Carrying capacity of trussed steel work

Autor(en): **Kohl, E.**

Objekttyp: **Article**

Zeitschrift: **IABSE congress report = Rapport du congrès AIPC = IVBH Kongressbericht**

Band (Jahr): **2 (1936)**

PDF erstellt am: **21.07.2024**

Persistenter Link: https://doi.org/10.5169/seals-3153

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Carrying Capacity of Trussed Steel Work.

Tragfähigkeit von Fachwerkträgern.

Résistance des poutres réticulées.

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Some fundamental questions relating to the strength of trussed steel constructions shall be explained by a simple example forming the basis of the following investigations.

1) *Fixed positions of loading.*

The truss according to fig. 1 represents a simply supported structure. The member U_3 in the bottom chord and the diagonals D_2 and D_5 are composed of ⅃L 70.70.7. All other members consist of ⅃L 90.90.9. In table 1 are shown the forces S_o in the members of the structure due to a concentrated load

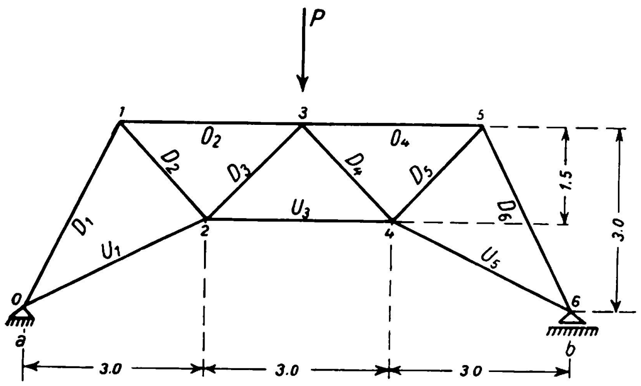

Fig. 1.

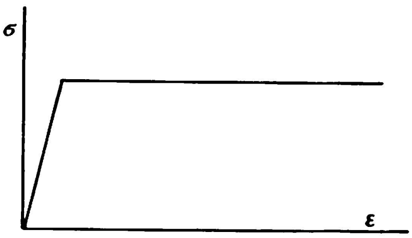

Fig. 2.

P acting in point 3, excluding the small influences due to dead weight of the structure. Considering a deduction for rivet holes of 20 mm diameter in tension members and based on a yield limit $\sigma_s = 2{,}4\ t/cm^2$ and buckling stresses σ_k according to the regulations of the German State Railways, the ultimate strengths of the members exhausting the carrying capacity of the members, are expressed by the following terms

$$S_{Gr} = F_n \cdot \sigma_s \text{ for members in tension,}$$

$$S_{Gr} = F \cdot \sigma_k \text{ for members in compression.}$$

The respective values are shown in the second colum in table I. The third row of this table shows the ultimate values of P_{Gr} of the load P which would, for an unrestricted validity of Hook's law, produce the ultimate values of strength in the members of the structure.

Table I.

Member	Force in members S_0	Ultimate Force S_{Gr} For tension	compression	Ultimate value of load P_{Gr}
U_1	+ 0,3727 P	+ 70 t	— 43,4 t	70/0,3727 = 188 t
U_3	+ 1,5 P	+ 41,8 t	— 19,5 t	41,8/1,5 = 27,9 t
O_2	— 1,0 P	+ 70 t	— 53 t	53 t
D_1	— 0,7453 P	+ 70 t	— 43,4 t	43,4/0,7543 = 58,2 t
D_2	+,0,9427 P	+ 41,8 t	— 39 t	41,8/0,9427 = 44,4 t
D_3	— 0,707 P	+ 70 t	— 69,8 t	69,8/0,707 = 98,6 t

The carrying capacity of the structure is ruled by the smallest value of $P_{Gr} = 27{,}9$ t for which load the yield point is reached first by member U_3. Assuming pin-jointed connections for all intersection points and accepting the stress-strain diagram of fig. 2, the strength of the structure is completely exhausted if only one member is stressed up to yield point, as otherwise unlimited elongations of member U_3 would be possible without any increase in load.

Through the introduction of a tie Z connecting the two bearing points a and b the play of forces and with it the strength of the structure are completely altered. According to the rules for hyperstatic systems, we receive for the tensile force in the tie, if S_a indicate the forces in the members due to $X_a = -1$, in the isostatic system: $Z = X_a = \frac{\delta_{a\,o}}{\delta_{a\,a}}$.

The numerator in this equation assumes the value

$$EF_c \sum S_o S_a s \frac{F_a}{F} = 34.5 \text{ P}$$

and the denominator

$$EF_c \sum S_a^2 s \cdot \frac{F_c}{F} = 50.64 + l_z \cdot \frac{F_c}{F_z}$$

where l_z represents the length and F_z the section of the tie respectively.

