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II c

Adoption of High-Tensile Steel.

Anwendung von hochwertigem Stahl.

Utilisation des aciers à haute résistance.

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IIc 1

The Use of Steel of High Yield Stress Limit in Reinforced Concrete.

Anwendung von Stahl mit hochliegender Streckgrenze im Eisenbetonbau.

L'emploi de l'acier à haute limite d'écoulement dans le béton armé.

Ing. A. Brebera,
Sektionsrat im Ministerium für öffentliche Arbeiten, Prag.

The carrying capacity of reinforced concrete structures is not only dependent upon the quality of the concrete, but also upon the grip and quality of the reinforcing bars. In view of the fact that the use of good-quality cements and good aggregates produces high-grade concrete, an appropriate use of steel of high yield stress limit for reinforcing bars in reinforced concrete construction means, for the same degree of safety, an economic improvement of the reinforced concrete — an improvement that can only be welcomed in the interest of the public purse.

All reinforced concrete structures subjected to bending are calculated on the assumption of a definite ratio *"n"* between the elasticity of steel and that of concrete. When the steel reinforcement is stressed beyond yield limit, its co-efficient of elasticity drops in consequence of the great elongation produced, to such an extent that the ratio *"n"* is reduced to the vicinity of *1*. The distance of the neutral axis from the extreme compressive fibre is again dependent on *"n Fe"*, *"Fe"* being the cross-sectional area of the tensile bars. The stressing of the reinforcement beyond yield point therefore acts, apart from its tensile strength, in the same way as would a reduction in cross section of the reinforcement in the same proportion as a decrease in the value of the co-efficient of elasticity. However, the smaller the cross-sectional area of the reinforcement, the lower is the pressure zone and the greater the pressure exerted upon the concrete. On the displacement of the neutral axis towards the compressive fibre, however, the leverage with which the internal forces act becomes greater and the stresses exerted upon the steel are thus not increased to any great extent. When the yield limit has been passed considerably greater compressive stresses are produced in the concrete; these lead to failure (breakage) even when the steel is not stressed more than 10 %, or at the very most 20 %, beyond yield point. Only when concrete of comparatively high strength is used, or when elongation of the steel is not very great after the yield limit has been passed, will the steel stand up to still higher stresses. Thus it is always the height of the yield point, and not the tensile strength, that determines the permissible stresses and also, in consequences, the degree of safety in reinforced concrete structures.

Using ordinary C 38 mild steel, a yield limit of 2300 kg/cm² is now being guaranteed. With permissible stresses of 1200 or 1400 kg/cm², the degree of safety is therefore 1.92 to 1.64. Permanent elongation at the upper yield limit amounts to about 0.2 %.

For high-grade steels the yield limit and permissible stressing are corresondingly higher. The ultimate load for reinforced concrete beams, which is comparatively independent of compressive strength, is generally determined by the height of the yield limit of the steel used for reinforcing bars. When the first cracks appear considerably greater stresses in the steel have also to be calculated with. Beams reinforced with high-quality steel, in consequence of the small cross section of their reinforcement, undergo, however, considerably greater deflection than beams reinforced with ordinary C 38 steel of the same carrying capacity.

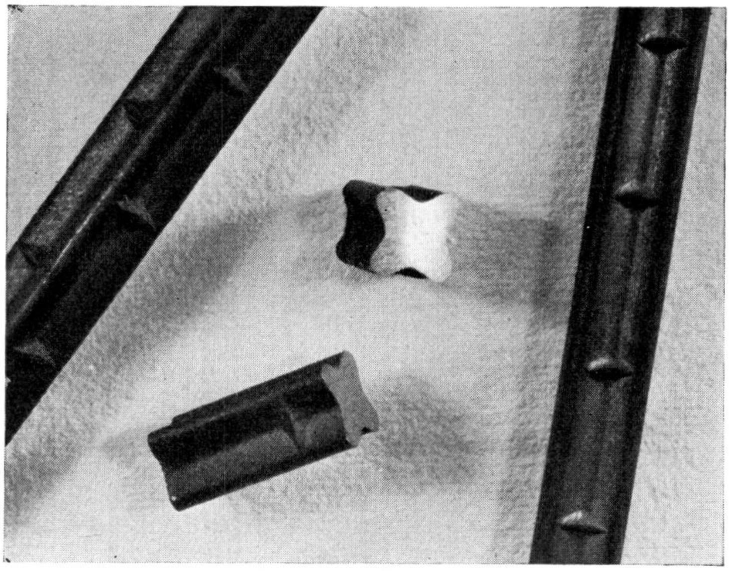

Fig. 1.

The raising of the permissible stresses in steel offers great advantages from a constructive as well as from an economic point of view. The advantages of higher permissible stressing in steel involve a reduction of the cross sections required, thereby decreasing the dead weight of reinforced concrete structures and permitting the construction of larger spans.

A high yield point in steel can be attained either in a natural manner *during production at the rolling mills,* or artificially *by cold stretch processing.*

A reinforcement bar belonging to the first of these two groups is the so-called "Roxor" type, which possesses a minimum yield limit of 3800 kg/cm². Greater grip is attained by means of cruciform cross section (Fig. 1), the surface of which is provided at intervals of about $1^1/_2$ times the maximum diameter with transverse ribs. The latter are so dimensioned that they are sufficiently impervious to injury when the bars are withdrawn from the concrete, and at the same time add as little as possible to the weight of the bars. The action of the ribs is to increase to a considerable extent the grip of the reinforcing bars in cooperation with the shearing strength of the concrete, for when the bars are withdrawn the space between two adjacent ribs remains filled with fine-grained concrete. At the same time the cross-section contour of these reinforcing bars is such as to prevent their being mistaken for any other type of steel.

The main data required for calculating the cross-sectional area and girth of these bars is the following:

Diameter of circumferential circle of "Roxor" bar

$$D = 1.2715\ d;$$

diameter of round bar equivalent to "Roxor" strength

$$d = 0.7856\ D;$$

girth of "Roxor" bar

$$U = 3.1106\ D = 3.9551\ d;$$

circumference of corresponding round bar

$$u = 2.4708\ D = \pi;$$

cross-sectional area of "Roxor" bar

$$F = 0.4816\ D^2 = 0.7786\ d^2.$$

By a round bar equivalent to "Roxor" strength is to be understood a round bar of the same weight per metre length. As the weight of the ribs amounts to 0.86 % of the total weight, however, the cross-sectional area of the corresponding round bar is 0.86 % greater than the actual cross section of the "Roxor" bar.

Mean values obtained by quality tests are classified in Table I:

Table I.

Quality test on reinforcing bar of	C 38 Steel	Roxor
Coefficient of elasticity in t cm^2	2050	2092
Yield point stress in kg/cm^2	2718	4037
Strength in kg/cm^2	3889	5259
Ratio of yield point stress to strength in %	70	77
Elongation	30	26
Constriction in %	64	55

The measured length in these tests amounted to ten times the diameter of the corresponding round bar. All test bars stood the cold bending tests carried out round a pin of the same diameter as that of the circumferential circle of the "Roxor" bar.

Comparative grip tests on "Roxor" reinforcing bars and others of ordinary C 38 steel were carried out by withdrawing the test bars from cubes of concrete in which they had been imbedded for periods of various lengths. When calculating grip strength it was assumed that the tensile stresses were evenly distributed throughout the whole length of the imbedded metal, whereby the circumference of the corresponding round bar was entered instead of the actual girth of the "Roxor" bar. The results of about 160 tests[1] made will be found in Table II.

[1] All tests and observations were carried out in the test house of the Czechoslovakian Institute of Technology in Prague under the guidance of the two engineers Prof. *F. Klokner* and Dr. *B. Hacar.*

Table II.

Cube strength of concrete		Grip strength of reinforcing bars in kg/cm² C 38 steel	Roxor
minimum 250 kg/cm²	min. value	42	59
	mean value	54	98
	max. value	68	161
minimum 350 kg/cm²	min. value	48	64
	mean value	69	121
	max. value	110	200

The above figures show that the grip strength of "Roxor" reinforcing steel is app. 80 % greater than that of ordinary C 38 steel bars. Calculating the actual girth of "Roxor" bars as 3.1106 D, the increase of grip strength amounts to about 43 %.

When testing ordinary C 38 steel bars, sliding was first observed at a tension somewhat less than half that of actual grip strength. In the case of "Roxor" reinforcing bars this took place at a tension somewhat below that of half grip strength; on the other hand, "Roxor" bars offered great resistance to further withdrawal.

By comparing the test results it was ascertained that grip strength

1) increases with the quality of the concrete,
2) increases with the setting time of the concrete,
3) decreases as the amount of water added is increased,
4) decreases as the imbedded length increases,
5) decreases as the diameter of the imbedded bar increases,
6) increases but little when stored dry as against storage under varying conditions. It therefore appeared that the conditions under which the test cubes were stored made little difference.

