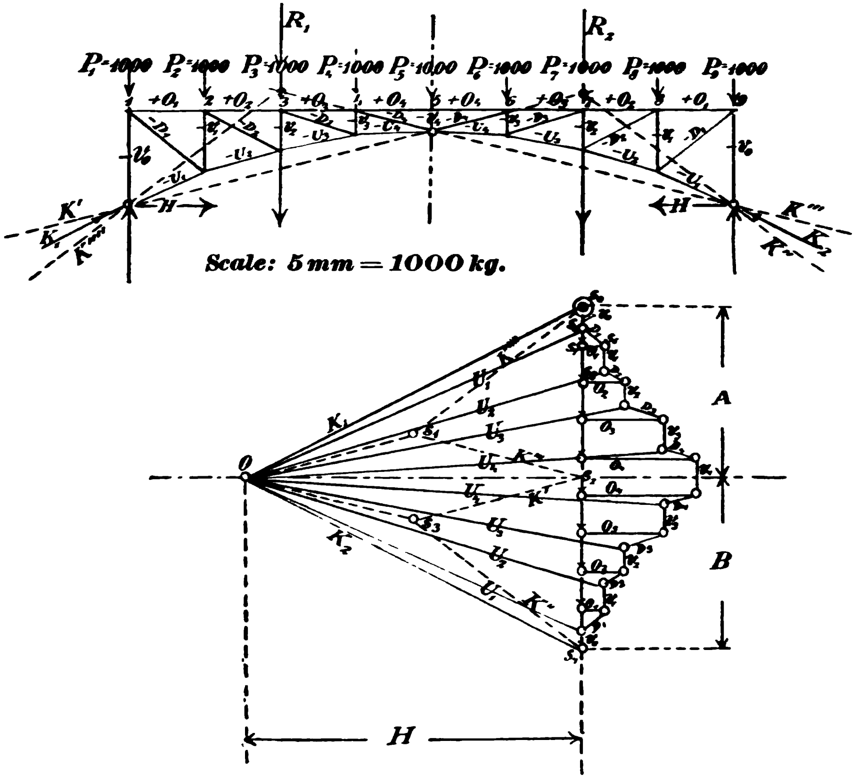
There is no stress in the bars U_1 and D_7.

V_0, V_1 to V_5 are (—), but V_6 is (+).

In the **middle** fields **opposite diagonals** are purposely arranged.

For the **calculation of the dimensions** of the single bars see p. 28.

VI. Curved beams with 3 joints.



Scale: 5 mm = 1000 kg.

Fig. 90—91.

With beams with 3 joints the working stress may be determined without the aid of the theory of elasticity.

Changes of temperature and slight displacements on the abutments are without influence.

Shocks injure the upper joint.

External forces:

The single forces P_1 to P_9, 1000 kg each, acting on the **upper bar.**

Determination of the stresses:

Set off the forces P_1 to P_9 in the force diagram, draw from s_0 and s_2 parallels to K'''' and K''', from s_2 a parallel to K', and from s_4 a parallel to K'', and the points of intersection s_1, s_3 will be obtained.

Upon s_1, s_2 and s_3 describe the parallelogram s_1 O s_3 s_2. The lines from O to s_0 and s_4 give us the reactions K_1 and K_2. Draw from O a parallel to U_1, and the point of intersection s_5 is obtained; then by drawing from s_5 and s_7 parallels to D_1 and O_1 respectively, s_6 is obtained. From s_6 a parallel drawn to V_1, to its intersection with U_2, gives us s_8.

Proceed in the manner shown till the polygon of forces is closed.

The horizontal reaction of the abutments = the distance of the pole H.

The position of R_1 and R_2 can easily be found by means of a funicular polygon.

In the case in question they lie at P_3 and P_7 respectively; usually R_1 falls between P_2 and P_3, R_2 between P_7 and P_8.

For the **calculation of the dimensions** of the single bars see p. 28.

VII. Suspension Bridges.

(Height of the arc usually $^1/_7$—$^1/_{25}$ of the chord.)

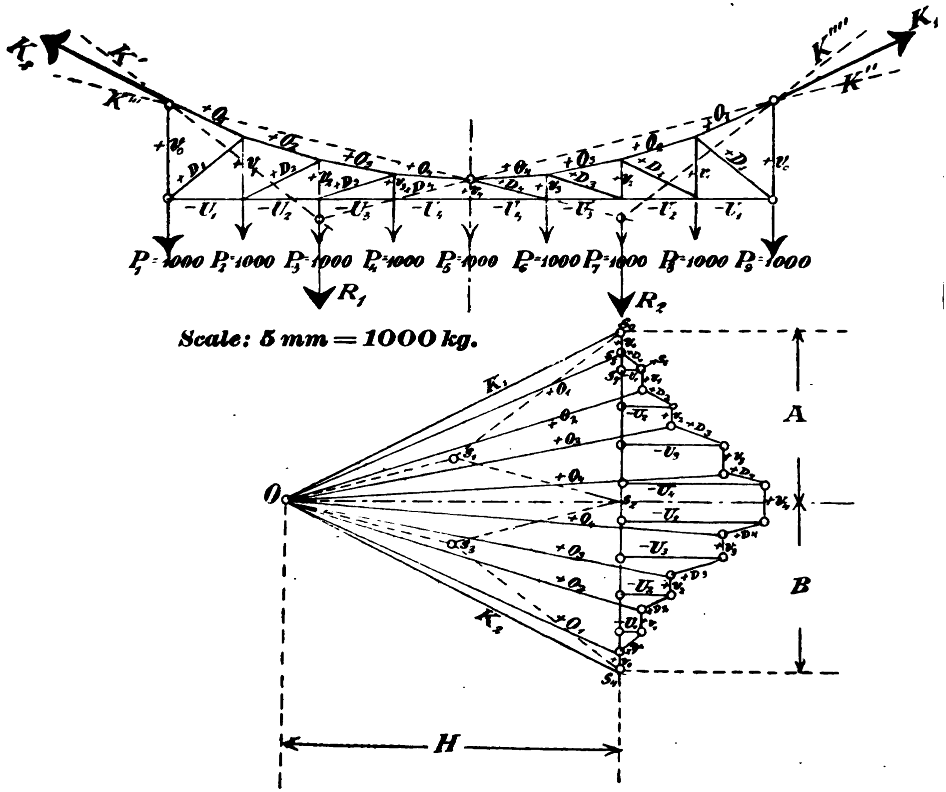


Fig. 92—93.

External forces:

The single forces P_1 to P_9, 1000 kg each, on the **lower member.**

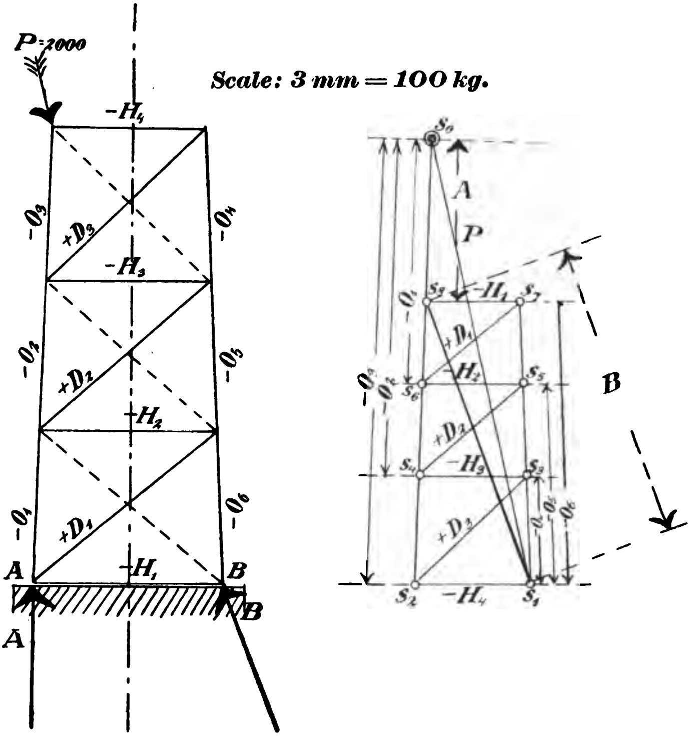
Determination of the stresses:

The application of the polygon of forces is the same as in the case of curved beams with 3 joints (Fig. 90), also with regard to R_1 and R_2, only the signs are different.

The **upper bar** is in tension, the **lower bar** in **compression.** All the transverse bars are in **tension.**

For the **calculation of the dimensions** of the single bars see p. 28.

VIII. Framework columns.



Scale: 3 mm = 100 kg.

Fig. 94—95.

External forces:

A **single force** $P = 2000$ kg, acting **obliquely upwards.**

Determination of the stresses:

Mark off the force P in the direction of the forces, draw from s_0 and s_1 respectively parallels to O_3 and H_4, and the point of intersection s_2 will be obtained.

By drawing from s_2 and s_1 parallels to D_3 and O_4 respectively, the point of intersection s_3 is obtained.

Proceed in this manner until the polygon is closed.

The final lines, s_8 s_0 and s_1 s_8 represent the reactions A and B.

Opposite diagonals are purposely arranged.

For the **calculation of the dimensions** of the single bars see p. 28.

IX. The earth pressure.

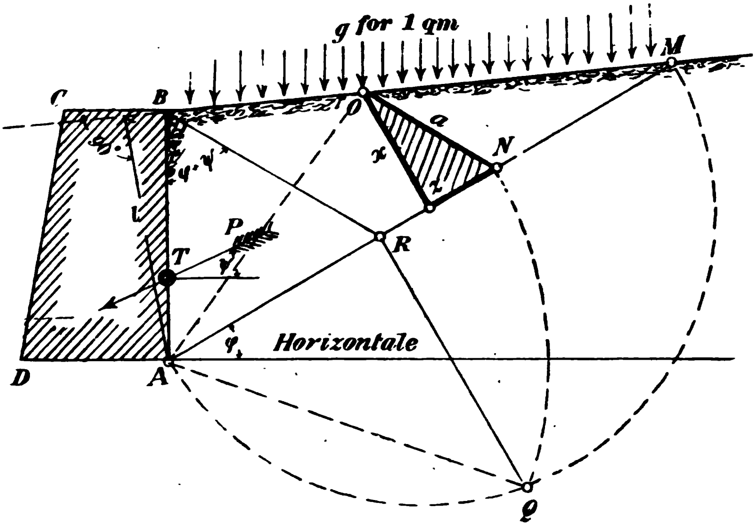


Fig. 96.

Determination of the force *P*:

Draw *AM* forming with the horizontal the angle φ.

φ is the natural angle of inclination of the earth = 20°—50°; in the most unfavourable case, if the earth be very wet 20°.

Then draw *BR*, making with *AB* the angle $(\varphi+\psi)$; on *AM* describe a semicircle and draw *QR* perpendicular to *AM*, also $AN=AQ$, draw $NO=a$ parallel to *BR*, and draw from *O* a perpendicular $OZ=x$ on *AM*; then the earth pressure is

$$(1)\quad P=\frac{1}{2}\gamma\cdot a\cdot x.$$

γ is the specific gravity of the earth = 1500—2200 kg/cbm.

ψ is the angle which the pressure of the earth makes with the normal of the inside wall *AB*, and, as a rule, is taken to be equal to φ, so that $\varphi+\psi=2\varphi$; but in the case of very wet earth $\psi=o$, i. e. supposing that *P* is horizontal.

If another burden of magnitude *g* for 1 sq. m. is brought up along *BM*, then will

$$(2)\quad P=\frac{1}{2}\left(\gamma+\frac{2g}{l}\right)\cdot a\cdot x.$$

l = the perpendicular from *A* upon *BM*.

The point of application of the earth pressure *T* lies upon *AB* and

$$AT \text{ for formula } (1)=\frac{1}{3}AB,$$

for formula (2) $AT=$ the height of the centre of gravity of a trapezium between *CB* and *DA*,

of which the upper parallel side $=\frac{g}{\gamma\cdot l}\cdot c,$

" " " lower " " $=\left(\frac{g}{\gamma\cdot l}+1\right)\cdot c$

when *c* represent any length.

X. Sustaining Walls.

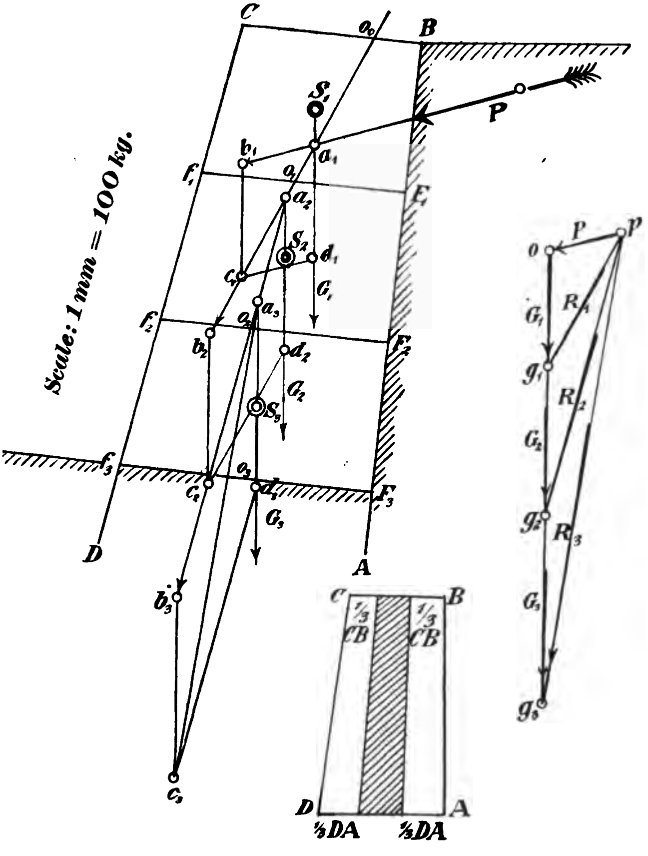


Fig. 97—99.

External forces:

A single force $P = 1000$ kg, acting on the **bulk of the wall.**

Determination of the stability:

To obtain the **line of pressure,** we divide the bulk of the wall A, B, C, D into single parts by the sections F_1, F_2, F_3, whose weights G_1, G_2, G_3 etc. can be considered to act through their centres of gravity S_1, S_2, S_3.

Drawing the parallelogram of forces a_1, b_1, c_1, d_1, for the force P and the weight G_1 of the uppermost part of the wall, we obtain their resultant R_1, represented by $a_1\ c_1$ and meeting the section F_1 in o_1. R_1 and G_2 intersect in a_2, hence the parallelogram of forces of R_1 and G_2 (a_2, b_2, c_2, d_2). Hence R_2, which is the resultant of $G_1 + G_2$.

Proceed in the manner given to the last division of the wall.

By joining the points o_0, o_1, o_2 and o_3, a continuous **curve** is obtained, which represents the **middle line** of the pressure, and is termed the **line of pressure.**

In order that there may be no danger of collapse, this **line of pressure** must remain within the middle third of the bulk of the wall (Fig. 99).