The following cases shall receive consideration:

a) The cross section of the tie shall remain constant over the length of 9 m (fig. 3 a),

b) The tie shall be composed of two angles 70.70.7 for a—c and b—d and F_z shall be the cross section of the tie for c—d. Fig. 3 b.

Assuming also for hyperstatic systems the carrying capacity of the structure to be dependent on one member only if stressed to yield point, it is obvious that based on the cross sections adopted for the members of the structure, the tie is entirely responsible for the strength of the structure. This, provided that the cross section of the tie does not go beyond a certain fixed value. Within this range the strength of the structure depends entirely on the cross

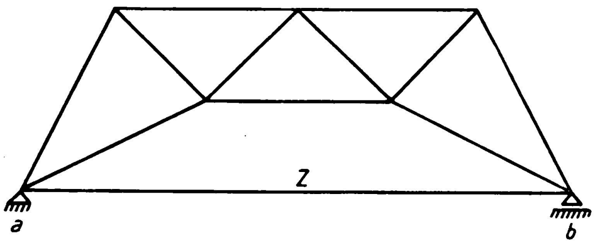

Fig. 3a.

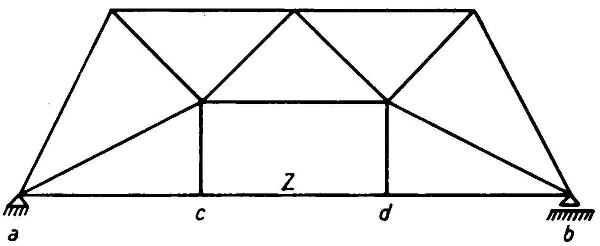

Fig. 3b.

section of the tie. This dependence of the ultimate load P'_{Gr} on the cross section F_z can be expressed by the following conditions:

$$Z = F_z \cdot \sigma_s{}^1 = \frac{34{,}5\, P'_{Gr}}{\frac{50{,}64 + l_z\, F_c}{F_z}} \text{ for case a}$$

$$Z = F_z \cdot \sigma_s = \frac{34{,}5\, P'_{Gr}}{\frac{60{,}54 + l_z\, F_c}{F_z}} \text{ for case b}$$

hence for $l_z = 9$ m and $l_z = 3$ m respectively and $F_c = 31$ cm^2 we receive

$$P'_{Gr} = 19{,}40 + 3{,}52\ F_z \text{ for case a} \qquad (1a)$$

$$P'_{Gr} = 6{,}47 + 4{,}21\ F_z \text{ for case b} \qquad (1b)$$

If a weak tie is chosen it will be seen that the carrying capacity of the isostatic truss can decrease because of the tie, as for $F_z \longrightarrow 0$; $P'_{Gr} \longrightarrow 19{,}4$ t respectively $P'_{Gr} \longrightarrow 6{,}47$ t compared with 27,9 t for the isostatic structure. Only if $F_z = 2{,}42$ cm^2 and $F_z = 5{,}1$ cm^2 respectively (fig. 5) does the statically indeterminate system with tie, assume the same strength as the corresponding structure without tie. The intended reinforcement of the structure by means of a tie proves in fact, apart from other deficiencies, to be a considerable weakening of the structure. It is, however, not reasonable, to think that the actual strength of the structure will fall below the strength of the isostatic system, provided the cross sections are kept unchanged. We have here an obvious contradiction which has its explanation only in the definition of the loading capacity of a structure.

The foregoing deductions lead to the conclusion that the carrying capacity

[1] No deduction is made for weakening of the cross section, the tie is assumed to be an aye bar.

of a once statically indeterminate system is by no means exhausted if a single member is stressed up to yield point; the same scale of judgment as regards the strength or ultimate loading capacity of isostatic systems cannot be applied for hyperstatic systems. After the yield point is reached by one superfluous member, the deformation depends entirely on the remaining members of the isostatic system. These deformations cannot go on growing indefinitely. Therefore, rupture or inadmissibly large deformations are not possible as long as at least one member of the remaining statically determinate system does assume yield stresses. Up to this point an increase in loading of the structure can be effected without endangering the safety of the structure. In general, as shown by *Grüning* in his wellknown treatise "The strength of hyperstatic systems in steel under consideration of frequently repeated loadings" the limit of the carrying capacity for an n-times statically indeterminate system is reached if at least n + 1 members are stressed up to yield point.