On the basis of 80 beam tests and subsequent calculations carried out in accordance with the respective regulations, it was ascertained that breakage was caused by stressing the steel reinforcement beyond its yield limit. In this connection beams whose reinforcement was not provided with hooks possessed the same carrying capacity as beams whose reinforcing bars were provided with the usual hooks. The total deflection of the beams reinforced with "Roxor" bars only amounted, under the same loading, to about 20 % more than that produced in beams reinforced with ordinary C 38 steel, though the reinforcement of the former was $^1/_3$ less. The elastic character of the steel was hereby transmitted for the greater part to the whole of the construction, so that the total deflections were for the main part composed of elastic deformations. Permanent deformations for beams reinforced with "Roxor" bars were approximately the same as those occurring in beams reinforced with C 38 steel.

Sixty-eight column tests, some for centric, others for eccentric pressure, were carried out and showed that "Roxor" reinforcing bars were considerably superior to C 38 steel bars. Thus it is possible to calculate for "Roxor" bars a correspondingly greater cross-sectional area, which comes to the same thing as increasing the permissible stresses. This enlargement factor can, with an adequate margin of safety, be put at 1.5; calculations for stressed sections can therefore be effected with 1.5×15 Fe = *22.5 Fe,* instead of the usual 15 Fe. Permissible stressing now remains the same as for C 38 steel reinforcement bars. The greater use made of the compressibility of concrete, however, makes stronger transverse reinforcement necessary.

If the factor of safety for C 38 steel with a permissible total stressing of 1400 kg/cm² is 1.64 or 1.94, according as the yield limit of 2300 kg/cm² guaranteed by the steel works or the effective yield limit of 2718 kg/cm² is calculated with, then permissible stressing works out at 2317 kg/cm² or 1960 kg/cm² for "Roxor" reinforcement bars, working with the same safety factors and a guaranteed yield limit of 3800 kg/cm². After the first cracks had appeared in the concrete, however — at approximately 850 kg/cm² for C 38 steel reinforcement and at about 1200 kg/cm² for "Roxor" bars — the *permissible tensile stressing* of "Roxor" reinforcement was determined at round *1900 kg/cm²*.

Consequently, in special cases — water reservoirs, for instance — absolute safety against cracking can be attained by a reduction of the permissible tensile stresses of the "Roxor" reinforcement bars to 1200 kg/cm²; for C 38 steel reinforcement this can only be effected by reducing the permissible tensile stressing to 850 kg/cm².

In consequence of their greater grip strength, "Roxor" reinforcement bars can also be employed *without hooks* and with a comparatively slight increase of the usual concreted length. This valuable property will particularly appeal to the designer for portions of the structure where an overcrowding with steel is liable to ensue.

"Roxor" reinforcement bars are rolled in strengths ranging from D = 8 mm to D = 70 mm and in lengths of 35 m and 25 m. The standard price per 100 kg works out on an average at Cz. Kr. 178, that of C 38 steel at Kr. 147.

Steel of high yield stress limit can be manufactured not only in the actual steel-works by a combination of suitable materials; it can also be produced mechanically from ordinary mild steel by cold stretch processing. This fact could for a long time not be utilised in reinforced concrete construction, because the single bar could not be stretched uniformly along its whole length and throughout its entire cross section. It only became possible to eliminate these deficiencies almost entirely througth the introduction of *Isteg Steel* (Fig. 2).

Two ordinary C 38 steel bars placed side by side with their respective ends rigidly clamped, are twisted cold by a special machine in such a manner that the pitch of the twist and the length of both bars remains constant, causing a certain amount of prestretching. The stretching takes place uniformly along the whole length of the individual bars and can be ascertained at any stage of the process from the pitch of the twist. As an elongation of the axis of the twisted bar does not ensue, the effective cross section of the latter is constant and equal to the total section of the two round bars before twisting took place. This process thus ensures even strength and uniformity throughout the material, and is at the same time a test of quality, since on inferior material this treatment would produce visible signs of damage.

A whole series of tests showed that this twisting process causes a raising of the yield stress limit by about 40 to 50 %, grip strength simultaneously increasing by app. 10 %. The coefficient of elasticity, however, decreases with the amount of twist applied, and for a pitch equivalent to 12.5 times the diameter of the single round bar works out at app. 80 % of the coefficient of elasticity of the straight rod. The ultimate yield elongation for "Isteg" Steel is about half that of ordinary C 38 steel. When judging the coefficients of elasticity of "Isteg" and ordinary C 38 steels it must be borne in mind that in the former not only the changes in length but also certain relative changes of position between the two individual bars (roping) plays an important part. This probably explains, too, the observation made that the lowering of the coefficient of elasticity at low tensions is somewhat more rapid than when the stresses become greater.

As the yield stress limit of "Isteg" Steel under steady increase of the stress-strain curve does not appear so pronounced as for ordinary C 38 steel, and as fracture only ensues at a total elongation of 0.4 %, the figure 0.3 % total elongation was taken as being decisive for the yield limit of "Isteg" Steel. The surge-load strength of "Isteg" Steel at a load frequency of two million (350 per

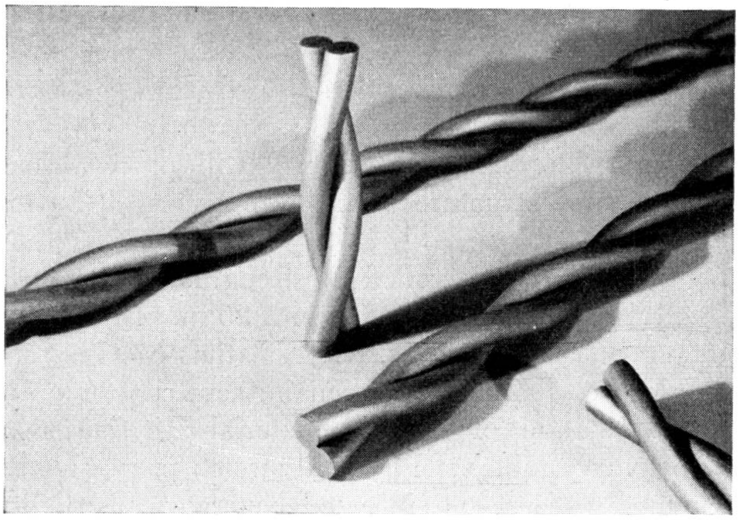

Fig. 2.

minute) amounts to from 2400 to 2500 kg/cm². The apprehension that the stretching of "Isteg" Steel under repeated impact action might have a deleterious effect, is therefore unfounded. The mean grip strength of "Isteg" Steel was also found to be about 25 % greater than that of ordinary round bars of C 38 steel. As regards safety against cracking, the tests carried out show that "Isteg" Steel is superior to the ordinary C 38 steel reinforcing bar, although great caution has to be observed when employing in practice the results obtained from ascertaining the first appearance of hair-cracks in reinforced concrete constructions. It is, however, a fact that with "Isteg" reinforcement the cracks were evenly distributed over the whole length, and even when the load was increased proved to be very much finer than in the case of C 38 steel. Here single cracks appeared, which widened more and more as the load was increased.

On the strength of the tests made, therefore, taking as a basis a *minimum yield stress limit of 3600 kg/cm²* at a total elongation of 0.3 %, a minimum strength of 4000 kg/cm² and a minimum ultimate elongation of 10 %, *a permissible* *tensile stressing of 1800 kg/cm²* could be prescribed for "Isteg" Steel. In this connection, when dimensioning the cross-sectional area, the ratio of the coefficients of elasticity of steel and concrete has to be assumed as being n = 15, while elastic changes of shape and statically undeterminable dimensions have to be calculated with n = 8. Welding and hot bending are not admissible. Apart from this, the same principles of construction (grip length, formation of hooks and so forth) apply for "Isteg" Steel as for ordinary reinforcing rods of C 38 steel.

Reinforcing bars of "Isteg" Steel are manufactured in diameters of from 5.5 mm to 30 mm, in lengths up to 30 m. Their average standard price per 100 kg is Cz. Kr. 168, that of C 38 steel bars Kr. 147.