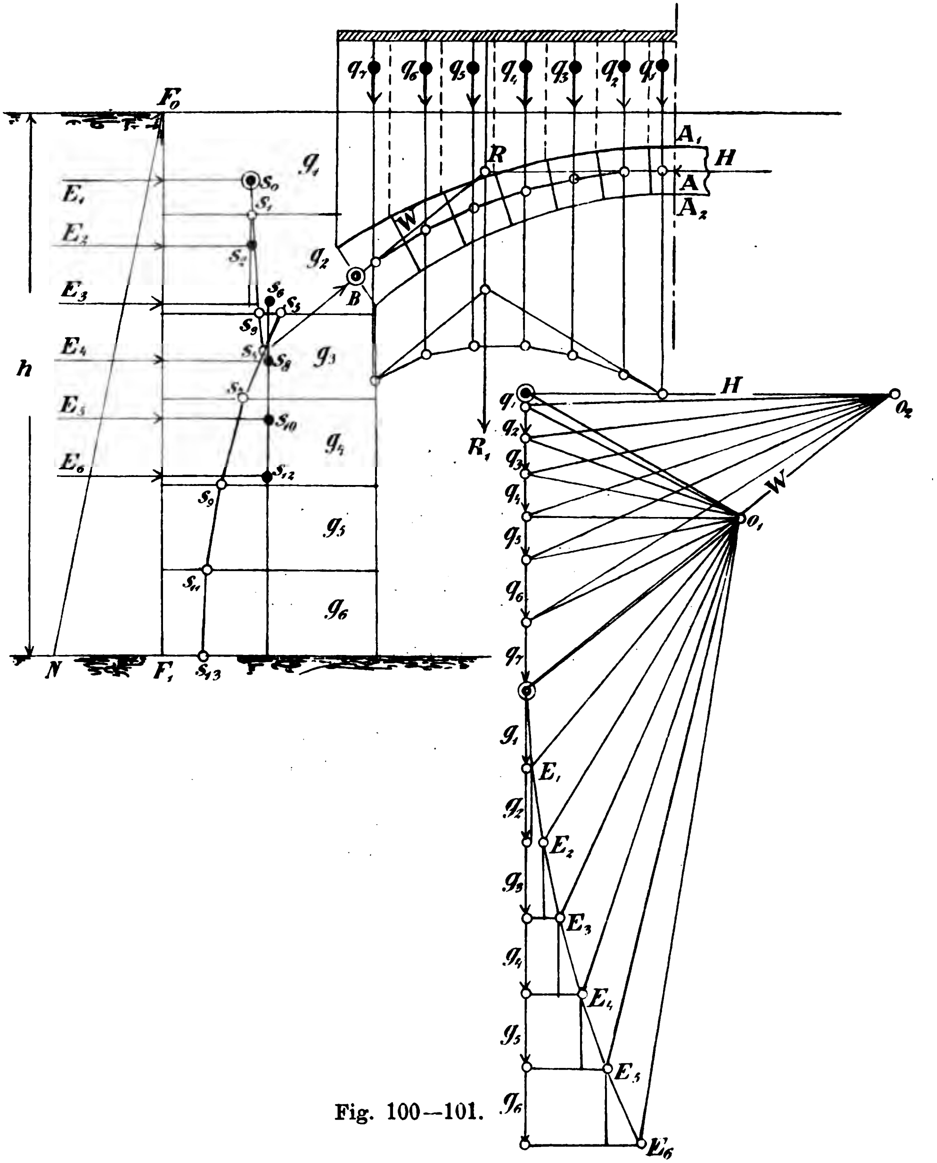

q7
q6
q5
q4
q3
q2
q1
F0
A1
H
A
A2
R
W
g1
g2
g3
g4
g5
g6
B
E1
E2
E3
E4
E5
E6
S0
S1
S2
S3
S4
S5
S6
S8
S9
S10
S11
S12
S13
h
N
F1
R1
H
O2
O1
W
E1
E2
E3
E4
E5
E6

Fig. 100—101.

XI. Vaults with abutments.

Fig. 100—101.

External forces:

The weight of **half the load of the vault** and the **pressure of the earth behind.**

Determination of the line of sustension:

Make a trial drawing of the abutment and the shape of the arc of the vault. Then mark off the weight of half the vault $A_1\ A_2\ B$ in the diagram of forces, so that this is divided up into the spaces denoted by $q_1, q_2 \ldots q_7$.

Then taking any pole o_1 construct a funicular polygon, which gives the middle force R_1.

The **push** W, the **middle force** R_1 and the **horizontal push** H meet in the point R; if we resolve the middle force R_1 in the directions W and H, we obtain the pole O_2. By the aid of this pole we draw AB **the line of sustension of the vault.**

If the latter deviates perceptibly from the middle axis of the vault, it is advisable to repeat the proceeding. The line of sustension at least must remain within the middle $^1/_8$ of the cross section.

Then mark off in the force diagram the weight of the single **abutments** g_1, g_2 to g_6 proceeding from q_7.

To fix the earth pressure E against the surface of the wall F_0F_1, set off $F_1N = {}^1/_5\ F_0F_1$; hence is F_0N obtained;

$$\text{then:}\quad E = \gamma_1 \frac{h^2}{2} \operatorname{tang}^2 \frac{90^0 - \varrho}{2},$$

in which $\gamma_1 = 1600\,\text{kg}$ = weight of the earth per cub. metre,
$= 36^0$ for average earth,
h = the height of the abutment regarded from the various sections of the bulk of the wall g_1 to g_6.

Mark off the ascertained value of the earth pressure E_1, E_2 to E_6 in the force diagram, for Ex. $g_1 E_1$, $g_2 E_2$ etc.

From the **point of intersection s_0** draw a parallel line to $q_7 E_1$ to its point of intersection s_1. By drawing through s_2 a parallel to $q_7 E_2$, the point of intersection s_3 is obtained. By joining the points s_1 and s_3, we obtain the line of sustension for g_1, g_2.

If we now produce the line $s_2 s_3$ to the **intersection with the vault pressure W,** we obtain the point s_4.

Through this point of intersection s_4 we draw a parallel to $E_3 o_1$, and this gives the point of intersection s_5 the point of **application of the resultant.** Then draw through the point of intersection s_6 a parallel to $q_7 E_3$, till it meets the vault pressure W, draw through this point of intersection a parallel to $E_3 o_1$, and the point of intersection s_7 is obtained.

Proceed in the manner shown, and the rest of the points of intersection s_8, s_9 to s_{18} are easily obtained.

It is convenient to draw the force diagram about five times as large as Fig. 101, in order that the auxiliary line of the sustension line of the abutment may be expressed as clearly as possible.

The **statical calculations** for the bulk of the wall should be made from a depth of 1 metre.

XII. Concrete constructions.

(„System Hennebique“.)

1. Examination of the bending.

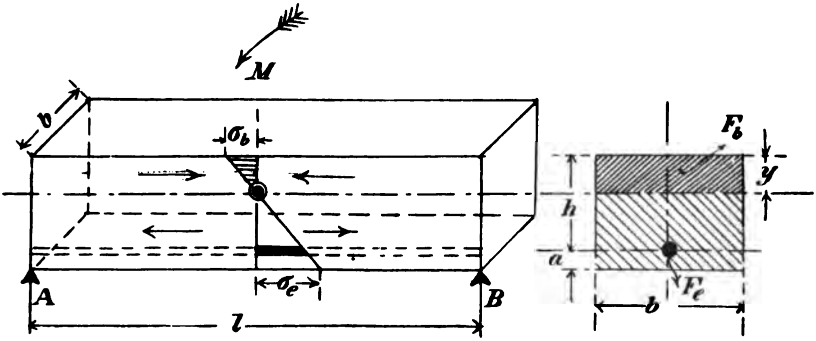


Fig. 102.

In this system compressive forces are supported by concrete, and tensile forces by iron. Hence the elastic properties of both materials have to be considered.

(Experiments on the modulus of elasticity E_b and the coefficient of the stability α of the concrete, see: "Zeitschrift des Vereins deutscher Ingenieure" 1896, p. 1381.)

In practice E_b is usually taken as = 200000, which gives us the proportion of

$$\frac{E_b}{E_e} = \frac{200000}{2000000} = \frac{1}{10} = \beta.$$

The **moment of application** for a beam under a bending strain is acc. Fig. 102:

$$M = \sigma_b \cdot \frac{b \cdot y}{2} \cdot \frac{2}{3} \cdot y + \sigma_e \cdot F_e (h - y),$$

$$\frac{\sigma_b}{\sigma_e} = \frac{y \cdot \beta}{h - y};$$

further: $$y = \frac{-2 F_e + \sqrt{4 F_e^2 + 8 b \cdot \beta \cdot h \cdot F_e}}{2 b \cdot \beta},$$

$$\sigma_b = \frac{M}{b \cdot \frac{y^2}{3} + \frac{b \cdot y}{2}(h - y)},$$

$$\sigma_e = \frac{\rho_b \cdot b \cdot \frac{y}{2}}{F_e}.$$

In this:

σ_b = the stress in cement,
σ_e = „ „ „ iron,
y = the distance of the neutral axis,
b = the breadth of the beam,
F_e = the cross section of the iron.

The calculations for a **concrete beam** are made, in the first instance as if it were not trussed; for σ_b we have 30 kg/cm².

A **method of approximation** very much in use for the determination of the **iron cross sections** requisite for **plates** is that of Kōnen, in which the distance of the centres of compression is set at the empirical value $^3/_4\, d$:

$$F_e = \frac{M}{\sigma_e \cdot {}^3/_4 d}.$$

(See also: "Centralblatt der Bauverwaltung" 1886.)

2. Examination of supports.

If the load works **within** the iron core **of the cross section,** there can only be **compressive forces,** a null line cannot be formed.

The **breaking force** K of a pillar is

$$K=\frac{\pi^2\cdot E\cdot J}{n\cdot l^2}$$

in which

$$E_x=\frac{\mu+m}{\mu+1}\cdot E_b;\quad \frac{F_b}{F_e}=\mu;$$

$$\frac{E_e}{E_b}=m;$$

for $m=15$, $\mu=50$

$$\boldsymbol{E_x=\sim 170000.}$$

For a factor of safety 8 to 10:

$$K=\frac{10}{8}\cdot\frac{170000\cdot J}{l^2},$$

$$J=\frac{8}{170000}\cdot K\cdot l^2,$$

in which K is in kg, l in cm.

If we take K in tons, l in m, then

$$\boldsymbol{J=\sim 50\,K\cdot l^2.}$$

In a **cross section** acc. Fig. 104, a **moment of inertia** is obtained:

$$\boldsymbol{J=\frac{1}{12}\cdot h^3\cdot b+2\frac{F_e}{\beta}\left(\frac{h}{2}-a\right)^2,}$$

in which $\beta={}^1/_{10}$.

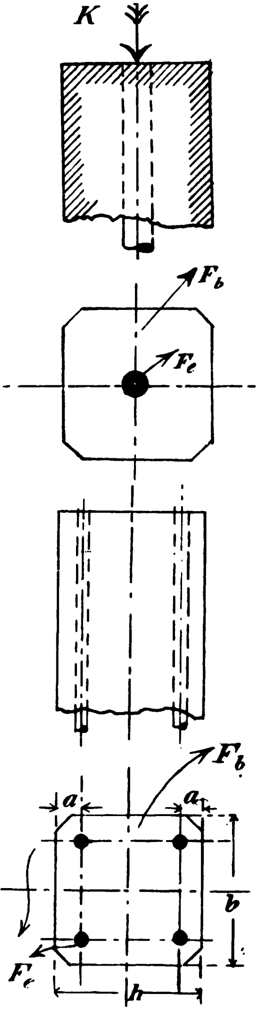


Fig. 103 and 104.

XIII. Moments of inertia and moments of resistance.

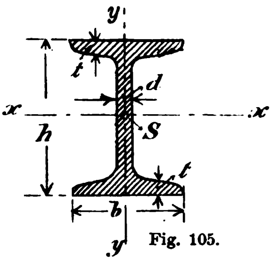


Fig. 105.

I-iron

(German Standard Sections.)

Profile No.	Dimensions in mm				Cross section F	Weight G for 1 m	Moments for axis xx		Moments for axis yy	
							Moment of inertia	Moment of resistance	Moment of inertia	Moment of resistance
	h	b	d	t	cm²	kg	T_x cm⁴	$W_x = \frac{T_x}{h/2}$ cm³	T_y cm⁴	$W_y = \frac{T_y}{b/2}$ cm³
8	80	42	3,9	5,9	7,57	5,91	77,7	19,4	6,28	2,99
9	90	46	4,2	6,3	8,99	7,02	117	25,9	8,76	3,81
10	100	50	4,5	6,8	10,6	8,28	170	34,1	12,2	4,86
11	110	54	4,8	7,2	12,3	9,59	238	43.3	16,2	5,99
12	120	58	5,1	7,7	14,2	11,1	327	54,5	21,4	7,38
13	130	62	5,4	8,1	16,1	12,6	435	67,0	27,4	8,85
14	140	66	5,7	8,6	18,2	14,2	572	81,7	35,2	10,7
15	150	70	6,0	9,0	20,4	15,9	734	97.9	43,7	12,5
16	160	74	6,3	9,5	22,8	17,8	933	117	54,5	14,7
17	170	78	6,6	9,9	25,2	19,7	1165	137	66,5	17,1
18	180	82	6,9	10,4	27,9	21,7	1444	161	81,3	19,8
19	190	86	7,2	10,8	30,5	23,8	1759	185	97,2	22,6
20	200	90	7,5	11,3	33,4	26,1	2139	214	117	25,9
21	210	94	7,8	11,7	36,3	28,3	2558	244	137	29,3
22	220	98	8,1	12,2	39,5	30,8	3055	278	163	33,3
23	230	102	8,4	12,6	42,6	33,3	3605	314	188	36,9
24	240	106	8,7	13,1	46,1	35,9	4239	353	220	41,6
25	250	110	9,0	13,6	49,7	38,7	4954	396	255	46,4
26	260	113	9,4	14,1	53,3	41,6	5735	441	287	50,6
27	270	116	9,7	14,7	57,1	44,5	6623	491	325	56,0
28	280	119	10,1	15,2	61,0	47,6	7575	541	363	60,8
29	290	122	10,4	15,7	64,8	50,6	8619	594	403	66,1
30	300	125	10,8	16,2	69,0	53,8	9785	652	449	71,9
32	320	131	11,5	17,3	77.7	60,6	12493	781	554	84,6
34	340	137	12,2	18,3	86,7	67,6	15670	922	672	98,1
36	360	143	13,0	19,5	97,0	75,7	19576	1088	817	114
38	380	149	13,7	20,5	107	83,4	23978	1262	972	131
40	400	155	14.4	21,6	118	91,8	29173	1459	1160	150
42½	425	163	15,3	23,0	132	103	36956	1739	1433	176
45	450	170	16,2	24,3	147	115	45888	2040	1722	203
47½	475	178	17,1	25,6	163	127	56410	2375	2084	234
50	500	185	18,0	27,0	179	140	68736	2750	2470	267
55	550	200	19,0	30,0	212	166	99054	3602	3486	349

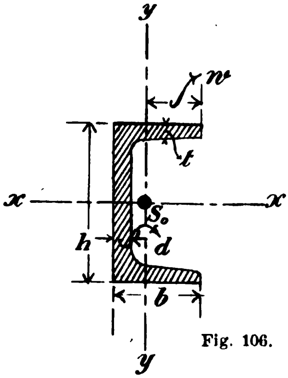


Fig. 106.

[- iron

(German Standard Sections.)