To define the actual limit of strength of the truss with tie it is necessary to determine the forces in the members of the isostatic system due to an exterior

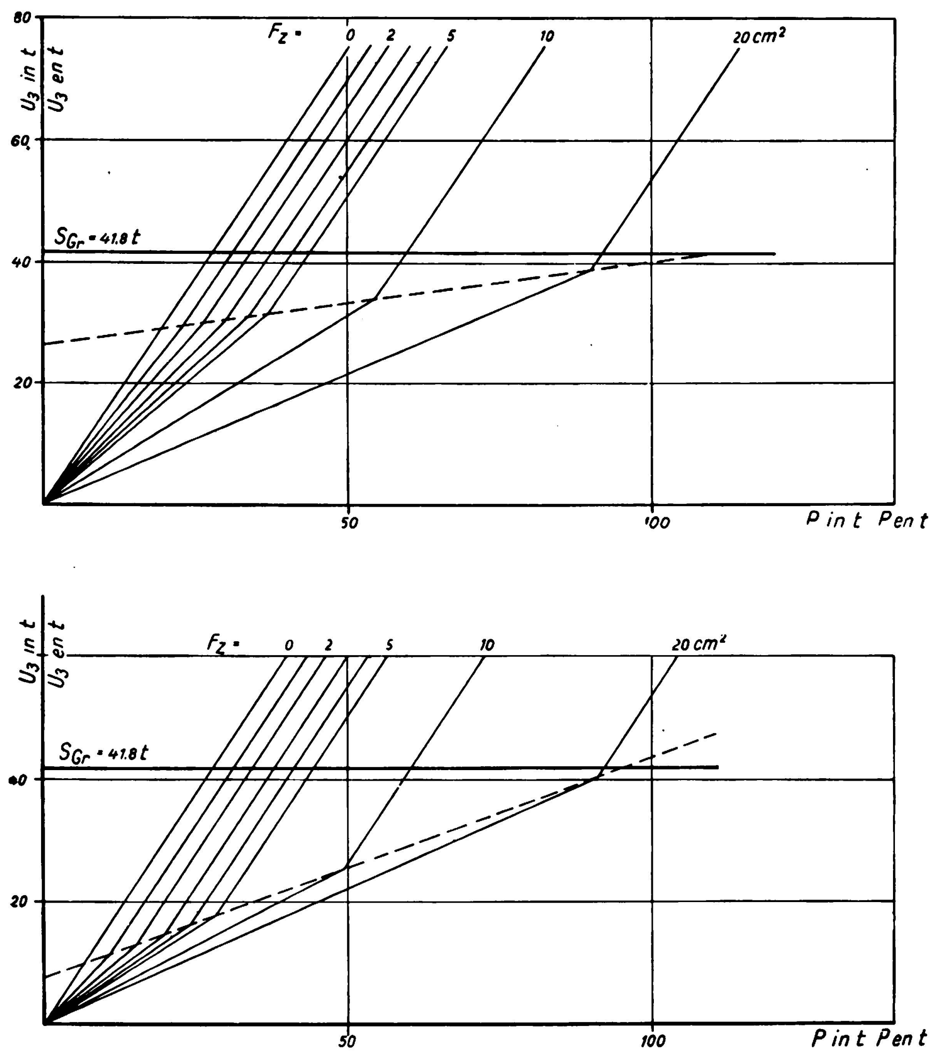

Fig. 4.

Force in member U_3 in relation to load P and cross sectional area of tie F_z.

load P. Under consideration of a constant force $F_z \cdot \sigma_s$ in the tie, but independent of P, the following general equation can be applied:

$$S = S_o - S_a \cdot F_z \cdot \sigma_s \tag{2}$$

Fig. 4 shows diagramatically the dependence of the force in member U_3, on P, for various values of F_z. Under the assumption of an unrestricted validity of Hook's law we should have

$$U_3 = 1.5\,P - 2 \cdot \frac{34.5\,P}{50.64 + l_z \cdot \dfrac{F_c}{F_z}}$$

At the moment when the tie is stretched, the forces in all members increase and for U_3 the following value would be obtained:

$$U_3 = 1{,}5\ P - 2 \cdot F_z \cdot \sigma_s = 1{,}5\ P - 4{,}8 \cdot F_z$$

The conditions for rupture are based on the following equations:

$$U_3 = 26{,}4 + 0{,}137\ P \quad \text{for case a}$$
$$U_3 = 7{,}36 + 0{,}362\ P \quad \text{for case b}$$

The ultimate strength of U_3 is 41,8 t; based on this value it is possible in fig. 4 to measure, for any value F_z, on the abscissae the ultimate strength which brings the stress in U_3 up to yield point.