In view of the above qualities and properties possessed by the two types of reinforcing bar *Roxor* and *Isteg*, they may claim both economic and technical superiority over the normal type of reinforcement with round bars of C 38 steel. Their high yield stress limit increases the carrying capacity of concrete constructions and effects a considerable saving in cross section and weight of the

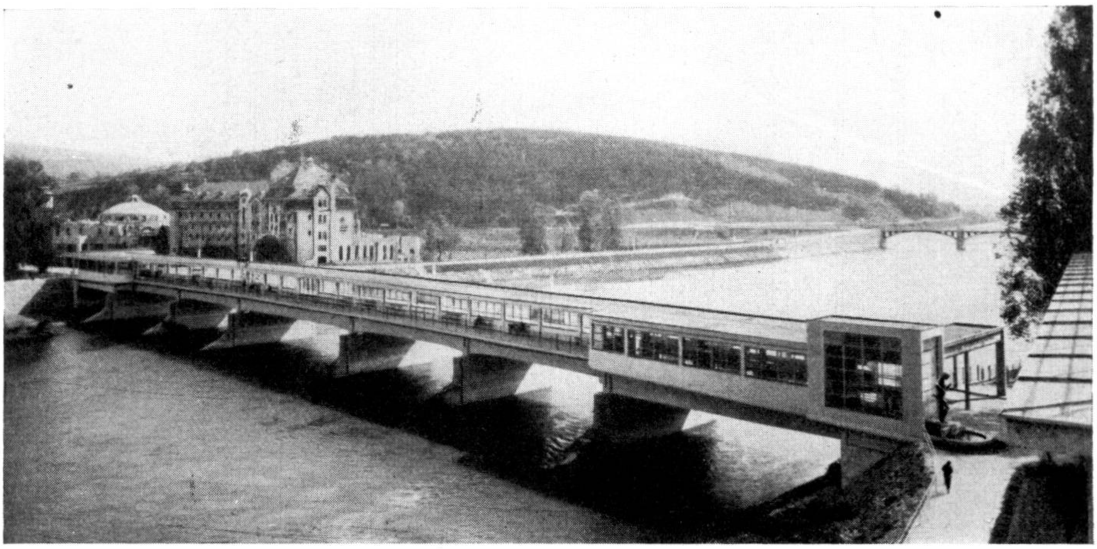

Fig. 3.

reinforcement, together with a corresponding economy as regards freight expenses and cutting, bending and laying wages. On account of their lighter weight and the impossibility of confusing normal and superior building steel, it is more advantageous to lay these bars at site; nor is high-quality cement an absolute necessity for these types of reinforcement steel. *In spite of their relatively higher unit price, they save the public purse at least 20%.*

The far-reaching development of the use of high-quality building steels in the construction of reinforced concrete bridges began in Czechoslovakia on the introduction of "*Isteg*" Steel in 1931.

One of the first structures in which it was used was the *bridge across the River Waag in Piešťany* (Fig. 3), which links the town of Piešťany on the right bank of the river with the thermal springs and baths on the island and is exclusively used for the traffic of the Spa. The whole planning of the bridge, from beginning to end, is in itself extremely peculiar, for the structure is partly a covered bridge whose roofed-in portion forms a colonnade used by visitors to the Spa (Fig. 4).

The bridge is 148 metres long and composed of seven spans, the middle of which is 28 metres long, all the others are of 20 metres. The superstructure is built of continuous T-beams over three spans each. With the exception of the end bearings, the structure is monolithically connected with the piers. The beams are cantilevering at both ends, with a projection of 4.3 m over the end bearings. The central span is also provided with cantilever arms to carry a hinged portion 20 metres long.

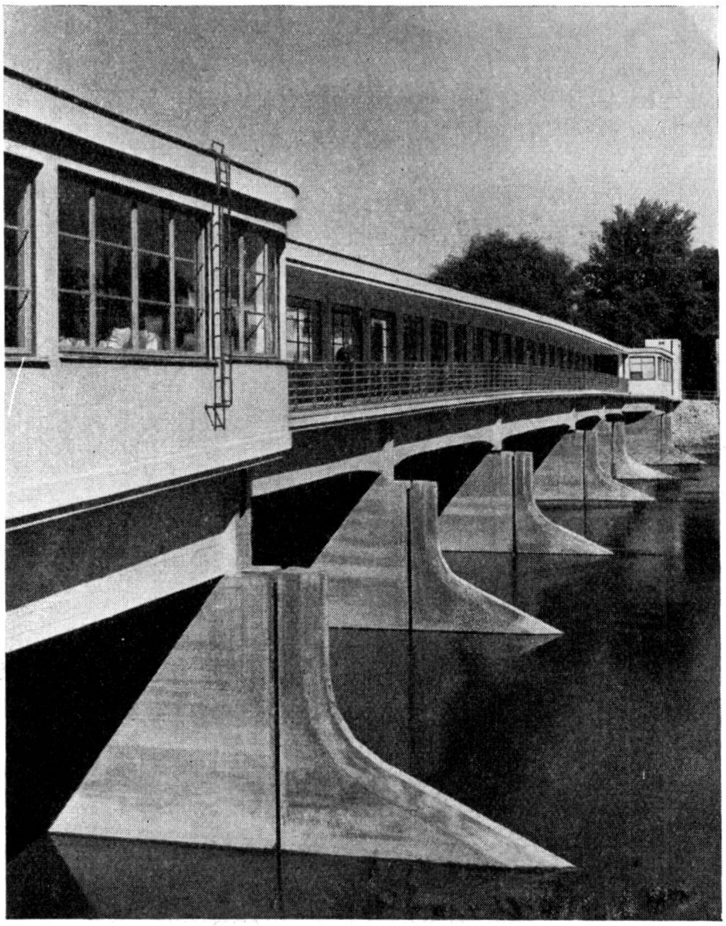

Fig. 4.

The clear width of the bridge for the five middle spans is 12.34 m, of which 5 m comprises the asphalted roadway and 6.40 m the useful width of the covered colonnade (Fig. 5). The columns are situated at the middle of the wide walk at 5 m intervals; they are connected at the top by a beam supporting the reinforced concrete roof. The covered colonnade is divided by glass partitions into two independent parts, so that when crossing the river visitors to the Spa may use the sheltered half, i. e. the leeward side. Over each end-span the two colonnades merge into a single hall, enclosed on all sides and containing permanent exhibitions of Czechoslovakian arts and crafts. For this reason 2.5 m was added to the width of the end bays of the bridge by insertion of a T-beam.

Apart from the live load of 4-ton wagons, i. e. 400 kg/m^2, the structure of the bridge is under extremely heavy load owing to the great weight of its superstructure. The employment of *"Isteg"* Steel as reinforcement (Fig. 6) thus

Fig. 5.

enabled the over-all dimensions of the reinforced concrete structure to be reduced and a considerable saving to be effected.

Fig. 6.

The total consumption of building materials amounted to about 30 wagons of steel, 10 wagons of Bauximent and 100 wagons of Portland cement. About 4000 cubic metres of concrete and 1000 cubic metres of timber were used.

One of the most interesting structures for which "Isteg" Steel has been used as reinforcement in bridge-building, is the State Road Bridge over the River

Strela at Plasy, near Pilsen (Fig. 7), where a new reinforced concrete girder bridge of 30.58 m span was erected to replace the old iron lattice-work construction.

The superstructure of this bridge consists of solid web girders, with sunk road construction. The over-all width of the bridge between the two main girders measures 6.00 m, of which 5.20 m comprises the paved roadway and twice 0.40 m the kerbs at either side. Outside of the main girders and borne on brackets, a footpath of 1.30 m clear width is arranged at each side.

The suspended girders have a width of 76 cm and a height of 2.80 m, i. e. approx. 1/11 of the span. The main girders are 1.30 m higher than the footpaths and kerbs, so that for the most part they are hidden by the 1.10 m high parapet. The structure itself is a skew bridge and the cross-girders are placed at right angles to the two main girders and at intervals of 1.39 m. To keep down the dead weight, openings were arranged in the middle parts of the main girders.

Fig. 7.

Decking slab and cross-beams are reinforced with ordinary round bars of C 38 steel, while the tensile reinforcing of the main beams was carried out with "Isteg" Steel bars of 30 mm diameter. The reinforcement was delivered to site in full-length bars, so that no splicings were necessary. The longest one-piece "Isteg" bar built into the structure measured 38.59 m.

Decking slab and cross-beams were dimensioned with due regard to the impact coefficient, on the basis of a permissible stressing for the concrete of 48 kg/cm^2 and a permissible tensile stressing for the steel of 1200 kg/cm^2. In the main beams the greatest tensile stresses measured 69.4 kg/cm^2 (permissible stressing 70 kg/cm^2), and 1662 kg/cm^2, (permissible stressing 1800 kg/cm^2), respectively. In order to avoid tension cracks in the main beams wire-netting was laid around the tensile reinforcement in addition to the stirrups, the object being to increase the tensile resistance of the concrete cover over the bars. When laying down the loading conditions, the Czechoslovakien regulations for the loading of 1st class road bridges were adhered to, viz, the use of a 22-ton steam plough and a human crowd equivalent to 500 kg/cm^2.

The specified cube strength of the concrete after 28 days curing was 170 kg/cm^2 for roadway and footpaths, and 330 kg/cm^2 for the main beams.