Profile No.	Dimensions				Cross section	Weight for 1 m	Distance of the centre of gravity	Moments for the axis xx		Moments for the axis yy	
	Height h mm	Breadth b mm	Web d mm	Flange t mm	F cm²	G kg	w cm	Moment of inertia T_x cm⁴	Moment of resistance $W_x = \frac{T_x}{h/2}$ cm³	Moment of inertia T_y cm⁴	Moment of resistance $W_y = \frac{T_y}{w}$ cm³
3	30	33	5	7	5,44	4,24	1,99	6,39	4,26	5,33	2,68
4	40	35	5	7	6,21	4,85	2,17	14,1	7,10	6,68	3,08
5	50	38	5	7	7,12	5,55	2,43	26,4	10,6	9,12	3,75
6½	65	42	5,5	7,5	9,03	7,05	2,78	57,5	17,7	14,1	5,06
8	80	45	6	8	11,0	8,60	3,05	106	26,5	19,4	6,37
10	100	50	6	8,5	13,5	10,5	3,45	206	41,1	29,3	8,50
12	120	55	7	9	17,0	13,3	3,90	364	60,7	43,2	11,1
14	140	60	7	10	20,4	15,9	4,25	605	86,4	62,7	14,8
16	160	65	7,5	10,5	24,0	18,7	4,66	925	116	85,3	18,3
18	180	70	8	11	28,0	21,8	5,08	1354	150	114	22,4
20	200	75	8,5	11,5	32,2	25,1	5,49	1911	191	148	27,0
22	220	80	9	12,5	37,4	29,2	5,86	2690	245	197	33,6
24	240	85	9,5	13	42,3	33,0	6,27	3598	300	248	39,6
26	260	90	10	14	48,3	37,7	6,64	4823	371	317	47,8
28	280	95	10	15	53,3	41,6	6,97	6276	450	399	57,2
30	300	100	10	16	58,8	45,8	7,30	8026	535	495	67,8

XIII. Moments of inertia and moments of resistance.

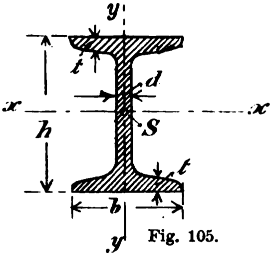


Fig. 105.

I-iron

(German Standard Sections.)

Profile No.	Dimensions in mm				Cross section F	Weight G for 1 m	Moments for axis xx		Moments for axis yy	
	h	b	d	t	cm²	kg	Moment of inertia T_x cm⁴	Moment of resistance $W_x = \frac{T_x}{h/2}$ cm³	Moment of inertia T_y cm⁴	Moment of resistance $W_y = \frac{T_y}{b/2}$ cm³
8	80	42	3,9	5,9	7,57	5,91	77,7	19,4	6,28	2,99
9	90	46	4,2	6,3	8,99	7,02	117	25,9	8,76	3,81
10	100	50	4,5	6,8	10,6	8,28	170	34,1	12,2	4,86
11	110	54	4,8	7,2	12,3	9,59	238	43.3	16,2	5,99
12	120	58	5,1	7,7	14,2	11,1	327	54,5	21,4	7,38
13	130	62	5,4	8,1	16,1	12,6	435	67,0	27,4	8,85
14	140	66	5,7	8,6	18,2	14,2	572	81,7	35,2	10,7
15	150	70	6,0	9,0	20,4	15,9	734	97.9	43,7	12,5
16	160	74	6,3	9,5	22,8	17,8	933	117	54.5	14,7
17	170	78	6,6	9,9	25,2	19,7	1165	137	66,5	17,1
18	180	82	6,9	10,4	27,9	21,7	1444	161	81,3	19,8
19	190	86	7,2	10,8	30,5	23,8	1759	185	97,2	22,6
20	200	90	7,5	11,3	33,4	26,1	2139	214	117	25,9
21	210	94	7,8	11,7	36,3	28,3	2558	244	137	29,3
22	220	98	8,1	12,2	39,5	30,8	3055	278	163	33,3
23	230	102	8,4	12,6	42,6	33,3	3605	314	188	36,9
24	240	106	8,7	13,1	46,1	35,9	4239	353	220	41,6
25	250	110	9,0	13,6	49,7	38,7	4954	396	255	46,4
26	260	113	9,4	14,1	53,3	41,6	5735	441	287	50,6
27	270	116	9,7	14,7	57,1	44,5	6623	491	325	56,0
28	280	119	10,1	15,2	61,0	47,6	7575	541	363	60,8
29	290	122	10,4	15,7	64,8	50,6	8619	594	403	66,1
30	300	125	10,8	16,2	69,0	53,8	9785	652	449	71,9
32	320	131	11,5	17,3	77.7	60,6	12493	781	554	84,6
34	340	137	12,2	18,3	86,7	67,6	15670	922	672	98,1
36	360	143	13,0	19,5	97,0	75,7	19576	1088	817	114
38	380	149	13,7	20,5	107	83,4	23978	1262	972	131
40	400	155	14.4	21,6	118	91,8	29173	1459	1160	150
42½	425	163	15,3	23,0	132	103	36956	1739	1433	176
45	450	170	16,2	24,3	147	115	45888	2040	1722	203
47½	475	178	17,1	25,6	163	127	56410	2375	2084	234
50	500	185	18,0	27,0	179	140	68736	2750	2470	267
55	550	200	19,0	30,0	212	166	99054	3602	3486	349

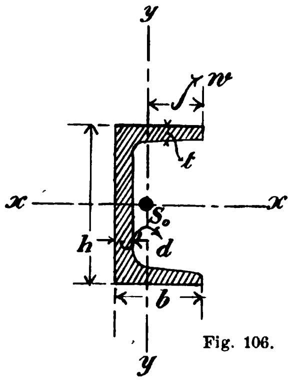


Fig. 106.

[-iron

(German Standard Sections.)

Profile No.	Dimensions				Cross section	Weight for 1 m	Distance of the centre of gravity	Moments for the axis xx		Moments for the axis yy	
	Height h mm	Breadth b mm	Web d mm	Flange t mm	F cm²	G kg	w cm	Moment of inertia T_x cm⁴	Moment of resistance $W_x=\frac{T_x}{h/2}$ cm³	Moment of inertia T_y cm⁴	Moment of resistance $W_y=\frac{T_y}{w}$ cm³
3	30	33	5	7	5,44	4,24	1,99	6,39	4,26	5,33	2,68
4	40	35	5	7	6,21	4,85	2,17	14,1	7,10	6,68	3,08
5	50	38	5	7	7,12	5,55	2,43	26,4	10,6	9,12	3,75
6½	65	42	5,5	7,5	9,03	7,05	2,78	57,5	17,7	14,1	5,06
8	80	45	6	8	11,0	8,60	3,05	106	26,5	19,4	6,37
10	100	50	6	8,5	13,5	10,5	3,45	206	41,1	29,3	8,50
12	120	55	7	9	17,0	13,3	3,90	364	60,7	43,2	11,1
14	140	60	7	10	20,4	15,9	4,25	605	86,4	62,7	14,8
16	160	65	7,5	10,5	24,0	18,7	4,66	925	116	85,3	18,3
18	180	70	8	11	28,0	21,8	5,08	1354	150	114	22,4
20	200	75	8,5	11,5	32,2	25,1	5,49	1911	191	148	27,0
22	220	80	9	12,5	37,4	29,2	5,86	2690	245	197	33,6
24	240	85	9,5	13	42,3	33,0	6,27	3598	300	248	39,6
26	260	90	10	14	48,3	37,7	6,64	4823	371	317	47,8
28	280	95	10	15	53,3	41,6	6,97	6276	450	399	57,2
30	300	100	10	16	58,8	45,8	7,30	8026	535	495	67,8

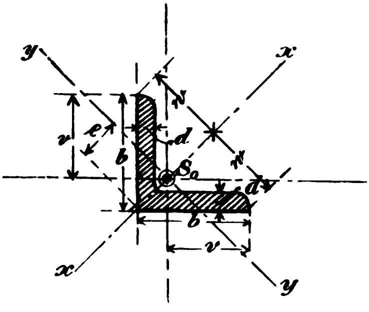


Fig. 107.

Equal angle-irons.

(German Standard Sections.)

The **root sign** for the determination of the holes in angle-iron is:

$$a = \frac{b + d}{2}.$$

Profile No.	Dimensions in mm		Cross section F	Weight G for 1 m	Distance of centre of gravity s_0 in cm		Moments for the axis xx		Moments for the axis yy	
	b	d	cm^2	kg	w	e	T_x cm^4	$W_x = \frac{T_x}{w}$ cm^3	T_y cm^4	$W_y = \frac{T_y}{e}$ cm^3
1½	15	3	0,82	0,64	1,06	0,67	0,24	0,23	0,06	0,08
		4	1,05	0,82		0,73	0,29	0,28	0,08	0,10
2	20	3	1,12	0,87	1,41	0,85	0,62	0,44	0,15	0,17
		4	1,45	1,13		0,90	0,77	0,55	0,19	0,21
2½	25	3	1,42	1,11	1,77	1,03	1,27	0,72	0,31	0,30
		4	1,85	1,44		1,08	1,61	0,91	0,40	0,37
3	30	4	2,27	1,77	2,12	1,24	2,85	1,35	0,76	0,61
		6	3,27	2,55		1,36	3,91	1,84	1,06	0,78
3½	35	4	2,67	2,08	2,47	1,41	4,68	1,90	1,24	0,88
		6	3,87	3,02		1,53	6,50	2,63	1,77	1,15
4	40	4	3,08	2,40	2,83	1,58	7,09	2,50	1,86	1,17
		6	4,48	3,49		1,70	9,98	3,52	2,67	1,57
		8	5,80	4,52		1,81	12,4	4,38	3,38	1,81
4½	45	5	4,30	3,36	3,18	1,81	12,4	3,91	3,25	1,80
		7	5,86	4,57		1,92	16,4	5,16	4,39	2,28
		9	7,34	5,73		2,04	19,8	6,24	5,40	2,65
5	50	5	4,80	3,75	3.54	1,94	17,4	4,91	4,59	2,32
		7	6,56	5,12		2,11	23,1	6,53	6,02	2,85
		9	8,24	6,43		2,21	28,1	7,94	7,67	3,47
5½	55	6	6,31	4,92	3,89	2,21	27,4	7,04	7,24	3,27
		8	8,23	6,42		2,32	34,8	8,96	9,35	4,03
		10	10,07	7,85		2,43	41,4	10,64	11,27	4,64

Profile No.	Dimensions in mm		Cross section F	Weight G for 1 m	Distance of centre of gravity s_0 in cm		Moments for the axis xx		Moments for the axis yy	
	b	d	cm^2	kg	w	e	T_x cm^4	$W_x = \frac{T_x}{w}$ cm^3	T_y cm^4	$W_y = \frac{T_y}{e}$ cm^3
6	60	6	6,91	5,39	4,24	2,39	36,1	8,51	9,43	3,95
		8	9,03	7,04		2,50	46,1	10.9	12,1	4,85
		10	11,07	8,63		2,62	55,1	13,0	14,6	5,58
6½	65	7	8,70	6,79	4,60	2,62	53,0	11,5	13,8	5,25
		9	10,98	8,56		2,73	65,4	14,2	17,2	6,31
		11	13,17	10,30		2,83	76,8	16,7	20,7	7,30
7	70	7	9,4	7,33	4,95	2,79	67,1	13,6	17,6	6,29
		9	11,9	9,26		2,90	83,1	16,8	22,0	7,57
		11	14,3	11,13		3,01	97,6	19,7	26,0	8,65
7½	75	8	11,5	8,94	5,30	3,01	93,3	17,6	24,4	8,11
		10	14,1	11,00		3,12	113	21,3	29,8	9,54
		12	16,7	13,00		3,24	130	24,6	34,7	10,71
8	80	8	12,3	9,57	5,66	3,20	115	20,3	29,6	9,25
		10	15,1	11,78		3,31	139	24,5	35,9	10,8
		12	17,9	13,94		3,41	161	28,4	43,0	12,6
9	90	9	15,5	12,1	6,36	3,59	184	28,9	47,8	13,3
		11	18,7	14,6		3,70	218	34,3	57,1	15,4
		13	21,8	17,0		3,81	250	39,3	65,9	17,3
10	100	10	19,2	14,9	7,07	3,99	280	39,7	73,3	18,4
		12	22,7	17,7		4,10	328	46,3	86,2	21,0
		14	26,2	20,4		4,21	372	52,6	98,3	23,4
11	110	10	21,2	16,5	7,78	4,34	379	48,7	98,6	22,7
		12	25,1	19,6		4,45	444	57,1	116	26,1
		14	29,0	22,6		4,54	505	64,8	133	29,2
12	120	11	25,4	19,8	8,48	4,75	541	63,8	140	29,4
		13	29,7	23,2		4,86	625	73,7	162	33,4
		15	33,9	26,5		4,96	705	83,2	186	37,5
13	130	12	30,0	23,4	9,19	5,15	750	81,6	194	37,8
		14	34,7	27,0		5,26	857	93,3	223	42,4
		16	39,3	30,6		5,37	959	104	251	46,7
14	140	13	35,0	27,3	9,90	5,54	1014	102	262	47,3
		15	40,0	31,2		5,66	1148	116	298	52,6
		17	45,0	35,1		5,77	1276	129	334	58,0
15	150	14	40,3	31,4	10,6	5,95	1343	127	347	58,3
		16	45,7	35,7		6,07	1507	142	391	64,4
		18	51,0	39,9		6,17	1665	157	438	71,1
16	160	15	46,1	35,9	11,3	6,35	1745	154	453	71,3
		17	51,8	40,4		6,46	1945	172	506	78,4
		19	57,5	44,9		6,58	2137	189	558	84,8

Adjacent Equal angles in iron in juxtaposition. (German Standard sections.)

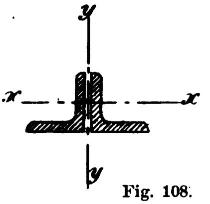


Fig. 108.
Two adjacent angles (without space between).

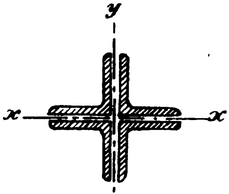


Fig. 109.
Four adjacent angles (without space between).

Profile No.	Cross section $2F$ qcm	Weight for 1 m $2G$ kg	Moments of inertia — Moments about the axis x T_2 cm^4	Moments of resistance — Moments about the axis x $W_2 = \frac{T_2}{v}$ cm^3	Cross section $4F$ qcm	Weight for 1 m $4G$ kg	Moments of inertia — Moments about the axis x or y T_4 cm^4	Moments of resistance — Moments about the axis x or y $W_4 = \frac{T_4}{b}$ cm^3
4	6,16	4,80	8,94	3,11	12,3	9,61	33,3	8,33
	8,96	6,99	12,7	4,52	17,9	14,0	51,1	12,8
	11,59	9,05	15,8	5,80	23,2	18,1	69,5	17,4
4½	8,61	6,71	15,7	4,87	17,2	13,4	59,5	13,2
	11,73	9,15	20,8	6,63	23,5	18,3	85,0	18,9
	14,68	11,50	25,2	8,25	29,4	22,9	111,2	24,7
5	9,61	7,49	22,0	6,10	19,2	15,0	81,7	16,3
	13,1	10,20	29,1	8,30	26,3	20,5	116	23,3
	16,5	12,90	35,8	10,39	33,0	25,7	152	30,4
5½	12,6	9,84	34,6	8,79	25,2	19,7	131	23,8
	16,5	12,8	44,2	11,5	32,8	25,7	177	32,2
	20,1	15,7	52,7	13,9	40,3	31,4	224	40,8
6	13,8	10,8	45,5	10,6	27,6	21,6	170	28,3
	18,1	14,1	58,3	13,8	36,1	28,2	230	38,3
	22,1	17,3	69,7	16,8	44,3	34,5	291	48,4
6½	17,4	13,6	66,8	14,4	34,8	27,2	252	38,4
	22,0	17,1	82,6	18,1	43,9	34,2	329	50,6
	26,4	20,6	97,5	21,7	52,7	41,1	406	62,5
7	18,8	14,7	84,6	16,8	37,6	29,3	315	45,0
	23,8	18,5	105	21,2	47,5	37,1	410	58,6
	28,6	22,3	124	25,4	57,1	44,5	506	72,3

Note: For the thickness of the profiles comp. Table p. 100.