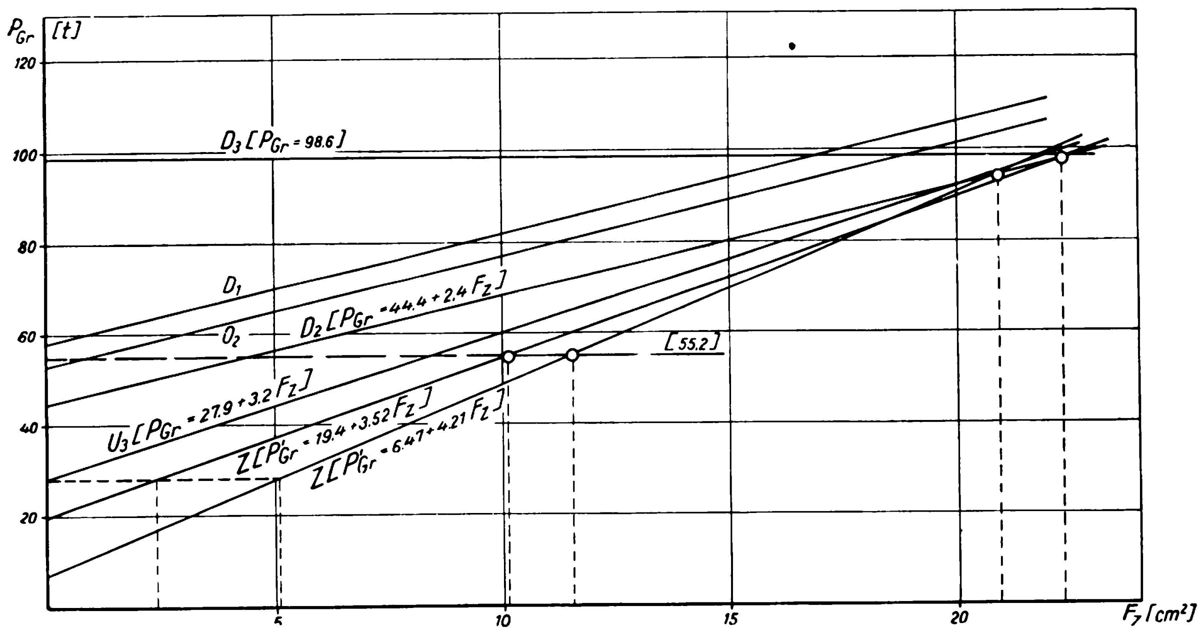

Fig. 5.

Ultimate loads P_{Gr} in relation to cross section, F_z of tie.

These conditions are shown better still and simultaneously for all members of the structure in fig. 5. The ultimate strength P_{Gr} for each particular member can be calculated by using the following formula

$$S = S_o - S_a \cdot F_z \cdot \sigma_s = S_{Gr}, \tag{3}$$

giving in relation to F_z the following values:

$$
\begin{array}{llll}
\text{for } U_1 & (S_a = +1{,}49067) & P_{Gr} = 188 & + 9{,}6\,F_z \\
\text{for } U_3 & (S_a = +2 \quad\quad) & P_{Gr} = 27{,}9 & + 3{,}2\,F_z \\
\text{for } O_2 & (S_a = -1 \quad\quad) & P_{Gr} = 53 & + 2{,}4\,F_z \\
\text{for } D_1 & (S_a = -0{,}7453\ ) & P_{Gr} = 58{,}2 & + 2{,}4\,F_z \\
\text{for } D_2 & (S_a = +0{,}9427\ ) & P_{Gr} = 44{,}4 & + 2{,}4\,F_z \\
\text{for } D_3 & (S_a = 0 \quad\quad\ ) & P_{Gr} = 98{,}6 &
\end{array}
\tag{4}
$$

The equations 4 apply for case a as well as for case b. These values of ultimate strength are shown in fig. 5 for all members of the system as a function of the cross sectional area of the tie. The ultimate strength increases for all members (not however, at the same rate) if the cross sectional area F_z of the tie increases, with the exception of D_3 for which the force is independent of the arrangement for the tie.

Up to a certain value of F_z we receive, apart from the tie, the smallest value of ultimate strength in member U_3 of the bottom chord, hence the carrying capacity of the structure is defined by the following equation:

$$P_{Gr} = 27{,}9 + 3{,}2\,F_z \tag{5}$$

which coincides for $F_z = 0$ with the isostatic system.