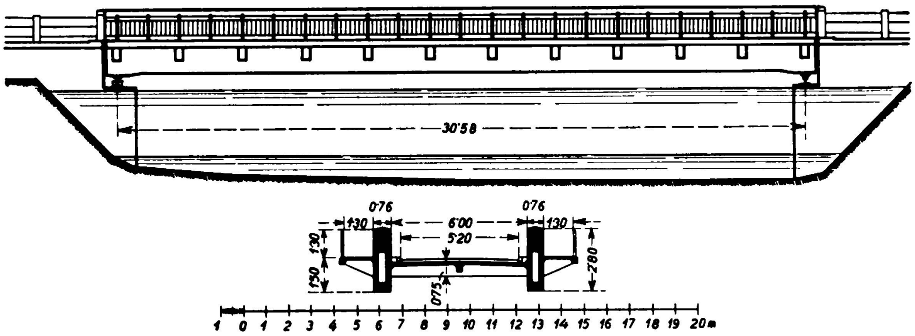

Fig. 8a.

The effective strengths attained in actual construction were 334 kg/cm^2 and 486 kg/cm^2 respectively for 250 kg and 420 kg respectively of Portland cement per cubic metre of dry sand-and-shingle mixture and a grain-size proportion of aggregates 5.70 and 6.30 respectively.

The results of quality tests carried out with the types of steel used are classified in Table III.

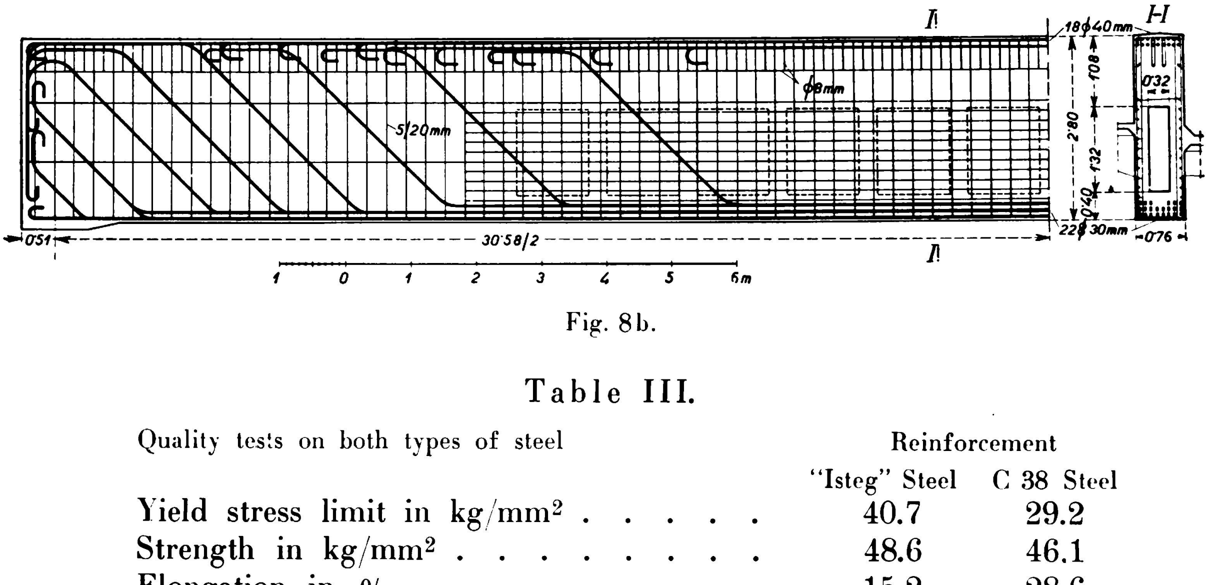

Fig. 8b.

Table III.

Quality tests on both types of steel	Reinforcement "Isteg" Steel	C 38 Steel
Yield stress limit in kg/mm^2	40.7	29.2
Strength in kg/mm^2	48.6	46.1
Elongation in %	15.2	28.6
Constriction in %	52.6	58.6

The loading tests on the bridge were carried out with 4 twelve-ton vehicles. The greatest elastic deformation undergone by the main beams measured 2.60 mm, as against a calculated figure of 3.10 mm; that of the cross-beams amounted to 0.15 mm as against the 1.30 mm calculated. No permanent deformations were recorded.

The total consumption of concrete per sq. metre covered area of the bridge is 38.5 cm, the total steel consumption 133 kg. Of the latter figure 48 kg is "Isteg" Steel, the remainder being ordinary round C 38 steel bars.

Besides "Isteg" Steel, which is produced artificially from ordinary building steel by cold-stretch processing, "Roxor" Steel, manufactured in a natural manner in the actual steelworks, has been employed as reinforcement in reinforced concrete structures in Czechoslovakia since 1933.

One of the first constructions in which "Roxor" was used was that of the *Bridge over the Svratka in Brunn* (Fig. 9) on the Vienna-Brunn highway. This bridge has a clear skewed width of 31.20 m.

When deciding upon the type of construction to be employed, and in working out the project as a whole, two factors had to be taken into particular consideration, viz. the limited constructional height, and the fact that the bridge had to be capable of being widened on both sides and that the municipal tram-lines could be laid as desired. Finally, it was specified that all reinforcing bars had to lie above high-water mark.

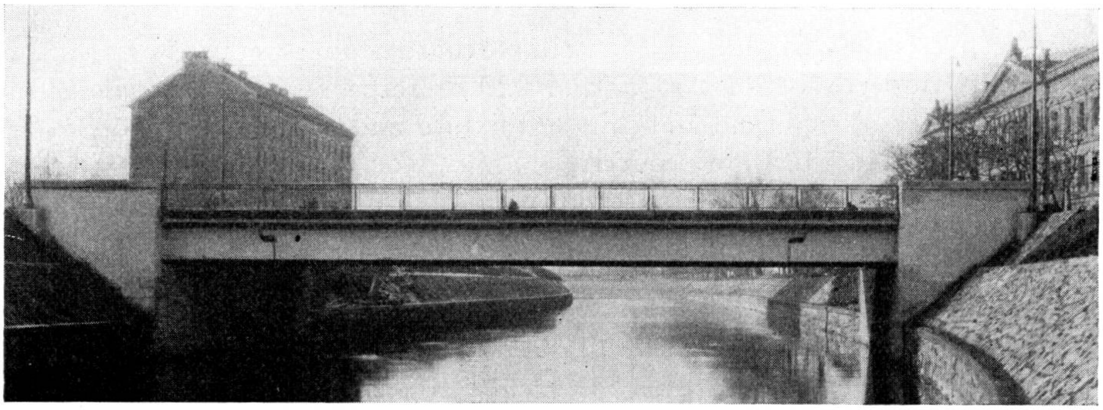

Fig. 9.

With the object of suiting the appearance of the bridge to its surroundings, the type of structure chosen was that of continuous T-girders over three spans with hinges in the middle span (Fig. 10 a b c). In this manner the advantage, offered by continuous girders, in reducing of the bending moments in mid-structure could be maintained, whereas the disadvantages that might be feared from subsidence of the supports were eliminated.

The suspended girder in the 32.30 m long middle span has a length of 22.80 m. The position of the hinges in the middle bay, as well as the length of span in both end bays — which had to be filled up in consequence of the river correction scheme — were so chosen that the positive moment in the span of the suspended girder became equal to the negative moments over the two middle supports. This yielded cantilever arms of 4.70 m and a 13.00 m span for the end bays. Thus it was possible to limit the constructional height to only 1.80 m, i. e. about $^1/_8$ middle span length, or $^1/_{13}$ that of the suspended girder. In order to secure a safety factor of 1.4 against overturning when the middle bay is under full load, those portions of the end bays not required to carry conduits were constructed with box-type girders weighted with concrete filling.

The clear width of the bridge measures 17.60 m, of which 11.50 is allotted to the paved roadway and 3.00 m on each side to the two footpaths. The whole arrangement comprises 8 T-beams at intervals of 2.20 m.

The tram-line base consists of a 13 mm thick shock-absorbing "Contravibron" slab between layers of 3 mm thick lead sheeting.