Continuation of p. 102.

Profile No.	Cross section $2F$	Weight for 1 m $2G$	Moments of		Cross section $4F$	Weight for 1 m $4G$	Moments of	
			inertia	resistance			inertia	resistance
			Moments about the axis x				Moments about the axis x or y	
			T_2	$W_2=\frac{T_2}{v}$			T_4	$W_4=\frac{T_4}{b}$
	qcm	kg	cm^4	cm^3	qcm	kg	cm^4	cm^3
7½	22,9	17,9	118	21,9	45,9	35,8	444	59,2
	28,2	22,0	142	26,9	56,4	44,0	561	74,8
	33,3	26,0	165	31,7	66,7	52,0	679	90,6
8	24,5	19,1	144	25,1	49,1	38,3	539	67,3
	30,2	23,6	175	30,9	60,4	47,1	680	85,0
	35,7	27,9	204	36,4	71,5	55,7	823	102,9
9	31,0	24,2	232	35,9	62,1	48,4	863	95,9
	37,4	29,2	275	43,1	74,9	58,4	1064	118
	43,7	34,1	316	50,1	87,4	68,1	1268	141
10	38,3	29,9	354	49,3	76,6	59,8	1317	132
	45,4	35,4	414	58,3	90,9	70,9	1593	159
	52,4	40,8	470	67,0	104,8	81,7	1871	187
11	42,3	33,0	478	60,2	84,6	66,0	1753	159
	50,2	39,2	560	71,4	100	78,4	2118	193
	58,0	45,2	638	81,9	116	90,5	2486	226
12	50,7	39,6	680	78,8	101	79,2	2505	209
	59,4	46,3	787	92,1	118	92,6	2979	248
	67,9	52,9	891	105	136	105,9	3456	288
13	59,9	46,8	944	101	120	93,5	3476	267
	69,3	54,1	1080	116	139	108	4079	314
	78,5	61,2	1209	131	157	123	4685	360
14	69,9	54,5	1276	127	140	109	4702	336
	79,9	62,4	1446	145	160	125	5454	390
	89,9	70,1	1610	162	180	140	6215	444
15	80,6	62,9	1690	157	161	126	6235	416
	91,4	71,3	1898	177	183	143	7160	477
	102,1	79,6	2103	198	204	159	8091	539
16	92,1	71,9	2198	191	184	144	8110	507
	104	80,8	2451	214	207	162	9232	577
	115	89,7	2695	237	230	179	10362	648

Note: For the thickness of the profiles comp. Table p. 100.

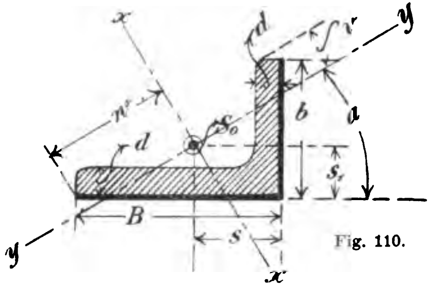


Fig. 110.

Unequal angles.

(German Standard sections.)

Proportion of the sides $\frac{B}{b} = \frac{1\frac{1}{2}}{1}$.

Profil No.	Dimensions			Cross section F	Weight G for 1 m	Distance of the centre of gravity S_0		Position of the chief axis	Distance from the chief axes		Moments for the axis xx		Moments for the axis yy	
									of the axis yy	of the axis xx	Moment of inertia	Moment of resistance	Moment of inertia	Moment of resistance
	b	B	d			s	s_1	yy	w	v	T_x	$W_x = \frac{T_x}{w}$	T_y	$W_y = \frac{T_y}{v}$
	mm	mm	mm	qcm	kg	cm	cm	tg α	cm	cm	cm^4	cm^3	cm^4	cm^3
2/3	20	30	3	1,42	1,11	0,99	0,49	0,4216	2,04	1,07	1,42	0,70	0,28	0,26
			4	1,85	1,44	1,03	0,54	0,4214	2,02	1,04	1,82	0,90	0,33	0,32
3/4½	30	45	4	2,87	2,24	1,48	0,74	0,4334	3,06	1,58	6,63	2,17	1,19	0,75
			5	3,53	2,75	1,52	0,78	0,4288	3,05	1,58	8,01	2,63	1,44	0,91
4/6	40	60	5	4,79	3,74	1,95	0,97	0,4319	4,10	2,12	19,8	4,82	3,66	1,73
			7	6,55	5,11	2,04	1,05	0,4275	4,06	2,10	26,3	6,47	4,63	2,20
5/7½	50	75	7	8,33	6,50	2,47	1,24	0,4304	5,11	2,62	53,1	10,4	9,58	3,66
			9	10,5	8,20	2,56	1,32	0,4272	5,07	2,60	65,4	12,9	11,9	4,56
6½/10	65	100	9	14,2	11,0	3,31	1,59	0,4101	6,79	3,47	160	23,6	26,8	7,73
			11	17,1	13,3	3,40	1,67	0,4074	6,74	3,45	189	28,1	32,9	9,54
8/12	80	120	10	19,1	14,9	3,92	1,95	0,4348	8,19	4,24	317	38,7	56,8	13,4
			12	22,7	17,7	4,00	2,02	0,4304	8,15	4,21	370	45,4	67,5	16,0
10/15	100	150	12	28,7	22,4	4,89	2,42	0,4361	10,2	5,26	747	73,0	134	25,4
			14	33,2	25,9	4,97	2,50	0,4339	10,2	3,27	854	83,8	153	29,0

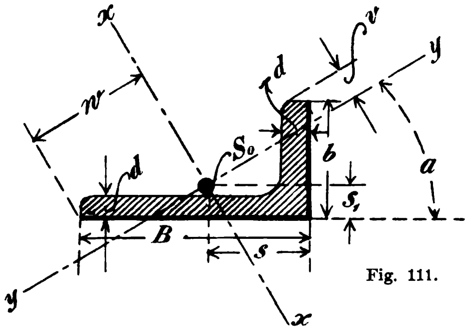


Fig. 111.

Unequal angles.

(German Standard sections.)

Proportion of the sides $\frac{B}{b} = \frac{2}{1}$.

Profil No.	Dimensions			Cross section	Weight G for 1 m	Distance of the centre of gravity S^0		Position of the chief axis	Distance from the chief axes		Moments for the axis xx		Moments for the axis yy	
									of the axis yy	of the axis xx	Moment of inertia	Moment of resistance	Moment of inertia	Moment of resistance
	b	B	d	F		s	s_1	yy tg α	w	v	T_x	$W_x = \frac{T_x}{w}$	T_y	$W_y = \frac{T_y}{v}$
	mm	mm	mm	cm²	kg	cm	cm		cm	cm	cm⁴	cm³	cm⁴	cm³
2/4	20	40	3	1,72	1,34	1,43	0,44	0,2575	2,60	1,19	2,96	1,14	0,31	0,26
			4	2,25	1,76	1,47	0,48	0,2528	2,57	1,17	3,78	1,47	0,40	0,34
3/6	30	60	5	4,29	3,35	2,15	0,68	0,2544	3,91	1,78	16,5	4,22	1,71	0,96
			7	5,85	4,56	2,24	0,76	0,2479	3,83	1,74	21,8	5,69	2,28	1,31
4/8	40	80	6	6,89	5,37	2,85	0,88	0,2568	5,21	2,38	47,6	9,14	4,99	2,10
			8	9,01	7,03	2,94	0,96	0,2518	5,14	2,35	60,8	11,8	6,41	2,73
5/10	50	100	8	11,5	8,93	3,59	1,12	0,2565	6,49	2,97	123	18,9	12,8	4,31
			10	14,1	11,0	3,67	1,20	0,2658	6,42	2,96	150	23,3	14,6	4,93
6½/13	65	130	10	18,6	14,5	4,65	1,45	0,2569	8,45	3,86	339	40,2	35,4	9,16
			12	22,1	17,2	4,75	1,53	0,2549	8,38	3,82	395	47,2	41,3	10,8
8/16	80	160	12	27,5	21,5	5,72	1,77	0,2586	10,4	4,76	762	73,4	79,4	16,7
			14	31,8	24,8	5,81	1,85	0,2679	10,3	4,65	875	84,8	86,0	18,5
10/20	100	200	14	40,3	31,4	7,12	2,18	0,2608	13,0	5,95	1754	135	182	30,6
			16	45,7	35,6	7,20	2,26	0,2586	13,0	5,93	1973	152	205	34,5

T-iron (with high vertex).

(German Standard sections.)

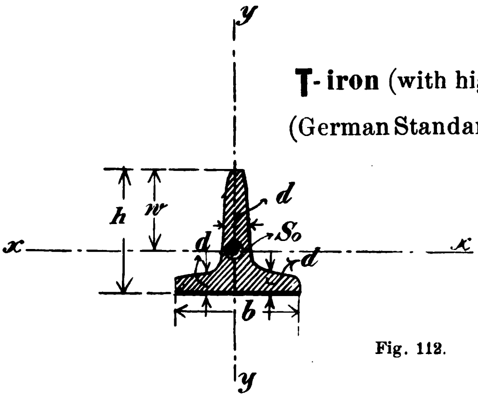


Fig. 112.

Profile No.	Dimensions			Cross section	Weight for 1 m	Distance of the centre of gravity	Moments for the axis xx		Moments for the axis yy	
	Breadth b mm	Height h mm	Thickness d mm	F qcm	G kg	w cm	Moment of inertia T_x cm⁴	Moment of resistance $W_x = \frac{T_x}{v}$ cm³	Moment of inertia T_y cm⁴	Moment of resistance $W_y = \frac{T_y}{b/2}$ cm³
2/2	20	20	3	1,12	0,87	1,42	0,38	0,27	0,20	0,20
$2\frac{1}{2}/2\frac{1}{2}$	25	25	3,5	1,64	1,28	1,77	0,87	0,49	0,43	0,34
3/3	30	30	4	2,26	1,76	2,15	1,72	0,80	0,87	0,58
$3\frac{1}{2}/3\frac{1}{2}$	35	35	4,5	2,97	2,32	2,51	3,10	1,23	1,57	0,90
4/4	40	40	5	3,77	2,94	2,88	5,28	1,84	2,58	1,29
$4\frac{1}{2}/4\frac{1}{2}$	45	45	5,5	4,67	3,64	3,24	8,13	2,51	4,01	1,78
5/5	50	50	6	5,66	4,42	3,61	12,1	3,36	6,06	2,42
6/6	60	60	7	7,94	6,19	4,34	23,8	5,48	12,2	4,05
7/7	70	70	8	10,6	8,27	5,06	44,5	8,79	22,1	6,32
8/8	80	80	9	13,6	10,6	5,78	73,7	12,8	37,0	9,25
9/9	90	90	10	17,1	13,3	6,52	119	18,2	58,5	13,0
10/10	100	100	11	20,9	16,3	7,26	179	24,6	88,3	17,7
12/12	120	120	13	29,6	23,1	8,72	366	42,0	178	29,7
14/14	140	140	15	39,9	31,1	10,2	660	64,7	330	47,2

T-iron (flat bottomed).

(German Standard sections.)

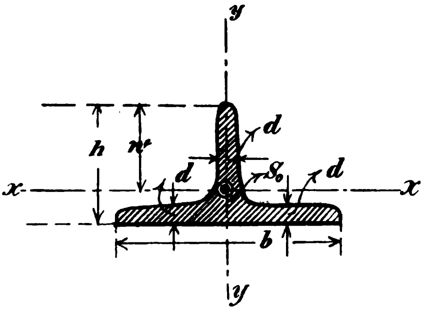


Fig. 113.

Profile No.	Dimensions			Cross section	Weight for 1 m	Distance of the centre of gravity	Moments for the axis xx		Moments for the axis yy	
	Breadth b mm	Height h mm	Thickness d mm	F qcm	G kg	w cm	Moment of inertia T_x cm^4	Moment of resistance $W_x = \frac{T_x}{v}$ cm^3	Moment of inertia T_y cm^4	Moment of resistance $W_y = \frac{T_y}{b/2}$ cm^3
6/3	60	30	5,5	4,64	3,62	2,33	2,58	1,11	8,62	2,87
7/3½	70	35	6	5,94	4,63	2,73	4,49	1,65	15,1	4,32
8/4	80	40	7	7,91	6,17	3,12	7,81	2,50	28,5	7,13
9/4½	90	45	8	10,2	7,93	3,50	12,7	3,64	46,1	10,2
10/5	100	50	8,5	12,0	9,38	3,91	18,7	4,78	67,7	13,5
12/6	120	60	10	17,0	13,2	4,70	38,0	8,09	137	22,8
14/7	140	70	11,5	22,8	17,8	5,49	68,9	12,6	258	36,9
16/8	160	80	13	29,5	23,0	6,28	117	18,6	422	52,8
18/9	180	90	14,5	37,0	28,8	7,07	185	26,1	670	74,4
20/10	200	100	16	45,4	35,4	7,86	277	35,3	1000	100

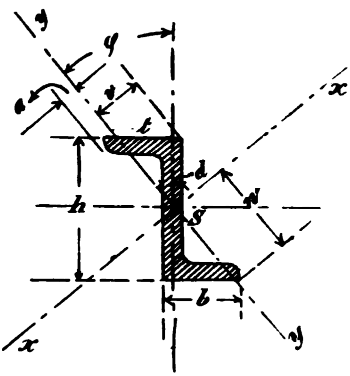


Fig. 114.

Z-iron

(German Standard sections).

Profile No.	Dimensions in mm				Cross section	Weight of 1 m	tg φ	xx axis Moment of		yy axis Moment of	
	Height	Breadth	Span	Flange	F	G		inertia T_x	resistance $W_x = \frac{T_x}{w}$	inertia T_y	resistance $W_y = \frac{T_y}{v}$
	h	b	d	t	qcm	kg		cm^4	cm^3	cm^4	cm^3
3	30	38	4	4,5	4,32	3,37	1,65	18,1	4,69	1,54	1,11
4	40	40	4,5	5	5,43	4,23	1,18	28,0	6,72	3,05	1,83
5	50	43	5	5,5	6,77	5,28	0,93	44,9	9,76	5,23	2,76
6	60	45	5	6	7,91	6,17	0,77	67,2	13,5	7,60	3,73
8	80	50	6	7	11,1	8,67	0,58	142	24,4	14,7	6,44
10	100	55	6,5	8	14,5	11,3	0,49	270	39,8	24,6	9,26
12	120	60	7	9	18,2	14,2	0,43	470	60,6	37,7	12,5
14	140	65	8	10	22,9	17,9	0,38	768	88,0	56,4	16,6
16	160	70	8,5	11	27,5	21,5	0,35	1184	121	79,5	21,4
18	180	75	9,5	12	33,3	26,0	0,32	1759	164	110	27
20	200	80	10	13	38,7	30,2	0,31	2509	213	147	33,4
											from the profile No. 10 ab $\frac{Ty}{a}$

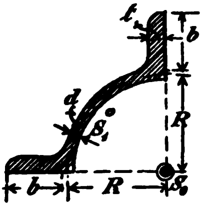
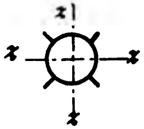
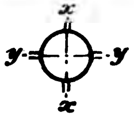


Fig. 115.