The same value of ultimate load $P_{Gr} = 93{,}9$ t is received for D_2 and U_3 if the cross section F_z measures 20,6 cm². For higher values of F_z and increased loading the yield point for member D_2 is reached earlier. The validity of equations 4 ceases for $F_2 = 22{,}3$ cm² in case a and for $F_z = 20{,}9$ cm² in case b respectively, as for $P = 97{,}9$ t and 94,5 t respectively the yield limit is reached simultaneously in the tie and in the diagonals D_2 and D_5. An increased carrying capacity or safety does not exist although assumed for hyperstatic systems. This fact cannot be altered by increasing the section of the tie as for $F_z > 22{,}3$ cm² and $> 20{,}9$ cm² respectively the two diagonals D_2 and D_5 will still be stretched. The remaining members therefore form a labile system. The relation between the carrying capacity P_{Gr} of the hyperstatic system and the ultimate load P'_{Gr} characterised by yielding of the tie, is given by the following terms:

$$\frac{27{,}9 + 3{,}2\ F_z}{19{,}4 + 3{,}52\ F_z} \text{ for case a, } \frac{27{,}9 + 3{,}2\ F_z}{6{,}47 + 4{,}21\ F_z} \text{ for case b.}$$

The results of these calculations are shown in fig. 6.

The property of hyperstatic systems called "self-help" or "stress distribution" does not develop in every case and under all circumstances; this could only occur under the condition of an excess of sectional area for particular members only. The present case does not give a noticeably increased value of carrying capacity due to section above 20 cm² for the tie, as a load of 90 to 95 t is simultaneously stressing several members of the structure up to yield point.

If the member D_3, whose force is independent of Z, were composed of 2 ∟ 70 · 70 · 7, the ultimate strength would be about 39 t, and the ultimate load 39/0,707 = about 55,2 t. But already with $F_z = 10{,}15$ cm² or 11,55 cm² respectively (fig. 5) the carrying capacity is exhausted, due to simultaneous buckling of the diagonals D_3 and D_4 and the tie being stressed to yield point.

an increase in section of the tie cannot increase the carrying capacity in this case, as the ultimate load $P_{Gr} = 55{,}2$ t remains decisive for D_3 and D_4.

It remains to be considered whether an increase in carrying capacity for the hyperstatic system is possible if a member in compression reaches the critical stress before any other member. Contrary to the member in tension, which in hyperstatic systems still remains a useful member even after the yield limit is reached,

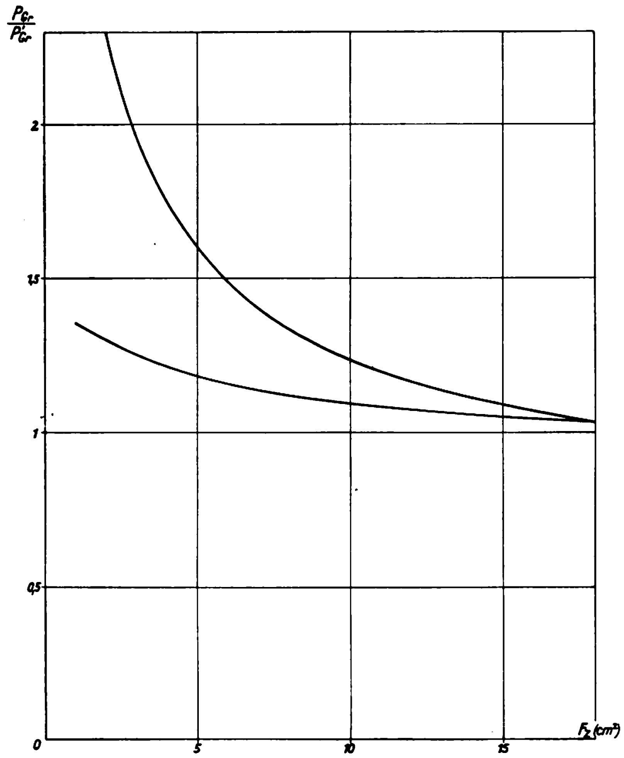

Fig. 6.

a member in compression is rendered useless at the moment of buckling. The buckling of a compression member takes place under the definite condition of bending and displacement of panel points, which can develop in isostatic systems without being influenced by other members. In hyperstatic systems the displacement of the ends of a member in compression is dependent on deformations of the remaining stable system having a tendency to counteract the bending of the buckling member. The condition of buckling for a compression member in a hyperstatic system is distinctly different from the condition of buckling for a single bar, these points have been explained by *Grüning* in his treatise, above mentioned.

2) *Variable positions of loading.*

In the following the case a load P acting at point 2 shall be examined with the object finding a reply to the question what will be the least section of the tie required to increase the carrying capacity of the isostatic system up to a definite value of P_{Gr} (= permissible load $\times$ safety factor). The calculation is based on the system shown in fig. 3b. The forces S_o of the structure without tie and the ultimate loads for each member are shown in table 2.