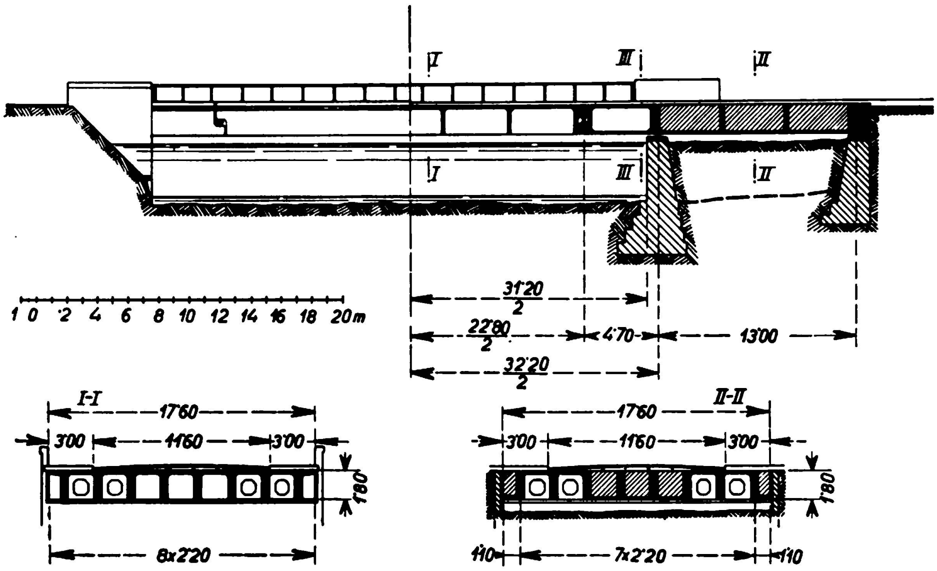

Fig. 10a.

The statical calculation of the bridge was carried out not only in respect of the loading of 1st class road bridges in accordance with the regulations in force in Czechoslovakia, but also for 22-ton railway wagons hauled by electrically driven locomotives, and then for 21-ton watering cars or motor-driven vehicles with 13-ton trailers — all rolling stock of the electric tramways. In addition, the

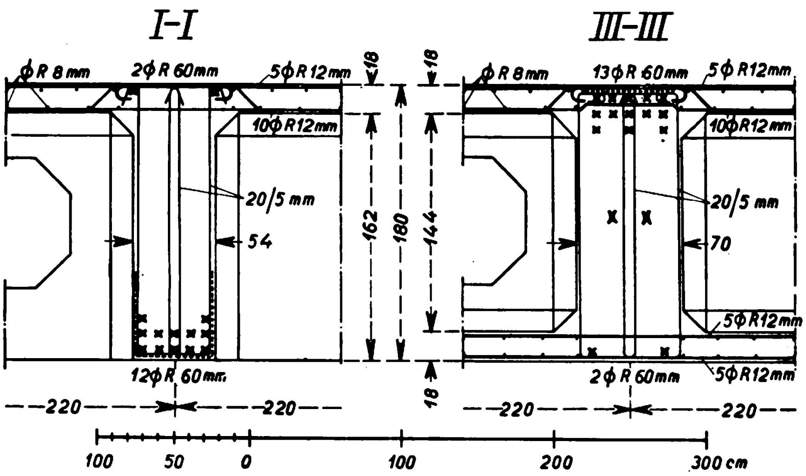

Fig. 10b.

bridge carries three water mains, the gas main, and electric power and telephone cables. For this reason, and in order to render the hinges in the middle bay accessible, openings were left in the cross braces.

With the exception of the stirrups, all the reinforcing bars used in the bridge are of "Roxor" Steel (Figs. 11 and 12). The advantage of employing this high-quality material is that it effects a reduction of the cross-sectional area of the reinforcement required and allows the constructional height available to be better utilised, since at least four rows of reinforcing bars, would have been required had ordinary C 38 reinforcement been employed and the ideal cross-sectional height thus reduced.

The greatest stressing of the 18 cm thick decking slab is 42.2 kg/cm^2, that of the steel 1623 kg/cm^2. In the main beams the greatest tensile stresses amount to 69.2 kg/cm^2 (permissible stressing 70 kg/cm^2), and 1750 kg/cm^2 (permissible stressing 1900 kg/cm^2) respectively.

To prevent the occurrence of tension cracks welded steel-wire mesh was placed around the reinforcing bars at the points of greatest tensile stress, viz. in the lower portion of the suspended beam, and in the upper part of the beams over the middle piers.

The regulation cube compression strength of the concrete used in the structure was 330 kg/cm^2 after 28 days curing. In actual construction, with a concrete mixture of 350 kg Portland cement per cubic metre of dry aggregates, grain-size proportion 6.06, a cube strength of 431 kg/cm^2 was attained.

The results of the quality tests carried out on the "Roxor" steel employed were as follows:

Yield stress limit average . . .	41.1 kg/mm^2
Strength average	59.2 kg/cm^2
Elongation average	24.4 %
Constriction average	54.2 %.

When planning execution of the bridge it was borne in mind that the scaffolding would settle under the weight of the concrete. For this reason the concreting work was proceeded with in such a manner that cross sections situated at points of greatest bending moments were concreted last. This applied chiefly to the cross sections over the two middle piers, and to the central cross section of the suspended beams. The bridge being a skewed structure ($d = 81^0\ 30'$) and relatively wide, the suspended beams in the middle bay were only concreted after the form-work of the end bays and cantilevers had been removed. This was done to prevent any torsion that might take place from being transmitted to the middle span.

When carrying out loading tests on the bridge the following vehicles were employed: two 21-ton watering cars belonging to the electric tramways, two 20.5-ton motor-driven cars, one 14-ton petrol-driven roller, one 12-ton naphtha-driven roller, besides 85.5 tons of paving stones on the footpaths. The total test load thus amounted to 180.5 tons. The greatest elastic deflection recorded for the main beams below the tram-lines was 2.35 mm, as against the 4.47 mm calculated; that of the other main beams was 2.05 mm as against 2.90 mm. The result of the loading test was therefore extremely satisfactory.

Total concrete consumption, including concrete filling, amounted to 79 cm per sq. metre of covered area; total steel consumption only 128 kg, of which the stirrups-ordinary C 38 building steel, accounts for 10 kg, the remainder being "Roxor" Steel.

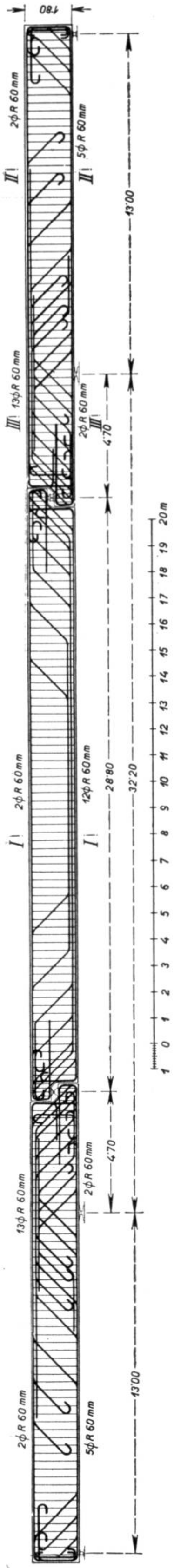

Fig. 10 c.

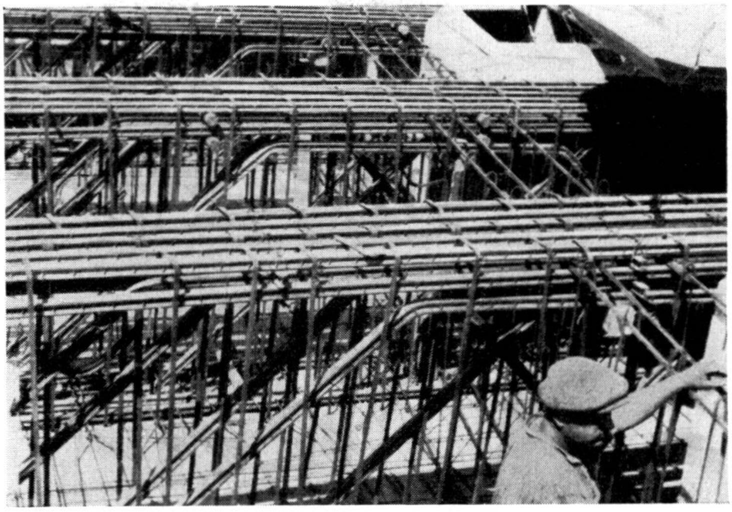

Fig. 11.