Quadrant-iron.

(German Standard sections.)

Profile No.	Dimensions in mm R	b	d	t	Cross section F of the full tubes qcm	Weight G per 1 m of the full tubes kg	Full Tubes Moment of resistance for each axis of flexion T cm⁴	 Moment of greatest resistance to flexion axis zz W_z cm³	 Moment of smallest resistance to flexion axis xx or yy $W_x = W_y$ cm³
5	50	35	4	6	29,8	23,3	576	89,3	66,2
5	50	35	8	8	48,0	37,4	906	135	102
7½	75	40	6	8	54,9	42,8	2068	237	175
7½	75	40	10	10	80,2	62,5	2982	331	248
10	100	45	8	10	88,1	68,7	5511	501	370
10	100	45	12	12	120	94,0	7478	663	495
12½	125	50	10	12	129	101	12161	917	676
12⅓	125	50	14	14	169	132	15788	1165	867
15	150	55	12	14	179	140	23637	1515	1120
15	150	55	18	17	249	194	32738	2051	1530

Trough decking.

(German Standard sections.)

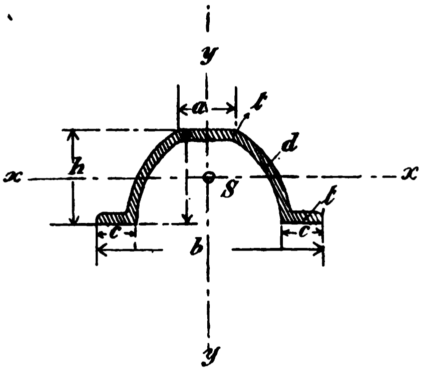


Fig. 116.

Profile No.	Height	Breadth			Thickness		Cross section	Weight G per 1 m	Moment of resistance		Moment inertia
		underneath	above	at the foot							
	h mm	b mm	a mm	c mm	d mm	t mm	F qcm	kg	T_y cm^4	T_x cm^4	W_x cm^3
5	50	120	33	21	3	5	6,71	5,24	86,4	23,2	9,27
6	60	140	38	24	3,5	6	9,34	7,28	164	47,2	15,8
7½	75	170	45,5	28,5	4	7	13,2	10,3	347	105	27,9
9	90	200	53	33	4,5	8	17,9	14,0	651	206	45,8
11	110	240	63	39	5	9	24,1	18,8	1272	421	76,5

Fist-shaped iron.

(German Standard sections.)

Length of bars 4—8 m; greatest length 12 m.

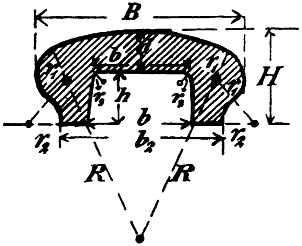


Fig. 117

$R = B$
$H = 0{,}45\ B$
$d = 0{,}2\ B$
$b = 0{,}5\ B$
$h = 0{,}25\ B$
$r_1 = 0{,}15\ B$
$r_2 = 0{,}10\ B$
$r_3 = 0{,}05\ B$
$b_1 = 0{,}45\ B$
$b_2 = 0{,}75\ B$

Profile No.	Dimensions in mm				Cross section F	Weight G for 1 m
	$B=R$	H	b	h	qcm	kg
4	40	18	20	10	4,20	3,28
6	60	27	30	15	9,46	7,38
8	80	36	40	20	16,8	13,1
10	100	45	50	25	26,3	20,5
12	120	54	60	30	37,8	29,5

Sections in use for corrugated iron.

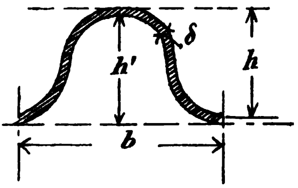


Fig. 118.

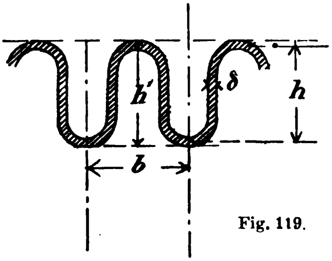


Fig. 119.

$$\text{Moment of inertia } T = \left(0{,}103 + 0{,}186\,\frac{h}{b}\right) h^2\,\delta$$

$$\text{Moment of resistance } W = \frac{2\,T}{h'} = \left(0{,}196 + 0{,}354\,\frac{h}{b}\right) h\,\delta\ .$$

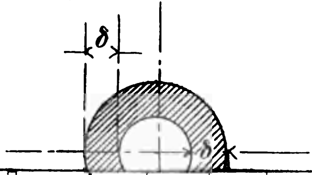
	$\frac{b}{h}$	h	b	$\frac{W}{\delta}$	Moment of resistance W in cm for 1 m with and for δ =							
		mm	mm	cm	1	1,5	2	2,5	3	4	5	6 mm
Sheet iron flat (Fig. 118)	2,5	20	50	6,6	6,6	9,9	13,2	—	—	—	—	—
		40	100	13,8	13,8	21,7	27,6	34,5	41,4	—	—	—
		60	150	20,6	—	30,9	41,2	51,5	61,8	82,4	—	—
		80	200	27,5	—	—	55,0	68,7	82,5	110,0	138,5	—
		100	250	34,4	—	—	—	86,0	103,2	137,6	172,0	206,4
		120	300	46,3	—	—	—	—	123,9	165,2	206,5	247,8
	2,0	40	80	15,3	15,9	22,9	30,6	38,3	—	—	—	—
		60	120	22,9	22,3	34,3	45,8	57,3	68,7	—	—	—
		80	160	30,5	—	45,7	61,0	76,3	94,5	122,0	—	—
		100	200	38,2	—	—	76,4	95,5	114,6	152,8	191,0	—
		120	240	45,8	—	—	—	114,5	137,9	183,2	229,0	274,8
Sheet iron beam (Fig. 119)	1,5	40	60	17,8	17,8	26,7	35,6	—	—	—	—	—
		60	90	26,6	26,6	39,9	53,2	66,5	—	—	—	—
		80	120	35,5	35,5	53,2	71,0	88,8	106,5	—	—	—
		100	150	44,4	—	66,6	88,8	111,0	133,2	177,6	—	—
		120	180	53,3	—	—	106,6	133,2	159.9	213,2	266,5	—
	1,0	60	60	34,1	34,1	51,1	68,2	—	—	—	—	—
		80	80	45,4	45,4	68,1	90,8	113,5	—	—	—	—
		100	100	56,9	56,9	85,3	113,8	142,3	170,7	—	—	—
		120	120	68,3	—	102,4	136,6	170,8	204,9	273,2	—	—
	0,8	120	100	77,5	77,5	116,5	155,0	193,8	232,5	310,0	—	—

The length of the tables of corrugated iron varies between 3 and 6 m according to the thickness of the plate, the breadth between 0,45 and 0,95 m, the length of the covering is usually 100 mm.

Table I, p. 113a.

XIV. Pillars.

Safe Loads for round cast iron pillars.



The thick line

20,0	2,5	5369	137	105						98	86	77	68	62	53	50	46	42
20,0	3,0	5908	160	122						109	96	85	76	68	61	56	51	46
20,0	3,5	6452	181	139						118	104	92	82	74	66	60	55	50
22,0	2,0	6343	126	96								90	81	72	65	59	54	49
22,0	2,5	7394	153	117								105	94	84	76	69	63	58
22,0	3,0	8300	179	137								118	105	95	85	77	70	65
22,0	3,5	9011	203	155								128	115	103	93	84	77	70
22,0	4,0	9608	226	174								136	122	110	99	90	82	75

(The loads are represented in tons. — 1 ton = 1000 kg.)

The loads here admissible, expressed in tons = 1000 kg are calculated by the Hypothesis generally in use, viz that both ends of the pillars are free but do not deviate from the axis alove and below.

Formula is:

$$P = \frac{\pi^2 \cdot T \cdot E}{n \cdot l^2} = \frac{10 \cdot T \cdot 1\,000\,000}{6 \cdot l^2}$$

whence

$$T = \frac{P \cdot l}{1\,666\,000}$$

T = moment of inertia for round pillars $T = \frac{\pi}{64}(D^4 - d^4)$.

E = modulus of elasticity (cast iron 1 000 000).

P = load in kg.

l = height of pillars in cm.

n = Factor of Safety = 6.

1—2 m must be allowed for the weight at the capital and at the base.

Table II, p. 113b.

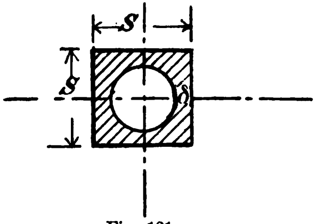


Fig. 121.

Safe Loads for square cross sections with circular hollow interior.

S = length of sides } in cm,
δ = strength of wall } in cm,
T = moment of inertia,
F = cross section in qcm,
G = weight for 1 m in kg.

and security against breaking.

S	δ	T	F	G	Height of pillars in m 2,0	2,2	2,4	2,6	2,8	3,0	3,2	3,4	3,6	3,8	4,0	4,2	4,4	4,6	4,8	5,0
8.	1,0	277	36	26	11,5	9,5	8	6,8	5,9	5,1	4,5	4	3,5	3,2	2,9					
8.	1,5	310	44	32	12,9	10,7	9	7,6	6,6	5,7	5	4,5	4,0	3,6	3,2	2,9	2,6	2,4	2,2	2,0
10.	1,0	632	50	36	25	21,8	18,3	15,5	13,4	11,7	10,3	9,1	8,1	7,3	6,6	6	5,5	5,0	4,6	4,2
10.	1,5	715	62	45	29,8	24,6	20,7	17,6	15,2	13,2	11,6	10,3	9,2	8,2	7,4	6,7	6,1	5,6	5,1	4,7
12.	1,5	1406	80	58	40	40	40	34,6	29,9	26	22,9	20,3	18,0	16,2	14,6	13,3	12,1	11,0	10,1	9,3
12.	2,0	1527	94	69	47	47	44,2	37,6	32,4	28,3	24,5	22,0	19,6	17,6	15,9	14,4	13,1	12,0	11,0	10,2
12.	2,5	1610	106	77	53	53	46,6	39,7	34,2	29,8	26,2	23,2	20,7	18,6	16,8	15,2	13,9	12,6	11,6	10,7
14.	1,5	2482	101	73				50,5	50,5	56	40,0	35,7	31,9	28,6	25,8	23,4	21,3	19,5	18,0	16,4
14.	2,0	2710	118	86			59	59	57,6	50,4	44,1	39,0	34,8	31,3	28,2	25,6	23,3	21,3	19,6	18
14.	2,5	2879	132	96			66	66	61,2	53,3	46,9	41,5	37,0	33,2	30,0	27,2	24,8	22,7	20,8	19,2
16.	1,5	4059	123	90						61,5	61,5	58,5	52,2	46,8	42,3	38,3	35	32	29,4	27,1
16.	2,0	4443	143	104						71,5	71,5	64,0	57,1	51,3	46,3	41,9	38,2	35	32,1	29,6

Table of the principle dimensions of plate beams.[1]

(For the cross section see p. 15 fig. 15.)

Width of support	Height of beam = $^1/_8$ width of support				Height of beam = $^1/_{10}$ width of support				Height of beam = $^1/_{12}$ width of support			
	4 L-iron	Vertical web	chord	Moment of resistance	4 L-iron	Vertical web	chord	Moment of resistance	4 L-iron	Vertical web	chord	Moment of resistance
m	mm	mm	mm	cm³	mm	mm	mm	cm³	mm	mm	mm	cm³
2,5	80/80 10	10	—	765	100/100 12	11	—	780	—	—	—	—
3,0	80/80 10	10	—	985	100/100 12	11	—	1010	90/90 11	13	200/13	980
3,5	90/90 11	10	—	1440	100/100 12	11	—	1350	90/90 11	13	220/13	1300
4,0	90/90 11	10	—	1730	80/80 10	10	200/10	1575	90/90 11	12	220/13	1560
4,5	90/90 11	10	—	2035	80/80 10	10	200/10	1760	90/90 11	12	220/13	1840
5,0	90/90 11	10	—	2350	80/80 10	10	200/10	2130	90/90 11	11	220/13	2110
5,5	70/70 9	10	180/8	2665	80/80 10	10	220/10	2530	90/90 11	10	220/18	2610
6,0	70/70 9	10	200/8	3130	90/90 11	10	220/10	3135	90/90 11	10	220/18	2940
6,5	70/70 9	11	200/8	3590	90/90 11	10	230/10	3550	90/90 11	10	240/18	3420
7,0	70/70 9	11	200/8	3975	90/90 11	10	230/10	3920	100/100 12	10	240/18	3970
7,5	80/80 10	11	200/8	4870	100/100 12	10	230/10	4715	100/100 12	10	240/20	4650
8,0	80/80 10	11	200/8	5335	100/100 12	11	230/10	5240	100/100 12	10	250/20	5190
8,5	80/80 10	11	200/8	5810	100/100 12	11	230/10	5680	100/100 12	10	250/20	5610
9,0	80/80 10	11	200/8	6300	100/100 12	11	240/10	6220	100/100 12	10	250/21	6136

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[1] Steiner, Konstruktion der eisernen Balkenbrücken.

Moments of inertia and resistance for hollow rectangular columns.

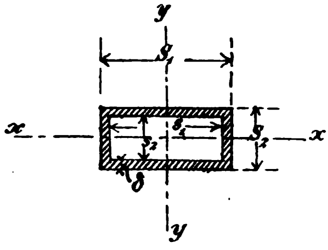

Fig 122.

S = length of sides in mm
δ = strength of wall in mm
F = cross section in qcm
G = weight pro m in kg
T = moment of inertia
W = moment of resistance

S_1	S_2	δ	F	G	T_x in cm	T_y in cm	W_x in cm	W_y in cm
200	**130**	10	62	46,5	1665	3321	256	332
200	**130**	15	93	69,8	2245	4573	345	457
200	**130**	20	124	93,0	2690	5595	415	559
200	**130**	25	155	116,3	3022	6417	465	642
200	**130**	30	186	139,5	3262	7066	502	707
250	**130**	10	72	54,0	2026	5774	312	462
250	**130**	15	108	81,0	2744	8054	422	644
250	**130**	20	144	108,0	3301	9981	508	800
250	**130**	25	180	135,0	3724	11594	573	928
250	**130**	30	216	162,0	4036	12926	621	1034
300	**130**	10	82	61,5	2387	9127	367	608
300	**130**	15	123	92,3	3243	11848	499	857
300	**130**	20	164	123,0	3913	16068	602	1071
300	**130**	25	205	153,8	4426	18833	681	1256
300	**130**	30	246	184,5	4807	21186	739	1412

The **values are calculated** by the formula:

$$W_x = \frac{S_1 \cdot S_2^3 - s_1 \cdot s_2^3}{6 \cdot S_2} \qquad T_x = \frac{S_1 \cdot S_2^3 - s_1 \cdot s_2^3}{12}$$

$$W_y = \frac{S_2 \cdot S_1^3 - s_2 \cdot s_1^3}{6 \cdot S_1} \qquad T_y = \frac{S_2 \cdot S_1^3 - s_2 \cdot s_1^3}{12}$$

Pillars composed of two pieces of I-Iron joined together by flat-iron.