Table II.

member	Force in member	Ultimate load P_{Gr}
U_1	+ 0,4969 P	141 t
U_3	+ 1,0 P	41,8 t
U_5	+ 0,2485 P	282 t
O_2	— 1,333 P	39,7 t
O_4	— 0,666 P	79,5 t
D_1	— 0,9938 P	43,6 t
D_2	+ 1,257 P	33,25 t
D_3	+ 0,4713 P	148 t
D_4	— 0,4713 P	148 t
D_5	+ 0,6285 P	66,5 t
D_6	— 0,4969 P	87,2 t

The force in the tie has the following value

$$Z = X_a = \frac{29{,}57\ P}{60{,}54 + \frac{93}{F_z}}$$

The yield limit in the tie is obtained if

$$P'_{Gr} = 7{,}55 + 4{,}91 \cdot F_z \tag{6}$$

and as long as the tie represents the first and highest stressed member the ultimate stressing values for the members U_3, O_2, D_1 and D_2 are defined by the following equations (see fig. 7),

$$\begin{aligned} \text{for } U_3:&\quad P_{Gr} = 41{,}8 + 4{,}8\ F_z \\ \text{for } O_2:&\quad P_{Gr} = 39{,}7 + 1{,}8\ F_z \\ \text{for } D_1:&\quad P_{Gr} = 43{,}6 + 1{,}8\ F_z \\ \text{for } D_2:&\quad P_{Gr} = 33{,}25 + 1{,}8\ F_z \end{aligned} \tag{7}$$

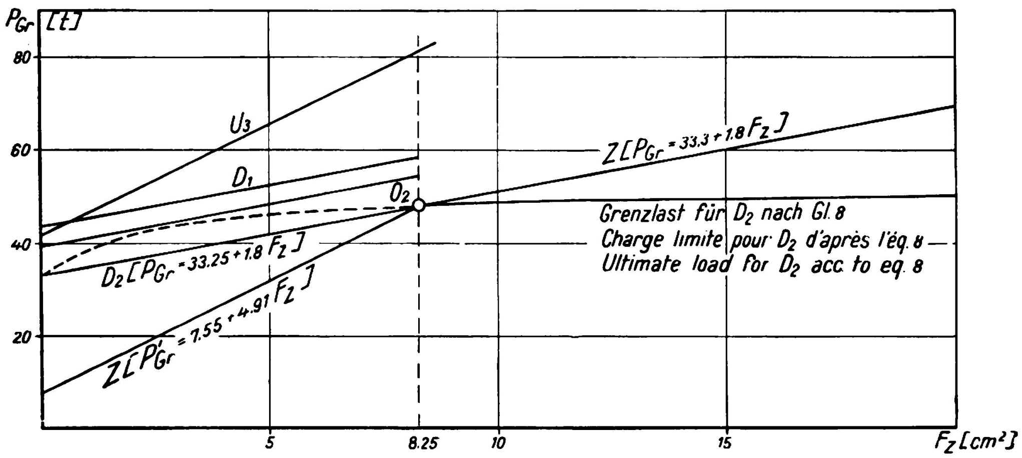

Fig. 7.

all other members do not require to be considered as their stresses are considerably below the ultimate values.

The yield limit is reached simultaneously in the tie for $F_z = 8{,}25$ cm² and in member D_2, due to a load $P = 48{,}05$ t. The carrying capacity is defined by the last of the equations 4, if $F_z < 8{,}25$ cm² and for $F_z > 8{,}25$ cm² the equations (7) become superfluous, on account of D_2 being the highest stressed member and as such yielding first. The ultimate load for D_2, as a function of F_z, in the hyperstatic system assumes the value

$$D_2 = 1{,}257\,P - 0{,}9427 \cdot \frac{29{,}57\,P}{60{,}54 + \frac{93}{F_z}} = S_{Gr} = 41{,}8$$

hence

$$P'_{Gr} = \frac{60{,}54\,F_z + 93}{1{,}15\,F_z + 2{,}8} \tag{8}$$

The results are diagramatically shown in fig. 7. The ultimate load for D_2 changes but little with an increasing value of F_z; it reaches, for instance 48,05 t for $F_z = 8{,}25$ cm² and 50,4 t if $F_z = 20$ cm². The same conditions of equilibrium apply for the other members of the structure, if $F_z > 8{,}25$ cm² and from the moment when D_2 reaches the yield limit, as for the statically determinate system with load P in point 2 and force $F_{D_2} \cdot \sigma_s$ in points 1 and 2 under elimination of member D_2. These forces can be calculated from the "self-stressing conditions" superposing the forces S_o (see table II) using the following equation:

$$S = S_o + (F_{D_2} \cdot \sigma_s - D_{2o}) \cdot \frac{S_a}{D_{2a}} \tag{9}$$

With this equation the following quantities are received for the various members:

$$\begin{aligned}
U_1 &= +\,66 - 1{,}49\,P \\
U_3 &= +\,88{,}8 - 1{,}666\,P \\
U_5 &= +\,66 - 1{,}74\,P \\
O_2 &= -\,44{,}4 \\
O_4 &= -\,44{,}4 + 0{,}666\,P \\
D_1 &= -\,33{,}2 \\
D_2 &= +\,41{,}8 \\
D_3 &= +\,0{,}4713\,P \\
D_4 &= -\,0{,}4713\,P \\
D_5 &= +\,41{,}8 - 0{,}6285\,P \\
D_6 &= -\,33{,}2 + 0{,}4969\,P \\
Z &= -\,44{,}4 + 1{,}333\,P
\end{aligned} \tag{10}$$