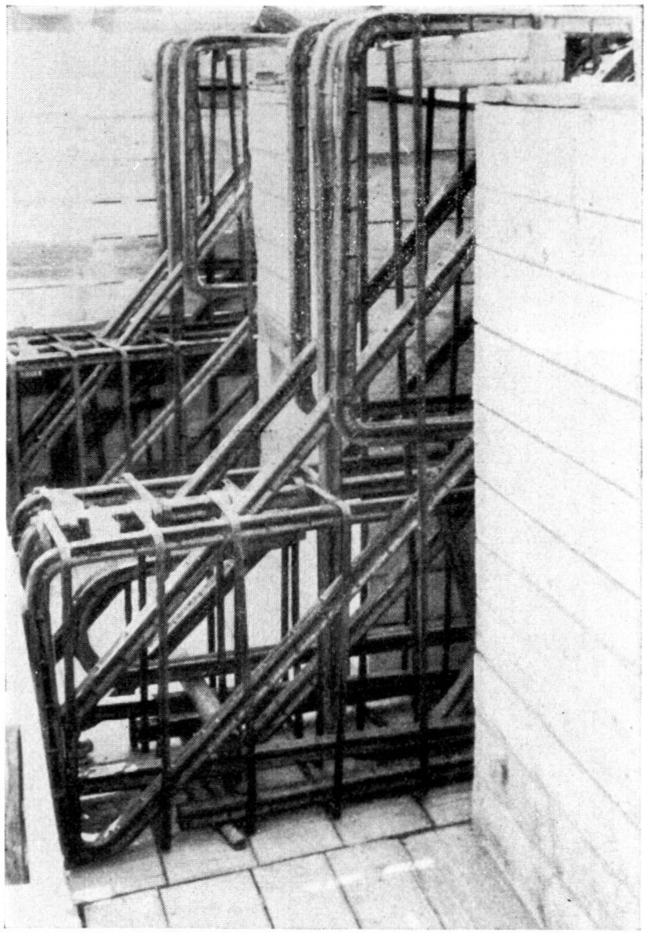

Fig. 12.

Summary.

The report considers from a theoretical standpoint the two high-quality types of steel, "Roxor" and "Isteg", employed in reinforced concrete construction in Czechoslovakia. The high yield stress limit of "Roxor" Steel is attained in a natural manner during actual manufacture in the steel-works, that of "Isteg" Steel by artificial cold stretching.

The report further describes the application of "Isteg" and "Roxor" Steel in the construction of some State Road bridges.

IIc 2

Use of High-Grade Steel in Reinforced Concrete.

Anwendung des hochwertigen Stahles im Eisenbetonbau.

Application de l'acier à haute résistance dans le béton armé.

Dr. Ing. W. Gehler,
ord. Professor an der Technischen Hochschule,
Direktor beim Staatl. Versuchs- und Materialprüfungsamt, Dresden.

Whilst the safety of steel structures in *high-grade steel* can easily be determined as against structures made of ordinary *commercial steel*[1], this question leads to intricate problems in the case of reinforced concrete constructions on account of the compound action of steel and concrete, which can only be solved by exhaustive research investigations. A considerable contribution to this is furnished by the large series of tests carried out by the German Commission for Reinforced Concrete (Deutscher Ausschuß für Eisenbeton), in particular also the tests undertaken at Dresden. The knowledge attained by such tests will be elucidated in the following:

A. High-grade steel as used in steel structures.

To classify an important element of building steel with its given property characteristics and at the same time *to compare steel structures and reinforced concrete structures,* it is advisable to start by elucidating our present-day conception about *the application of high-grade building steel in structural engineering and bridge building.*

I. The high-quality steel St. 52 for steady or more or less steady loading, i. e. for structural steel work and road bridges in steel.

Whilst the minimum value of tensile strength e. g. $\sigma_B = 52$ kg/mm² or 37 kg/mm² is generally used to classify a certain kind of steel (e. g. St 52 or St 37), the ratio of yield limit forms the actual basis of the permissible strength of the two kinds of steel.

$$\sigma_{adm\,52} : \sigma_{adm\,37} = \sigma_{s\,52} : \sigma_{s\,37} = 36 : 24 = 3 : 2 \qquad (1)$$

hence the permissible stressing for $\sigma_{adm\,37} = 1400$ kg/cm²

and correspondingly $\sigma_{adm\,52} = 2100$ kg/cm² (2)

[1] Commercial bulding steel is of mild-steel quality with a minimum tensile strength of kg/mm², a maximum tensile strength of 50 kg/mm² and a minimum elongation before rupture of 18 % for the long standard test bar. This kind of steel must suffice for cold bending of 180° round a steel pin of D = 2 a. (For commercial round bars for concrete these figures are not yet guaranteed).

To make full use of the advantages offered by 50 % higher stressing for St 52 is unfortunately not possible for two sub-fields of static investigation, since Young's modulus $E = 2\,100\,000$ kg/cm^2 is practically constant for all kinds of steel.

a) Although the deflection f is not in general restricted by regulations, the disadvantage still exists, that the deflection increases in proportion to the stresses for a constant beam section; for instance a simply supported beam with the depth h $\left(M = \sigma \cdot W = \sigma \cdot \frac{2J}{h}\right)$ the deflection f has the value:

$$f = \frac{5}{48} \cdot \frac{Ml^2}{EJ} = \frac{5}{24} \cdot \frac{l^2}{h} \cdot \frac{\sigma}{E} \qquad (3)$$

which proves very unfavourable, particulary in structural steel work.

b) Since the buckling load within the Euler-range $P_k = \frac{\pi^2 EJ}{s_k^2}$ is practically the same for a given length s_k for all kinds of steel, no advantage is attained for slender bars (with $s_k : i < 100$) made of *high-grade steel.*

The chief advantages of high-grade steel lie in the saving in dead weight, particulary for wide spans (e. g. about 26 % for the bridge over the Little Belt, span $l = 200$ m), and with it the possibility of being able to execute structures standing on weak subsoil (coal handling bridges of wide spans) and finally the reduction of weight and customs duty in case of exporting such steel.

As a simple characteristic for the quality of steels for statically stressed structures, the elongation δ_B before rupture takes place has been introduced, since this characteristic, similar to the behaviour in a cold bending test, is an indicator for the *toughness for steel worked cold in the shop as well as on site.* From the stress-strain-lines of the tensile test Fig. 1 (see also Table I) it follows that

Table I

Kind of Steel	Ultimate strength σ_B kg/mm^2	Yield limit σ_S kg/mm^2	Elongation δ_B %	Constriction %	Quality coefficient $\sigma_B \cdot \delta_B$ (kg/cm^3)	Deformation energy		$\frac{A_B}{\sigma_B \cdot \delta_B}$
						Rupture energy A_B kgcm/cm^3	Energy capacity A_{ges} kg cm/cm^3	
St.37 (min)	42,8	31,0	18	59,7	770	490	650	0,637
St.38 (max)	42,8	31,0	30	59,7	1294	860	1180	1,11
St.48	56,8	33,9	21	48,7	1193	760	1000	0,637
St.52	56,0	38,2	26,5	59,5	1484	910	1280	0,614
St.52	56,4	42,5	27	56,0	1523	940	1290	0,617
								mean abt. 2/3

for the usual qualities of steel the area of the specific deformation energy (up to point B of the tensile strength) has a mean value of

$$A_B = \frac{2}{3} \sigma_B \cdot \delta_B \qquad (4)$$

The area of the circumscribed rectangle is expressed by the term

$$A = \sigma_B \cdot \delta_B, \tag{5}$$

and represents the so-called "work of rupture" which can be regarded as a *practical quality coefficient for the toughness of steel.* According to table I this value fluctuates between 800 and 1500 kg/cm²;[2].

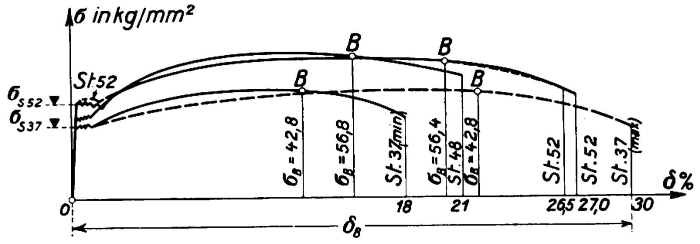

Fig. 1.
Stress-strain lines for various steels.

II. The high-quality steel St. 52 in riveted steel bridges, under rail tracks, with frequently changing loads.

Whilst dynamic influences on road bridges are compensated by introducing ample rolling loads and by multiplication of the forces in the members produced by these loads, by an impact coefficient φ, which depends on the span l (where $\varphi = 1.4 - 0.0015\ l$);[3], the investigation only requires to be made for static loads, but for railway bridges, riveted or welded, the fatigue strength has to be considered[4].