Normal Profile No.	Dimensions in mm					Cross section F	Weight for 1 m = G	Moment of resistance $T_x = T_y$
	h	b	d	t	a	qcm	kg	cm
8	80	42	3,9	5,9	61	15,22	13,2	156,8
9	90	46	4,2	6,3	69	18,10	16,7	236
10	100	50	4,5	6,8	77	21,38	19,3	344
11	110	54	4,8	7,2	85	24,72	22,2	482
12	120	58	5,1	7,7	93	28,54	25,4	662
13	130	62	5,4	8,1	100	32,38	28,7	882
14	140	66	5,7	8,6	108	36,70	32,3	1158
15	150	70	6,0	9,0	116	40,10	36,0	1486
16	160	74	6,3	9,5	124	45,80	40,0	1890
17	170	78	6,6	9,9	128	50,8	44,0	2354
18	180	82	6,9	10,4	140	56,0	48,6	2920
19	190	86	7,2	10,8	147	61,4	53,0	3558
20	200	90	7,5	11,3	155	67,4	57,7	4324
21	210	94	7,8	11,7	163	73,2	62,5	5174
22	220	98	8,1	12,2	171	79,6	67,8	6180
23	230	102	8,4	12,6	178	85,8	73,4	7284
24	240	106	8,7	13,1	186	92,8	79,0	8576
26	260	113	9,4	14,1	202	107,4	91,0	11596
28	280	119	10,1	15,2	216	122,8	103,5	15316
30	300	125	10,8	16,2	232	138,8	116,5	19776
32	320	131	11,5	17,3	244	156,4	130,5	25244
34	340	137	12,2	18,3	262	174,4	145	31654
36	360	143	13,0	19,5	278	195,0	161,8	39532
38	380	149	13,7	20,5	292	215,0	177,8	48416
40	400	155	14,4	21,6	308	236,6	195,2	58892
42½	425	163	15,3	23,0	328	266,0	218,5	74532
45	450	170	16,2	24,3	346	295,4	242,2	92408
47½	475	178	17,1	25,6	366	327,2	267,5	113824
50	500	185	18,0	27,0	384	360,4	294	138490

Remarks:

The distance a is so determined that $T_y = T_x$. The strength of the flat-iron is purposely made equal to the strength of the web of I-iron.

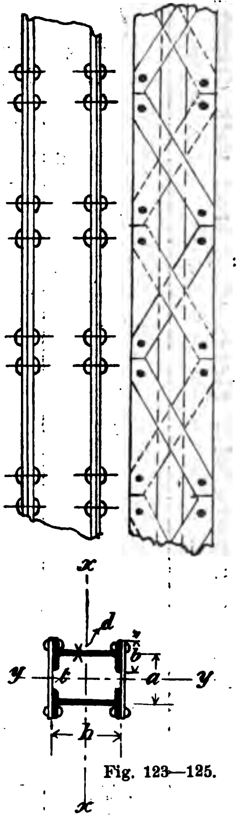

Fig. 123—125.

Pillars composed of 2 pieces of]-Iron with diagonal joints.

1—2 m must be allowed for the weight at the capital and base.

Normal Profile No.	Dimensions in mm				Moment of resistance T_y for a slight distance a between the]-iron in mm				Moment of resistance $T_x = T_y = T$ when the distance a is in mm	Cross section F	Weight G for 1 m
	h	b	d	t	5	10	15	20		qcm	kg
3	30	33	5	7	41,3	51	62,2	74,7	$a = 0$ $T = 13$	10,84	9,8[1]
4	40	35	5	7	50,8	62,2	75,1	89,6	$a = 0$ $T = 28,4$	12,40	11,1
5	50	38	5	7	62,5	75,7	90,6	107,3	$a = 1$ $T = 53,4$	14,24	13,0
6½	65	42	5,5	7,5	89,3	106,7	126,2	148,2	$a = 13$ $T = 116,4$	18,10	16,2
8	80	45	6	8	113,2	138,0	162,3	189,3	$a = 24$ $T = 214$	22,08	19,5
10	100	50	6	8,5	167,7	195,8	226,9	261,7	$a = 38$ $T = 414$	27,00	23,9
12	120	55	7	9	233,4	269,6	309,8	354,5	$a = 52$ $T = 736$	34,08	30,0
14	140	60	7	10	332,9	379,4	431,3	488,0	$a = 65$ $T = 1218$	40,8	35,7
16	160	65	7,5	10,5	441,1	489,5	562,1	631,5	$a = 78$ $T = 1864$	48,2	42,0
18	180	70	8	11	569,7	639,2	715,5	799,1	$a = 91$ $T = 2728$	56,0	48,7
20	200	75	8,5	11,5	729,6	812,9	904	1003,5	$a = 103$ $T = 3854$	64,6	55,7
22	220	80	9	12,5	955,9	1058,1	1169,2	1290,1	$a = 116$ $T = 5424$	75,2	64,6
26	260	90	10	14	1505,4	1648,6	1803,5	1971	$a = 141$ $T = 9714$	96,8	82,4
30	300	100	10	16	2332,2	2527,4	2737,9	2962,6	$a = 167$ $T = 16128$	117,6	99,5

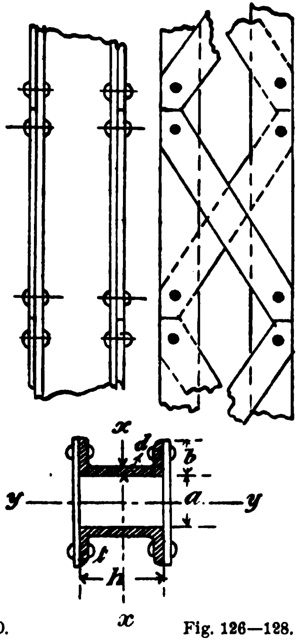

Fig. 126—128.

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[1] Taken exactly Ty is in both these cases greater than Tx, viz = 82,8 and 41,0.

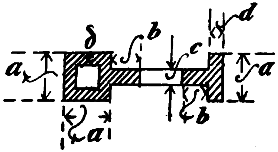


Fig. 129.

Cast-iron pedestals for bow windows.

T = moment of inertia } without b
F = flat surface } without b
G = weight pro m

				Strength of the wall δ																	
				12 mm			15 mm			18 mm			20 mm			25 mm			30 mm		
a	b	c	d	T	F	G	T	F	G	T	F	G	T	F	G	T	F	G	T	F	G
mm	mm	mm	mm	cm⁴	cm²	kg	cm⁴	cm²	kg	cm⁴	cm²	kg	cm⁴	cm²	kg	cm⁴	cm²	kg	cm⁴	cm²	kg
100	75	25	30	805	72	85	883	81	92	944	89	99	975	94	101						
110	75	25	30	1097	80	91	1212	90	99	1303	99	105	1353	105	110	1425	118	119			
120	75	25	40	1596	100	106	1757	111	114	1889	121	121	1963	121	121	2104	143	137	2196	156	147
130	75	25	40	2060	109	112	2279	121	121	2462	133	130	2565	140	135	2771	157	147	2912	172	158
140	75	25	40	2607	117	118	2896	131	128	3141	144	138	3283	152	144	3570	171	158	3775	188	170
150	75	25	40				3616	141	136	3936	155	146	4124	164	153				4791	197	177
160	75	25	50				4788	167	162	5198	182	165	5440	192	173						

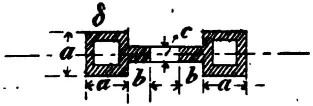


Fig. 130.

Cast-iron pedestals for bow windows.

T = moment of resistance } without *b*
F = flat surface } without *b*
G = weight pro m

a	b	c	Strength of the wall δ																	
			12 mm			15 mm			18 mm			20 mm			25 mm			30 mm		
			T	*F*	*G*	*T*	*F*	*G*	*T*	*F*	*G*	*T*	*F*	*G*	*T*	*F*	*G*	*T*	*F*	*G*
mm	mm	mm	cm⁴	cm²	env. kg	cm⁴	cm²	env. kg	cm⁴	cm²	env. kg	cm⁴	cm²	env. kg	cm⁴	cm²	env. kg	cm⁴	cm²	env. kg
100	75	25	1110	84	90	1266	102	104	1388	118	116	1450	128	122						
110	75	25	1528	94	94	1758	114	112	1940	132	126	2040	144	134	2224	170	154			
120	75	25	2040	104	106	2362	126	122	2626	146	136	2774	160	146	3056	190	168	3240	216	182
130	75	25	2656	114	112	3094	138	130	3460	162	148	3666	176	158	4078	210	178	4360	240	200
140	75	25	3384	122	118	3962	150	138	4452	176	158	4736	192	170	5310	230	192	5720	264	218
150	75	25				4982	162	148	5622	190	168	5998	208	176	6770	250	206	7344	288	234
160	75	25				6162	174	156	6982	204	174	7466	224	188	8482	270	222	9256	312	252
170	75	25										9160	240	200	10464	290	236	11480	336	270
180	75	25										11094	256	212	12736	310	250	14040	360	288
190	75	25										13282	272	222	15318	330	266	16960	384	304
200	75	25										15744	288	234	18230	350	280	20264	408	322
220	75	25										12546	320	258	25122	390	310	28120	456	358
240	75	25										28630	352	282	33576	430	338	37800	504	392
260	75	25													43750	470	368	49496	552	428
300	75	25																79704	648	498

Table I

Chart showing the weight of **Road Bridges**, expressed in tons per metre of breadth.

(according to Harkort).

Example:

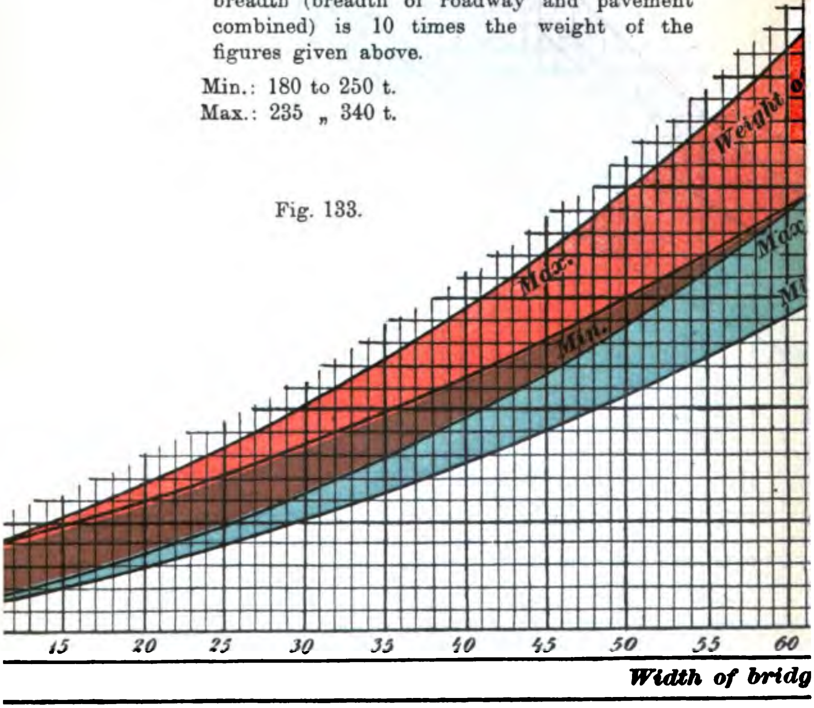
A bridge 70 m wide weighs:

Min.: 18 to 25 tons for every **1 metre** of breadth
Max.: 23½ „ 34 „ „ „ „ „ „ „

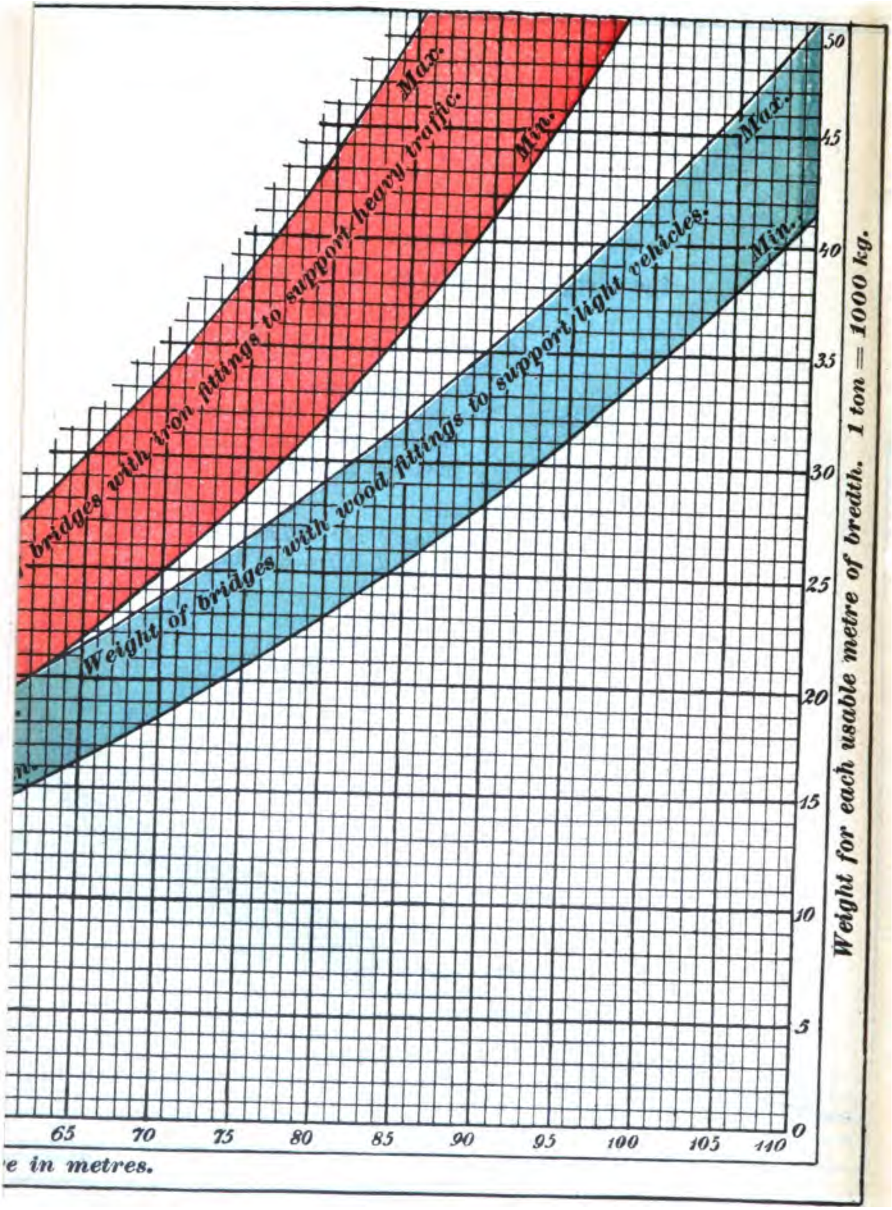
The Total weight of a Bridge with **10 m** breadth (breadth of roadway and pavement combined) is 10 times the weight of the figures given above.

Min.: 180 to 250 t.
Max.: 235 „ 340 t.

Fig. 133.



V, p. 119b.