These values are independent of F_z. The validity of equation (10) starts for $F_z > 8{,}25$ cm² and $P \geqq P'_{Gr}$ according to equation (8), since the ultimate load for D_2 depends on the cross sectional area of the tie. The ultimate load at which the tie yields, and which therefore is a criterion for the carrying capacity, if $F_z > 8{,}25$ cm², results from the last equation of N° 10:

$$-44{,}4 + 1{,}33\ P_{Gr} = F_z \cdot \sigma_s$$

hence

$$P_{Gr} = 33{,}3 + 1{,}8\ F_z \tag{11}$$

The equations N° 10 allow calculation of the ultimate loads of the other members. The forces in O_2, D_1 and D_2 are constant and independent of P. The smallest values for P_{Gr} are received for members in the bottom chord and are:

for U_1 out of $+66 \quad -1{,}49\ P_{Gr} = -43{,}4$ hence $P_{Gr} = 73{,}5$ t
for U_3 out of $+88{,}8 - 1{,}666\ P_{Gr} = +41{,}8$ hence $P_{Gr} = 65$ t
for U_5 out of $+66 \quad -1{,}74\ P_{Gr} = -43{,}4$ hence $P_{Gr} = 63$ t

The external load P = 63 t therefore represents the highest load which the system can carry, whatever the respective cross section F_z of the tie, provided only that

$$33{,}3 + 1{,}8\ F_z \geqq 63$$
$$\text{or}\ \ F_z \geqq 16{,}5\ \text{cm}^2.$$

An increase in sectional area of the tie does not prevent the members D_2 and U_5 being stressed simultaneously to yield point, due to P = 63 tons.

The tie, for the purpose of solving the problem mentioned at the beginning, shall be dimensioned in such a way that the carrying capacity for the isostatic truss could be increased from 27,9 to 45 t or 60 t respectively. The dimensioning shall follow the principle of equal safety for all members, according to the ideas developed previously, if the external load shall act in point 3 as well as in point 2.

The required section of the tie F_z for an ultimate load of 45 t can be taken from fig. 5 or 7 or equation 5 or 7 respectively:

$$F_z = 5{,}35\ \text{cm}^2 \text{ for the load in point 3,}$$
$$F_z = 6{,}5\ \text{cm}^2 \text{ for the load in point 2,}$$

the higher value of the two to be used for dimensioning of the tie.

If the carrying capacity were be exhausted on the tie reaching the yield limit, the required section would be (according to fig. 5 and 7, equations 1b and 6) $F_z = 9{,}15$ cm² or $F_z = 7{,}65$ cm² respectively.

If the load is to be increased up to 60 t, the load in point 3 would demand a sectional aera of the tie of 10 cm² (compared with 12,7 cm² according to equation 1b) and the load in point 2 a section of $F_z = 40{,}8$ cm². This latter section is decisive. If the carrying capacity is considered as exhausted, due to a member being stressed up to yield limit, it will be found that an increase in the carrying capacity up to 60 t due to the arrangement of a tie as in the present case, would not be possible as for the load in position 2, the ultimate value for D_2 remains permanently less than 60 t according to equation (8). (See fig. 7.)

3) *The limits of carrying capacity due to deformations.*

The equations (4) for ultimate loads producing the ultimate forces in members are independent of the form of the tie as shown for instance in fig. 3a or 3b. The carrying capacity of such systems is identical for one and the same ultimate load causing the stress in the bottom chord member U_3, for a given value F_z, to reach the yield limit. The question requires to be considered whether the carrying capacity of both systems is the same and actually independent of the form of the tie. To solve this problem it is necessary to investigate the deformations of the system. Provided that only the tie is stressed up to yield limit, the deformations of the system depend entirely on the deformations of the members forming the remaining stable system. The displacement between a and b for system 3a can be expressed by the following term:

$$E F_c \Delta\, ab = \sum S \cdot S_a \cdot s \frac{F_c}{F}$$

wherein the summation applies for all members with the exception of the tie. For

$$S = S_o - S_a F_z \sigma_s$$

we receive

$$E F_c \Delta\, ab = \delta_{ao} - F_z \sigma_s \cdot \sum S_a^2 \cdot s \frac{F_c}{F}$$