The safety of such bridges is therefore based on the *static basis of calculation;* the fatigue, however, is laid down by fatigue tests through the number n of loading repetitions, while in bridges the fatigue is characterized by the number of passing trains. The *fatigue strength* depends to a very great extent on the nature of loading, e. g. alternating loads and surge loads (oscillation of loads without change of direction), or on the ratio of the extreme values of stresses in the members:

$$\xi = S_{min} : S_{max} \tag{6}$$

As found by tests, the fatigue strength σ_D for riveted connections for St 52 and St 37 no longer gives the same ratio as the yield stresses (see Eq. 1) and with it the permissible stresses $\sigma_{D\,adm}$ are altered too. *For St 52* $\sigma_{D\,adm}$ *= 1800 kg/cm², however, for St 37 1400 kg/cm² again applies.* It is essential to note that *the yield strength has to be ruled out in judging the safety of a structure,* its place being taken by the *cohesion or separating strength. The safety can only be conceived after fatigue tests have been carried out.*

III. Welded connections, in structural work, railway-and road bridges of building steel, require, based on tests, to have their permissible stresses differently reduced in respect to the various types of welds and the various *"Form*

[2] See *W. Gehler:* The development and importance of high-grade steels in steel structures and reinforced concrete, World Engineering Congress Tokio 1929, Paper Nr. 218. — „Die Entwicklung und Bedeutung der hochwertigen Baustähle im Eisenbau und Eisenbetonbau."

[3] See *W. Gehler:* Hand book for Civil Engineers, Edition V, Vol. II, p. 375. Berlin 1928. Julius Springer.

[4] See *W. Gehler:* Contribution to „Discussions" ad IIIb.

coefficients α"; for butt and fillet welds (end and side fillets) and if it concerns welding of ordinary or high-class workmanship.

IV. The two outstanding features of the progress made in the last decade i. e. the introduction of high-grade building steels and electric welding in steel structures, both enabling a reduction of costs of about 15 % (for wide-spans even more), have induced us to study the various questions regarding the safety of steel structures as indicated by Pars. I to III. Fundamentally the same questions arise with high-grade steels in reinforced concrete construction, only that the conditions are altered on account of the compound action of steel and concrete.

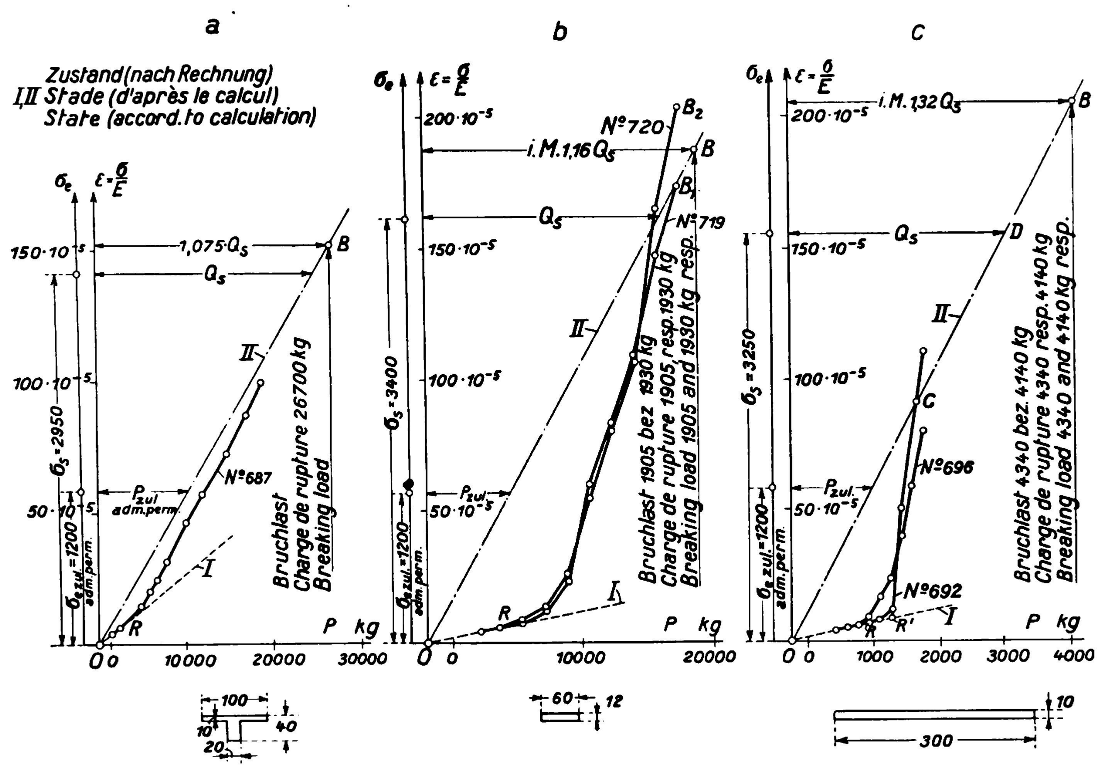

Fig. 2a—c.

Load-elongation diagram for steel for:

a T-beam. b Slabs. c Slabs cross-wise forced supported all sides.

B. The ratios expressing the safety of structural elements in reinforced concrete against failure and the formation of cracks.

1) *The safety against rupture based on the measured load-steel elongation lines* (q — ε_e*-lines*).

In Fig. 2a—c are shown, for a T-beam, a slab-strib, and a slab supported on all four sides, the measured load-steel elongation lines (full lines) in comparison to the usual lines (dotted) received by calculation[5]. (The dermination of the stresses from the measured elongations was based on the total elongations.)

[5] Dissertation by *Walter Heide:* The Dresden tests on crosswise reinforced concrete slabs, in comparison to the usual mode of calculation. „Die Dresdener Versuche mit kreuzweise bewehrten Eisenbetonplatten im Vergleich mit der üblichen Berechnung." *Chair:* Prof. Gehler, Inst. of Tech. Dresden, 1933, pp. 12 and 28.

a) *T-beams.* Fig. 2a (Dresden tests[5] 1928, Issue 66 p. 65, No. 687, reinforced with St. 37 dimensioned for $M = \frac{ql^2}{8}$). *Test results and calculations coincide for state I.* The tensile area, being small, has only little influence on the rising of the curve. The values received from the usual mode of calculation for state II coincide with those already measured for cases of small stages of loading. *The stress in steel max σ_e calculated from the ultimate strength* (rupture strength) is only 7,5 % above the yield limit $\sigma_s = 2950$ kg/cm² (compare also D III):

$$\max \sigma_e = 3170 = 1{,}075\,\sigma_S \quad \text{or} \quad = \frac{\max \sigma_e - \sigma_S}{\sigma_S} = 7{,}5$$

The *safety against* rupture is therefore:

$$\nu_B = \frac{q_B}{q_{adm}} = \frac{\sigma_s}{\sigma_{adm}} = \frac{2950}{1200} = 2{,}5$$

To state the safety against rupture th eexpression $\nu_B = \sigma_s : \sigma_{adm}$ is used with advantage: it is based on the yield limit. The expression

$$\nu'_B = \frac{\max \sigma_e}{\sigma_{e\,adm}} = \frac{3170}{1200} = 2{,}63$$

should, however, not be used (as it will also be seen from explanations given under D III).

b) *Slab-strip* (with rectangular section) (Dresden slab tests, 1932, Issue 70, pp. 179 and 180, No. 719 and 720, span 3.0 m, reinforced with St 37, designed for $M = \frac{ql^2}{8}$, Fig. 2b). For state I good agreement exists between calculation and test results. The line OR rises under a steep incline, since the tensile area of concrete is large in this case and therefore the steel reinforcement is to a great extent relieved. The values measured for steel elongation are very small until the appearance of the first cracks (see point R).

From point R a great deviation takes place between the line OR for measured and OB for calculated elongation. From this point steel alone takes over all the work of tension, and the steel elongations increase with the increase of loading. As the conclusion of the test (on attaining the yield limit $\sigma_s = 3400$ kg/cm²) the two lines OB and OB_1 (or OB_2) join almost at one point, which again allows the safety against rupture to be based on the yield limit of steel, as follows:

$$\nu_B = \frac{q_B}{q_{adm}} = \frac{\sigma_s}{\sigma_{adm}} = \frac{3400}{1200} = 2{,}8$$

c) *Crosswise reinforced slab, supported on all sides.*

(Dresden slab tests 1932, Issue 70, pp. 52 and 100, No. 692 and 696[6], $l_x = l_y = 3.0$ m reinforced with St 37, designed for $M = \frac{1}{27{,}4}\,ql^2$ Fig. 2c). The values calculated compare well with those measured, for state I. Fundamentally the same remarks apply as for the slab-strip (see above under b). After the appearance of cracks (see point R) the steel-elongation increases com-

[6] See Preliminary Publication of Ist Congress of I.A.B. St.E. Paris 1932, p. 205 and 237.

paratively rapidly. The lines for calculated and measured values cross each other in point C. For rupture (in poin B) we receive:

$$\nu_B = \frac{q_B}{q_{adm}} = \frac{4200}{990} = 4{,}2$$

Since the two lines do not meet in point D of the yield limit, *the value*

$$\frac{\sigma_S}{\sigma_{adm}} = \frac{3250}{1200} = 2{,}7$$

of the yield limit can therefore not be decisive for the safety.