Max.
bridges with iron fittings to support heavy traffic.
Min.
Max.
Weight of bridges with wood fittings to support light vehicles.
Min.
50
45
40
35
30
25
20
15
10
5
0
65
70
75
80
85
90
95
100
105
110
e in metres.
Weight for each usable metre of bredth. 1 ton = 1000 kg.

XV. Weights, Tables etc.

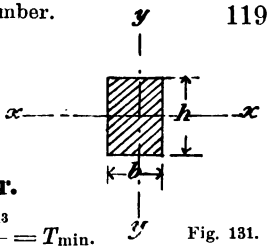


Fig. 131.

Tables for timber.

$$W_x = \frac{b\cdot h^2}{6} = W_{\max};\ T_y = \frac{h\cdot b^3}{12} = T_{\min}.$$

Dimensions in cm		Cross section F qcm	Moment of resistance W_x	Moment of inertia T_y	Dimensions in cm		Cross section F qcm	Moment of resistance W_x	Moment of inertia T_y
breadth b	height h				breadth b	height h			
6	8	48	64	144	15	26	390	1690	7312
8	8	64	85	341	16	16	256	683	5461
8	10	80	133	426	16	18	288	864	6144
10	10	100	166	833	16	21	336	1176	7168
10	11	110	202	917	16	24	384	1536	8192
10	12	120	240	1000	16	26	416	1803	8874
10	13	130	282	1083	16	29	464	2243	9899
11	18	198	594	1996	16	31	496	2563	10581
11	21	231	808	2329	18	18	324	972	8748
11	24	264	1056	2662	18	21	378	1323	10206
11	26	286	1239	2884	18	24	432	1728	11664
12	12	144	288	1728	18	26	468	2028	12636
12	15	180	450	2160	18	29	522	2523	14094
12	18	216	648	2592	18	31	558	2883	15066
12	21	252	882	3024	21	21	441	1543	16207
12	24	288	1152	3456	21	24	504	2016	18522
12	26	312	1352	3744	21	26	546	2366	20066
13	13	169	366	2380	21	29	609	2943	22381
13	16	208	555	2929	21	31	651	3363	23924
13	18	234	702	3295	24	24	576	2304	27648
13	21	273	955	3845	24	26	624	2704	29952
13	24	312	1248	4394	24	29	696	3364	33408
13	26	338	1465	4760	24	31	744	3844	35712
13	29	377	1822	5309	26	26	676	2929	38081
13	31	403	2082	5675	26	29	754	3644	42476
15	15	225	562	4219	26	31	806	4164	45405
15	18	270	810	5062	29	29	841	4065	58940
15	21	315	1102	5906	29	31	899	4645	63005
15	24	360	1440	6750	31	31	961	4965	76961

Specific weights of the principal materials.

Material	kg pro 1 cbm
Asphalt	1 500
Basalt	3 200
Concrete	2 000
Lead	11 400
Cement, broken	1 300
Chamotte stone	1 850
Oak	800
Ice	910
Iron framework per m³	250
Iron framework per m²	200
Peas	780
Earth and clay	1 600
Barley	640
Plaster, dry	1 000
Glass	2 600
Grass and clover	350
Granite	2 700
Cast iron	7 250
Oats	420
Hay	110
Trellis work in solid stone per m²	200
Trellis work in porous stone per m²	150
Charcoal	180
Chalk (broken)	1 300
Limestone masonry	2 400
Potatoes	730
Pinewood	650
Gravel	1 800
Copper	8 900
Marble	2 700
Flour	1 500
Dung	800
Rye	650
Rape	500
Sand (coarse)	1 400
Sandstone masonry	2 400
Slate	2 700
Steel	7 860
Coke	950
Cutstone	2 000
Straw	90
Wrought iron	7 800
Snow	125
Masonry	850
Turf	550
Peat moss litter	275
Tufaceous limestone	1 300
Water	1 000
Wheat	750
Zinc, smelted	6 860
„ rolled	7 200
Brick masonry	1 600
„ „ porous, or in markstone	1 100

Specific weights and loads,

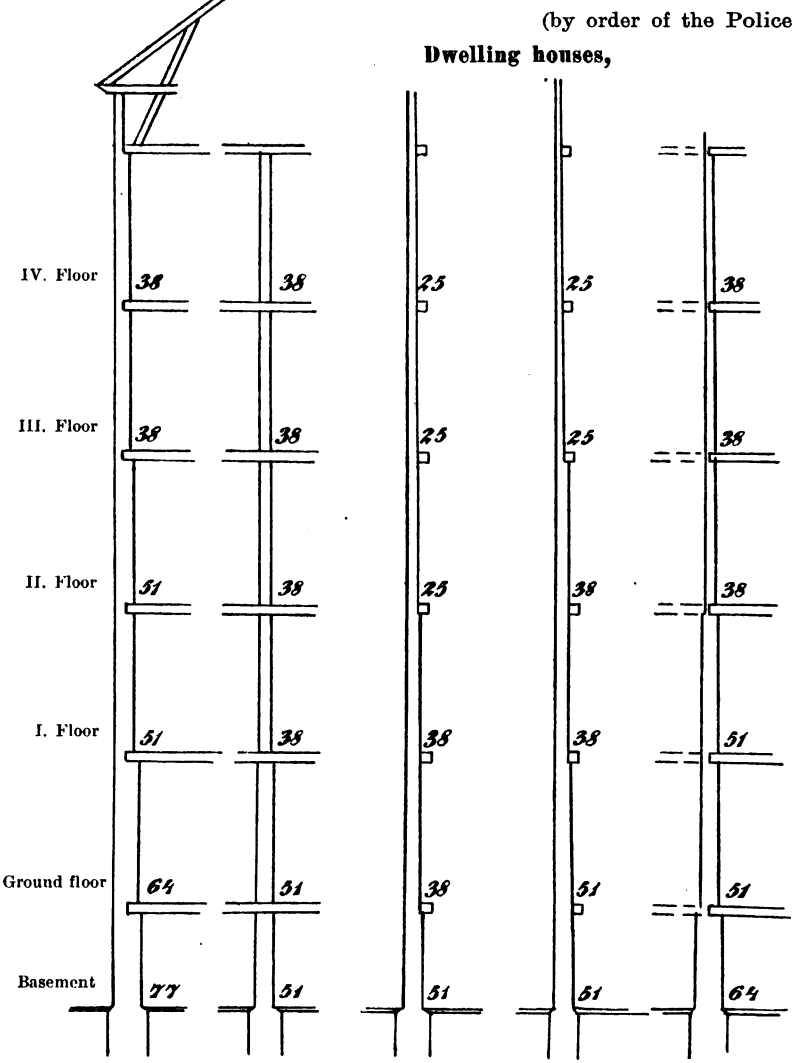
according to the police regulations for building, in Berlin, Frankfort o/M. etc.

Material	kg/qm
Framing of joists in dwelling houses	250
Do., including load	500
Framing of joists in factories and warehouses	250
Do., including load	750
Framing of joists in granaries including load, as shown . . .	850—1000
Framing of joists in salt storehouses . .	800
Entablatures for magazines above the Ist floor	1000
Entablatures for magazines above the ground floor	1500
Entablatures for magazines above the cellar	1700
Vaulted roofs of porous stone in dwelling houses	350
Do., including load	600
Do., roofs in factories,including load	1000
Roofs partly strengthened with stone, or concrete roofs. .	750
Vaulted roofs under passages or used thoroughfares,including load	1250
Do., including load in use	150
Sheet iron roofs including load as shown	500—1000
Balconies, including load in use	800
Vaulted staircases	500
Do., including load	1000
Roof levels calculated as projecting horizontally, including pressure of snow or wind, in metal or glass as desired	125—150
Do., with slate roof	200—240
Do., with tile roof .	250—300
Do., with wood and cement roof.	350
Mansard-roofs . . .	400
Frame-worked closings	230
Walls of tufaceous limestone	110
Burden of snow, horizontal projection	100
Windpressure . . .	100

The minimum thickness

(by order of the Police

Dwelling houses,



Front wall

Middle wall (loaded)

Gable wall without openings

Gable wall

High wall in dwelling houses, Loaded middle wall in factories

Fig. 134—138.

of walls to be used in

Building Administration, Berlin 1897).

Factories,

Staircase.

Note: Should the staircase be more than 3,50 m broad, it is strongly recommended to increase the thickness of the walls by 38 cm in the upper stories.

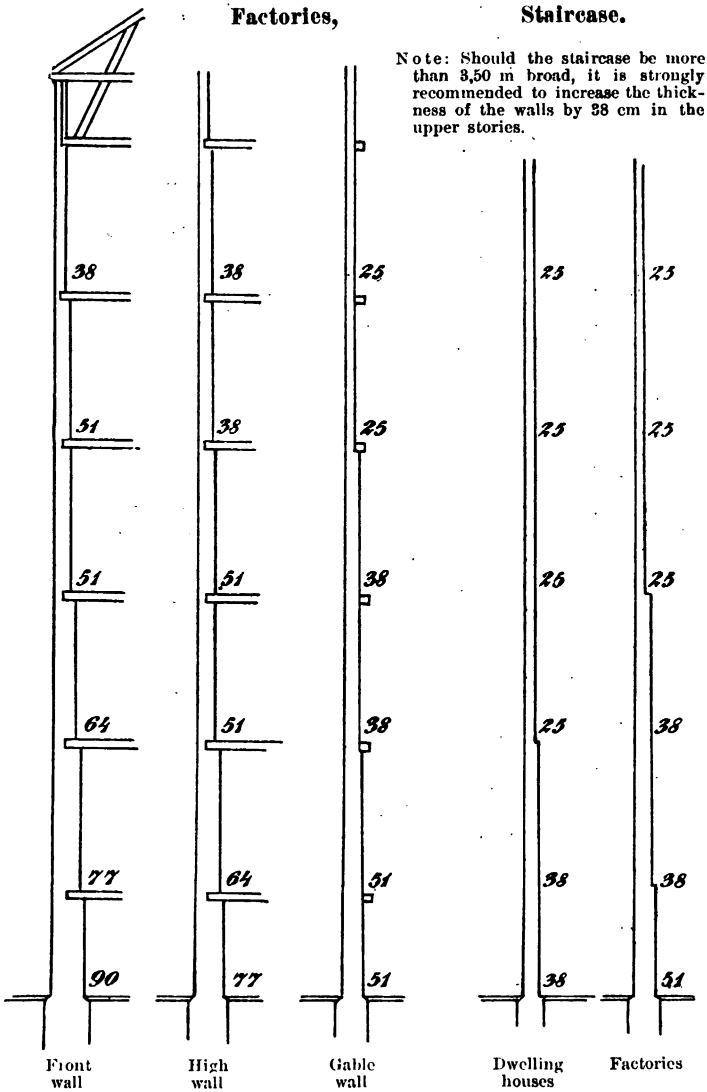


Fig. 139—143.

Working stress of building materials,

as ordered by the Police Building Administration in Berlin and Frankfort o/M. etc.

	Material	kg. per sqr. cm.		
		Tension	Pressure	Shear
Iron	Wrought iron	875	875	600
	Sheet iron	875	875	600
	Corrugated iron	500	500	—
	Iron wire	1200	—	—
	Cast iron	250	500	200
	Cast steel, hardened	3000	3000	2200
	Lead, rolled	30	100	—
	Wire ropes mine ropes	200	—	—
	cables	350	—	—
	Hemp ropes tackle	100	—	—
	cable ropes	135	—	—
Timber	Ash	100—120	66	20
	Oak	100	80	20
	Beech	100	80	20
	Pine	100	60	10
	Fir	60	50	10
Stones	Limestone masonry with chalk mortar	—	5	—
	Common brickwork do.	—	7	—
	Rudersdorfer limestone	—	25	—
	Glass	70—100	70—100	—
	Nebra sandstone red	—	15	—
	„ „ light	—	30	—
	Best brickwork with cement	—	12—14	—
	Porous vault bricks slightly baked	—	3	—
	„ „ „ hard baked	—	6	—
	Brohltal tufaceous limestone	—	6	—
	Cement stones	—	12	—
	Granite	—	45	—
	Marble	—	24	—
	Basalt	—	75	—
	Niedermendinger basalt lava	—	15—30	—
Earth	Slag and sand	—	12	—
	Good building ground	—	2,5	—

The Breaking point of some materials.

Material	kg. per sqr. cm.			
	Breaking point			Modul of elasticity for traction and pressure
	Traction K_1	Pressure K_2	Push T	E
Iron bars	3800	3500	3500	2 000 000
„ „ well hammered	—	—	—	2 000 000
Sheet iron	3500	3000	—	1 750 000
Iron wire	5600	—	—	2 000 000
Cast iron	1250	7500	2000	1 000 000
Unhardened common steel	7500	—	—	2 000 000
Hardened, treated „	—	—	—	2 000 000
Unhardened cast steel	—	—	—	2 000 000
Hardened, treated „	—	—	—	2 000 000
Finest spring steel	—	—	—	2 000 000
„ „ „, treated	—	—	—	2 500 000
Steel wire	11500	—	—	—
Sheet copper hammered	—	—	—	1 070 000
Sheet copper annealed	2100	4100	—	1 070 000
Copper wire	4200	—	—	1 210 000
Brass	1240	730	—	640 000
„ hammered	—	—	—	1 000 000
Brass wire	3650	—	—	987 000
Phosphor bronze	4040	—	3000	985 700
Zinc, cast	526	—	—	950 000
Lead	130	500	—	50 000
Lead wire	220	—	—	70 000
Tin	350	—	—	400 000
Aluminium	2030	—	—	675 000
Ash wood $\parallel$	1200	660	—	98 500
Ash wood $\perp$	—	350	—	—
Oak wood $\parallel$	1100	660	79	117 000
Oak wood $\perp$	50	350	—	—
Beech wood $\parallel$	1170	660	66	92 100
Beech wood $\perp$	73	350	—	—
Pine wood $\parallel$	1130	450	42	120 000
Pine wood $\perp$	48	220	—	—
Hemp rope	500	—	—	1000
Leather straps	300	—	—	1250
Glass	250	—	—	700 000

Rivets for constructions in iron.

Shapes of the rivet heads according to the Prussian Circular of Nov. 25th. 1891.

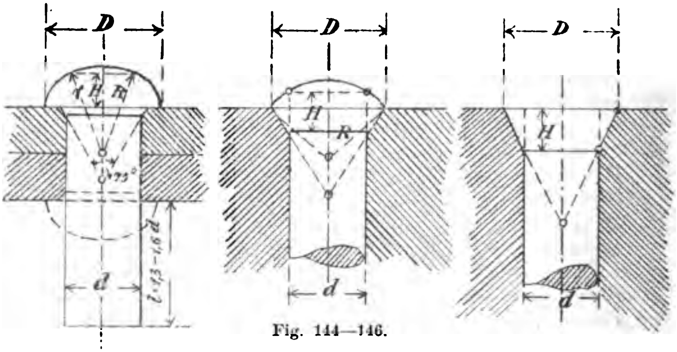


Fig. 144—146.

$$R = d;\quad r = \frac{d}{2};\quad H = 0{,}5\ d;\quad D = 1{,}5 \cdot d;\quad h = 1/8\ d.$$

Diameter of the shaft *d* mm	Cross section qcm	Diameter of the head *D* mm	Height of the sunken head *H* mm	Weight per 1000 rivets	
				round kg	sunken kg
10	0,79	15	3,75	4,52	3,64
12	1,13	18	4,5	7,82	6,29
14	1,54	21	5,25	12,41	9,98
16	2,01	24	6,00	18,53	14,90
18	2,54	27	6,75	26,38	21,21
20	3,14	30	7,5	36,19	29,10
22	3,80	33	8,25	48,17	38,73
24	4,52	36	9,00	62,54	50,28
26	5,31	39	9,75	79,51	63,93

The heads must be allowed a length of $l = 1{,}3$—$1{,}6\ d$, according to the **exactitude of the rivet holes.** In order that the rivet should fit into the hole without any trouble when it is warm, it is customary to **make the hole** 1 mm larger.