$$E F_c \Delta ab = 34{,}5\ P - 121{,}5\ F_z.$$

If the load is increased in such a way as to produce the ultimate strength of U_3, we receive with $P = 27{,}9 + 3{,}2\ F_z$

$$E F_c \Delta ab = 963 - 11{,}1\ F_z$$

and with

$$E = 2100\ t/cm^2,\quad F_c = 31\ cm^2$$

$$\Delta ab = 1{,}48 - 0{,}017\ F_z\ \text{(in cm)} \tag{12}$$

Correspondingly we receive for system 3b with a similar calculation an elongation of c — d of:

$$\Delta cd = 1{,}48 - 0{,}0536\ F_z\ \text{(in cm)} \tag{13}$$

The permissible elongation within the range of yield, in other words the elongation for which rupture will occur, cannot be defined properly. As we are concerned only with fundamental deduction we are permitted to assume these values to 5 $^0/_{00}$ and we accordingly receive the permissible elongations of the tie as under

for $l_z = 9$ m $\quad \Delta l = 4{,}5$ cm
for $l_z = 3$ m $\quad \Delta l = 1{,}5$ cm

It will be seen that for the ultimate load, the actual displacement Δ a b or Δ c d respectively, previous to the yielding of U_3, is less than the permissible elongation within the range of yield. Therefore the deformation in both cases is of no importance as regards the carrying capacity of the systems.

The equation (4) would remain valid for a tie having a central portion composed of two ⌊ 70.70.7 over the length of 10 cm only and the displacements of the ends of the central portion of l_z would be for

$$\sum S_a^2\, s \frac{F_c}{F} = 65{,}33:$$

$$\Delta = 1{,}48 - 0{,}0713\ F_z\ \text{(in cm)}. \tag{14}$$

The permissible elongation within the range of yield is only 0,05 cm but is considerably exceeded due to deformations and other reasons before the member U_3 is stressed up to yield limit. Thus we have the possibility that a member being stressed first up to yield limit may fracture before another member becomes capable of taking over a portion of the load. A compensation of forces among the members cannot develop, unless incomplete.

The correctness of the statement has been proved by tests carried out with a continuous trussed girder in steel in the test house in Hannover[2].

The estimation of the carrying capacity of hyperstatic systems according to the ideas developed above is based on the assumption that the deformations of stretched members remain within certain limits of the range of yield, an assumption which can be considered as fulfilled in general.

4) *Temperature changes (elastic supports).*

Apart from the influence of load P acting in point 3 the variation of temperature from $\pm t^0$ shall be considered. The ultimate load P'_{Gr} of the tie will be influenced by such a change in temperature. In case of an increase or decrease in temperature of 15^0 respectively we receive for system 3 a

$$P'_{Gr} = 19{,}4 + 3{,}52\ F_z \pm 3{,}06.$$

The equations (4) still apply. The change in temperature influences only the values of deformations. A simple consideration shows that the deformations of stretched members can go on increasing indefinitely provided the tie is subject to repeated changes of temperature. The value of the load P remaining little under the ultimate load U_3, defined by equation 4, and assuming an increase in temperature of t^0 for the members of the system above the temperature of the tie, we find that the conditions of equilibrium remain unchanged. The tie is subject only to an additional elongation of $\varepsilon \cdot t \cdot l_z$. If the tie is subject to an increase in temperature, the stress in member U_3 will reach the yield limit and with the subsequent variation of temperature between $-t^0$ and $+t^0$ the tie and the member U_3 of the bottom chord will be subjected alternately to new additional elongations, which are not subject to restrictions. In this connection we wish to draw attention to the results obtained by tests carried out in Hannover[2] where the influences of uneven temperature in the chords of a girder were studied through the lifting and lowering of the extreme supports.

In case the temperature variations have to be considered, the values of deformations must be taken into account for judging the carrying capacity of a system. Elastic displacements of supports are of the same importance, but unelastic displacements (subsidence of the soil) are without influence on the carrying capacity.

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[2] *Grüning-Kohl:* Tragfähigkeitsversuche an einem durchlaufenden Fachwerkbalken aus Stahl. „Der Bauingenieur" 1933, p. 67/72 (Versuchsreihe II).

Summary.

The carrying capacity of a hyperstatic system is not dependent as a rule on the critical stress of a single member, as for isostatic systems. A few fundamental questions important for the judging of the carrying capacity have been explained by the example of a truss with tie. The actual ultimate load (safe actual load × safety factor) has been calculated for various positions of loads in relation to the cross sectional area of the tie. The investigations have revealed that an increased carrying capacity or safety does not always exist with hyperstatic systems as expected, compared with isostatic systems.

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