Number of rivets necessary for angle-irons.

(Cross section p. 100.)

Angle-iron		Rivets		Angle-iron		Rivets		Angle-iron		Rivets		Angle-iron		Rivets		Angle-iron		Rivets		Angle-iron		Rivets	
Normal Profile No.	Thickness mm	Diameter mm	Number required	Normal Profile No.	Thickness mm	Diameter mm	Number required	Normal Profile No.	Thickness mm	Diameter mm	Number required	Normal Profile No.	Thickness mm	Diameter mm	Number required	Normal Profile No	Thickness mm	Diameter mm	Number required	Normal Profile No.	Thickness mm	Diameter mm	Number required
3½	4	12	3	5	5	16	3	6½	7	20	3	8	8	20	5	11	10	26	5	14	13	26	8
3½	6	12	4	5	7	16	4	6½	9	20	4	8	10	22	5	11	12	26	6	14	15	26	9
—	—	—	—	5	9	16	5	6½	11	20	5	8	12	22	5	11	14	26	6	14	17	26	10
4	4	14	3	5½	6	18	3	7	7	20	4	9	9	22	5	12	11	26	6	15	14	26	9
4	6	14	3	5½	8	18	4	7	9	20	4	9	11	24	5	12	13	26	7	15	16	26	10
4	8	14	4	5½	10	18	5	7	11	20	5	9	13	24	6	12	15	26	8	15	18	26	11
4½	5	14	3	6	6	18	3	7½	8	20	4	10	10	24	5	13	12	26	7	16	15	26	10
4½	7	14	4	6	8	18	4	7½	10	20	5	10	12	26	5	13	14	26	8	16	17	26	12
4½	9	14	5	6	10	18	5	7½	12	20	6	10	14	26	6	13	26	26	9	16	19	26	13

Number of rivets necessary for flat-irons.

Flat-iron		Rivets		Flat-iron		Rivets		Flat-iron		Rivets		Flat-iron		Rivets		Flat-iron		Rivets		Flat-iron		Rivets	
Breadth *b*	Thickness *δ*	Diameter	Number	Breadth *b*	Thickness *δ*	Diameter	Number	Breadth *b*	Thickness *δ*	Diameter	Number	Breadth *b*	Thickness *δ*	Diameter	Number	Breadth *b*	Thickness *δ*	Diameter	Number	Breadth *b*	Thickness *δ*	Diameter	Number
mm	mm	mm		mm	mm	mm		mm	mm	mm		mm	mm	mm		mm	mm	mm		mm	mm	mm	
40	4	14	1	55	5	18	1	65	6	20	2	75	7	20	2	85	8	22	2	95	9	24	2
40	5	14	2	55	6	18	2	65	7	20	2	75	8	20	2	85	9	22	2	95	10	24	2
45	4	14	2	55	7	18	2	65	8	20	2	75	9	20	2	85	10	22	3	95	11	24	3
45	5	14	2	55	8	18	2	65	9	20	2	75	10	20	3	85	11	22	3	95	12	24	3
45	6	14	2	55	9	18	2	65	10	20	2	75	11	20	3	85	12	22	3	95	13	24	3
50	5	16	2	60	6	18	2	70	7	20	2	80	8	22	2	90	9	24	2	100	10	26	2
50	6	16	2	60	7	18	2	70	8	20	2	80	9	22	2	90	10	24	2	100	11	26	2
50	7	16	2	60	8	18	2	70	9	20	2	80	10	22	2	90	11	24	3	100	12	26	3
50	8	16	2	60	9	18	2	70	10	20	2	80	11	22	3	90	12	24	3	100	13	26	3
50	9	16	2	60	10	18	3	70	11	20	3	80	12	22	3	90	13	24	3	100	14	26	3

Number of rivets necessary for flat-iron.

Flat-iron		Rivets		Flat-iron		Rivets		Flat-iron		Rivets		Flat-iron		Rivets		Flat-iron		Rivets		Flat-iron		Rivets	
Breadth b mm	Thickness δ mm	Diameter mm	Number	Breadth b mm	Thickness δ mm	Diameter mm	Number	Breadth b mm	Thickness δ mm	Diameter mm	Number	Breadth b mm	Thickness δ mm	Diameter mm	Number	Breadth b mm	Thickness δ mm	Diameter mm	Number	Breadth b mm	Thickness δ mm	Diameter mm	Number
110	10	26	2	130	10	26	3	150	10	26	3	170	10	26	4	190	10	26	4	250	14	26	8
110	11	26	3	130	11	26	3	150	11	26	4	170	11	26	4	190	11	26	5	250	16	26	9
110	12	26	3	130	12	26	3	150	12	26	4	170	12	26	5	190	12	26	5	250	18	26	10
110	13	26	3	130	13	26	4	150	13	26	4	170	13	26	5	190	13	26	6	250	22	26	12
110	14	26	3	130	14	26	4	150	14	26	5	170	14	26	5	190	14	26	6	250	25	26	14
120	10	26	3	140	10	26	3	160	10	26	4	180	10	26	4	200	10	26	5	300	20	26	13
120	11	26	3	140	11	26	3	160	11	26	4	180	11	26	4	200	11	26	5	300	22	26	15
120	12	26	3	140	12	26	4	160	12	26	4	180	12	26	5	200	12	26	5	300	25	26	17
120	13	26	3	140	13	26	4	160	13	26	5	180	13	26	5	200	13	26	6	350	22	26	17
120	14	26	4	140	14	26	4	160	14	26	5	180	14	26	6	200	14	26	6	350	25	26	20

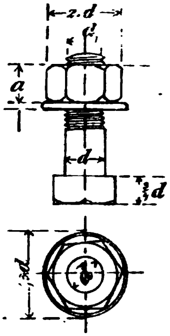


Fig. 147.

Screws.

Withworths Thread System (English measures).

Diameter of the bolts d		Diameter of the core d_1		Number of threads		$Q = 2{,}2\ d_1^2$	Height of nut rounded off	Height of head rounded off	Width of key rounded off
Inches	mm	Inches	mm	for 1 inch	for the length d	kg	mm	mm	mm
$\frac{1}{4}$	6,3	0,186	4,72	20	5	48	6	4	13
$\frac{5}{16}$	7,9	0,241	6,09	18	$5\frac{5}{8}$	81	8	6	16
$\frac{3}{8}$	9,5	0,295	7,36	16	6	118	10	7	19
$\frac{7}{16}$	11,1	0,346	8,64	14	$6\frac{1}{8}$	164	11	8	21
$\frac{1}{2}$	12,7	0,393	9,91	12	6	215	13	9	23
$\frac{5}{8}$	15,9	0,509	12,92	11	$6\frac{7}{8}$	370	16	11	27
$\frac{3}{4}$	19,0	0,622	15,74	10	$7\frac{1}{2}$	542	19	13	33
$\frac{7}{8}$	22,2	0,733	18,54	9	$7\frac{7}{8}$	752	22	15	36
1	25,4	0,840	21,33	8	8	998	25	18	40
$1\frac{1}{8}$	28,6	0,942	23,87	7	$7\frac{7}{8}$	1253	29	20	45
$1\frac{1}{4}$	31,7	1,067	26,92	7	$8\frac{3}{4}$	1590	32	22	50
$1\frac{3}{8}$	34,9	1,162	29,46	6	$8\frac{1}{4}$	1900	35	24	54
$1\frac{1}{2}$	38,1	1,287	32,68	6	9	2350	38	27	58
$1\frac{5}{8}$	41,3	1,369	35,28	5	$8\frac{1}{8}$	2740	41	29	63
$1\frac{3}{4}$	44,4	1,494	37,84	5	$8\frac{3}{4}$	3140	44	32	67
$1\frac{7}{8}$	47,6	1,591	40,38	$4\frac{1}{2}$	$8\frac{7}{16}$	3590	48	34	72

Diameter of the bolts d		Diameter of the core d_1		Number of threads		$Q = 2{,}2\, d_1^2$	Height of nut rounded off	Height of core rounded off	Width of key rounded off
Inches	mm	Inches	mm	for 1 inch	for the length d	kg	mm	mm	mm
2	50,8	1,716	43,43	$4\frac{1}{2}$	9	4140	51	36	76
$2\frac{1}{4}$	57,1	1,930	49,02	4	9	5280	57	40	85
$2\frac{1}{2}$	63,5	2,180	55,37	4	10	6750	64	45	94
$2\frac{3}{4}$	69,8	2,384	60,45	$3\frac{1}{2}$	$9\frac{5}{8}$	8030	70	49	103
3	76,2	2,634	66,80	$3\frac{1}{2}$	$10\frac{1}{2}$	9800	76	53	112
$3\frac{1}{4}$	82,5	2,857	72,57	$3\frac{1}{4}$	$10\frac{9}{16}$	11600	83	58	121
$3\frac{1}{2}$	88,9	3,107	78,92	$3\frac{1}{4}$	$11\frac{3}{8}$	13700	89	62	130
$3\frac{3}{4}$	95,2	3,323	84,40	3	$11\frac{1}{4}$	15700	95	67	138
4	101,6	3,573	90,75	3	12	18100	102	71	147
$4\frac{1}{4}$	107,9	3,805	96,65	$2\frac{7}{8}$	$12\frac{7}{32}$	20600	108	76	156
$4\frac{1}{2}$	114,3	4,055	103,00	$2\frac{7}{8}$	$12\frac{15}{16}$	23300	114	80	165
$4\frac{3}{4}$	120,6	4,285	108,84	$2\frac{3}{4}$	$13\frac{1}{16}$	26100	121	85	174
5	127,0	4,535	115,19	$2\frac{3}{4}$	$13\frac{3}{4}$	29200	127	89	183
$5\frac{1}{4}$	133,3	4,790	121,67	$2\frac{5}{8}$	$13\frac{25}{32}$	32600	133	93	192
$5\frac{1}{2}$	139,7	5,020	127,51	$2\frac{5}{8}$	$14\frac{7}{16}$	36000	140	98	201
$5\frac{3}{4}$	146,0	5,238	133,05	$2\frac{1}{2}$	$14\frac{3}{8}$	39000	146	102	209
6	152,4	5,488	139,40	$2\frac{1}{2}$	15	43000	152	106	118

Calculation of the Screws.

(Groves System.)

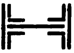
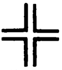
Without twisting of the bolts:

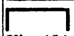
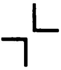
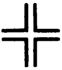
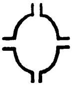
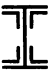
$$P = 600\ (d_1 - 0{,}2\ \text{cm})^2 \frac{\pi}{4};$$

With twisting:

$$P = 360\ (d_1 - 0{,}2\ \text{cm})^2 \frac{\pi}{4}.$$

Cross sections for girders and frame work.

No.	Girders	Practical limit of the cross section cm²
1	 -chord Fig. 148.	400
	Strengthened with L -iron	650
2	 -chord Fig. 149.	440
	Strengthened	650
3	 -form Fig. 150.	230
	Strengthened with L -iron	850
4	 -chord Fig. 151.	2000

No.	Frame work bars	Practical limit of the cross section cm²
5	 flat-iron Fig. 152. (traction only)	30
6	 -iron Fig. 153.	56
	Strengthened with flat-iron	75
7	 -iron Fig. 154.	58
8	 -iron Fig. 155.	45
	with flat-iron	95
	 with 2 L-iron Fig. 156.	120
9	 Simplest form Fig. 157	115
	 Strengthened Fig. 158.	180
10	 -form Fig. 159.	240
11	 Form for the greatest span Fig. 160.	650

The Worlds greatest bridges.

1. Wooden Bridges.	Width of span l in m	Year of completion
Wiebeking Bridge, over the Regnitz at **Bamberg;** was faultily built and collapsed	72	—
Cascade Bridge, in America . .	83	—
Colossus Bridge, at Termount Philadelphia; burnt 1838 . .	104	1812
Limmat Bridge, Wettingen in Argau, Switzerland; burnt by the French 1799	119	1778
2. Stone Bridges.		
Viaduct over the Göltzschtal 73 m high	31	—
Nydeck Bridge, Bern, Switzerland	45	—
Marne Bridge, Nogent, France .	50	1855
Vielle-Brionde, Departement Upper Loire France	56	1454
Dee Bridge, Chester	61	—
Cabie John, Tide Bridge, Washington, America	67	—
Adda Bridge, Trezzo, Italy; destroyed in war 1427	72	1355 to 1385
Street Bridge, over the Petrustal Luxembourg, widest spanned vaulted bridge in the world .	85	1902

3. Iron Bridges.	Width of span l in m	Year of completion
Arcole Bridge, Paris	80	1855
Rhine Bridge, Mainz	105	1860
Rhine Bridge, Coblenz	106	1876
Allier Bridge, Brionde, France .	115	1883
Weichsel Bridge, Dirschau . .	121	1850
Tarno Bridge, Saltash	139	1854
Britannia Bridge, England . .	140	1846
Leck Bridge, Kuilenburg . . .	150	1867
St. Louis Bridge, over the Missouri, America	157	1877
Douro Bridge, Oporto, Portugal .	160	1876
Bridge at Poughkeepsie over the Hudson River, America . . .	160	—
Garabit Bridge, St. Flour . . .	165	1880
Covington Bridge, over the Ohio Cincinnati, America	168	1887
Kaiser Wilhelm Bridge, over the Wupper at Müngsten, bow shaped bridge without joints 465 m long	170	1896
Kentucky and Indiana Bridge, America	170	1881
Menai Bridge, Suspension Bridge	176	—
Colorado Bridge, America . . .	201	1889
Chain Bridge in Budapest . . .	202	
Chain Bridge in Bristol	214	1862
Sukkur Bridge	241	1886

3. Iron Bridges (continuation).

	Width of span l in m	Year of completion
Street Bridge over the Monogahela at Pittsburg, bridge made of stiffened chains	244	1875
Railway Bridge over the Niagara Falls, America, wire-rope bridge	250	—
Clifton Bridge over the Niagara Falls America	385	1869
Elbe Bridge, Hamburg, supported on pillars	420	—
East River Bridge, between New York and Brooklyn, wire-rope bridge	518	1870
Bridge over the Firth of Forth .	521	—
Hudson Bridge, New York, stiffened suspension bridge . . .	930	—

	whole length in m	
Railway Bridge at Graudenz . .	1092	—
Railway Bridge at Thorn . . .	1272	—
Syzran Bridge over the Volga, Russia	1438	—
Moerdy Bridge, Holland . . .	1470	—
Bridge over the Firth of Tay, Dundee.	3200	—

Litterature.

Adams, Strains in Ironwork.

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Ruff, „Schnellstatiker“, Auskunftsbuch für statische Berechnungen.

Ruff, „Statique éclair“, Manuel de Renseignements pour Calculs statiques de constructions civiles. Vol. I.

Ruff, „Statique éclair“ pour les machines. Vol. II.

Notes for statical calculations.

Notes for statical calculations.

Notes for statical calculations.

Notes for statical calculations.

Notes for statical calculations.

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