Conclusion: For slabs (with rectangular cross-section) and T-beams the safety against rupture is expressed by the term:

$$\nu_B = \frac{q_B}{q_{adm}} = \frac{\sigma_S}{\sigma_{adm}} \tag{7}$$

This relation, however, does not apply for slabs crosswise reinforced, supported on all sides, but here the ratio ultimate load to working load has to be used:

$$\nu_B = \frac{q_B}{q_{adm}} \tag{8}$$

2. *The safety against cracking.*

If q_R indicates, for uniformly distributed load, that particular stage of loading for which the first crack becomes visible, and if q_{zul} stands for the working load (or permissible load) then the safety against cracking is given by the ratio:

$$R = \frac{q_R}{q_{adm}} \tag{9a}$$

Loading ratio. For point loads the values q_R and q_{adm} are replaced by the bending moments M_R and M_{adm} or by the stresses σ_{eR} and $\sigma_{e\,adm}$ which are proportional to the moments M_R and M_{adm} therefore the following relation applies:

$$\nu_R = \frac{q_R}{q_{adm}} = \frac{M_R}{M_{adm}} = \frac{\overline{\sigma_{ek}}}{\sigma_{e\,adm}} \tag{9b}$$

If, however, the steel-elongation ε_R is measured, in other words if the cracking point stress $\sigma_{eR} = E \cdot \varepsilon_R$ is established by tests, and hence the relation

$$\nu'_R = \frac{\sigma_{ek}}{\sigma_{e\,adm}} \tag{10}$$

(stressing ratio) is formed, then the question arises whether the term ν'_R is equal in value to the safety against cracking ν_R. (Acc. Eq. 9b). Evidently this is only true if the load steel elongation line, or the load steel stressing line OA in Fig. 3 increases in linear proportion up to the value of ultimate load. In this case the point R coincides with R', and comes to fall on line OA, and for $\sigma_{eR} = \overline{\sigma_{eR}}$. The equation 10 changes into Eq. 9b. This result is to be expected and is sufficiently accurate for T-beams according to Fig. 2a, as proved by Fig. 4. In Fig. 4 the values ν_R and ν'_R (according to Eqs. 9 and 10) have been plotted in relation to the cracking point stress σ_{eR}, for which the values for σ_{eR} have been measured by means of the steel elongations. For T-beams, according to the Dresden tests of 1935, the values for ν_R and ν'_R coincide in a satisfactory manner as shown by lines CD and EF. This was not the case with earlier tests, (1928, Issue 66) [an indication that a specially developed technique was

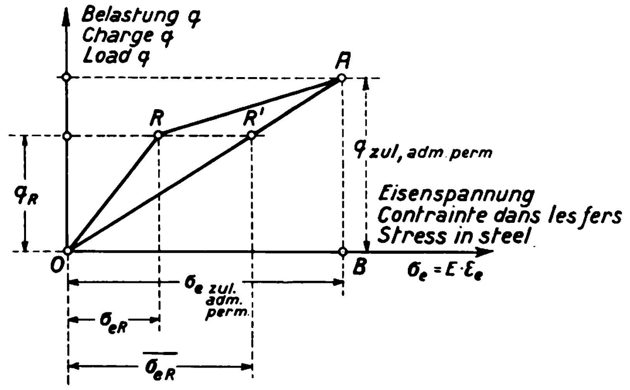

Fig. 3.
Load-stress diagram of steel.

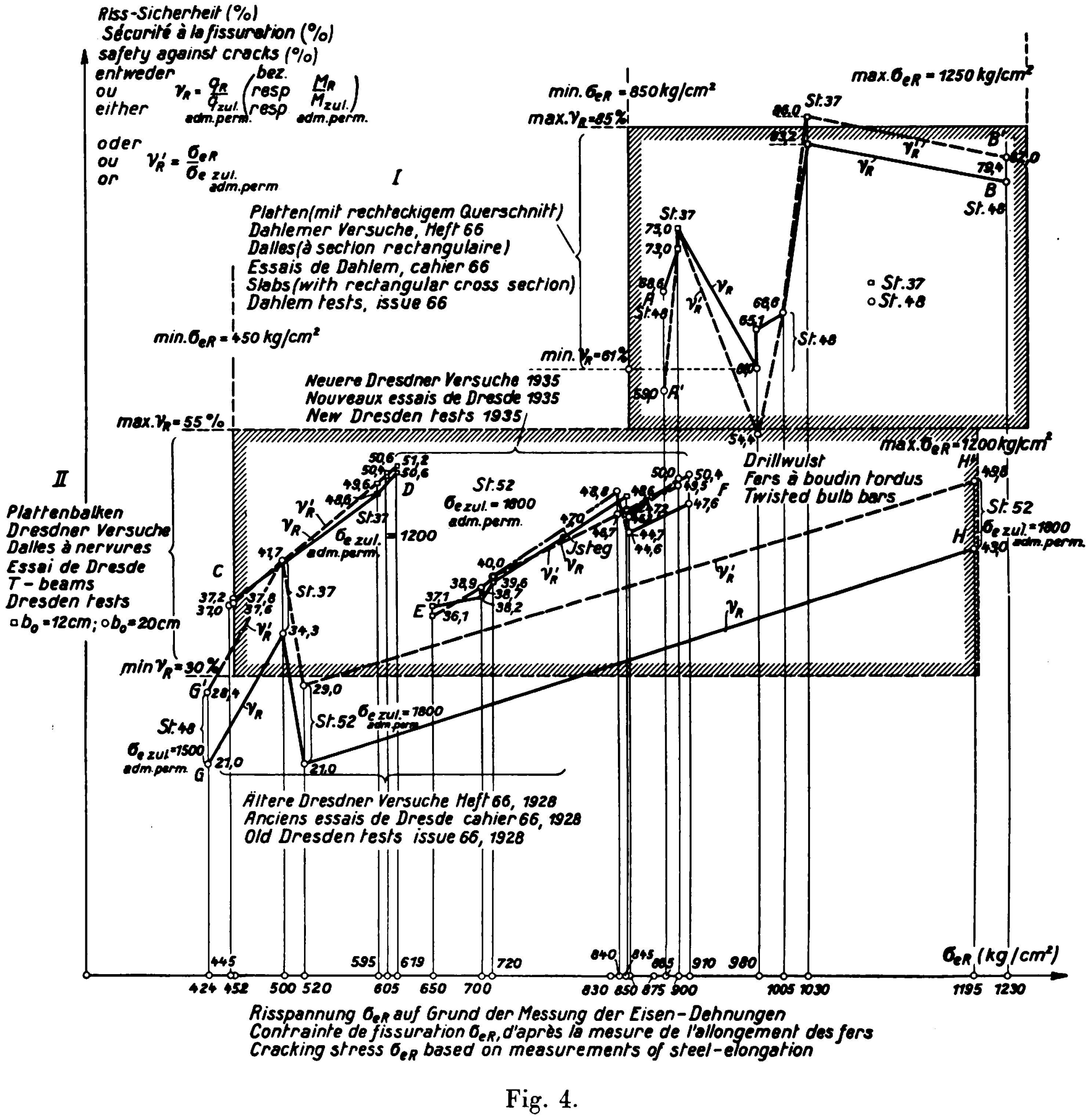

Fig. 4.

Dependence of safety against cracks ν_R (scale of loads) or ν'_R (scale of elongations) from the cracking stress σ_{eR} for slabs and T-beams.

required to investigate the safety against cracking]. For slabs with rectangular section (see lines AB and A'B') for which such new investigations have not yet been carried out, this question has to remain open until the Dresden tests now in progress have been concluded. *It is therefore advisable to base such slabs only on the loading ratio:*

$$\nu_R = \frac{q_R}{q_{adm}} \tag{9}$$

but the stressing ratio $\nu'_R = \frac{\sigma_{eR}}{\sigma_{e\,adm}}$ *(Eq. 10) applies also for T-beams, not only the ratio according to equation 9.*

C. The safety against cracking for slabs and T-beams in concrete reinforced with high-grade steel.

I. The quantities to be measured from tests are:

1) steel elongation ε_{eR} at the moment when the first crack appears, and the resulting cracking point therefrom:

$$\sigma_{eR} = E \cdot \varepsilon_{eR},$$

2) depth of cracks:

t_l for $\sigma_{e\,adm} = 1200$ kg/cm² for St 37

t_l for $\sigma_{e\,adm} = 1800$ kg/cm² for St 52 and special steels.

3) width of cracks for various stages of loading, especially for:

b_R for $\sigma_{e\,adm}$

b'_R at the yield limit σ_s.

The procedure for the tests of 1935 was as follows:

a) The width of the first through cracks was measured for every test beam at the place of the centre of gravity of the reinforcement, by means of a microscope with attached ocular micrometer. Two of the cracks of each test beam were photographed 23 times enlarged (See Fig. 5).

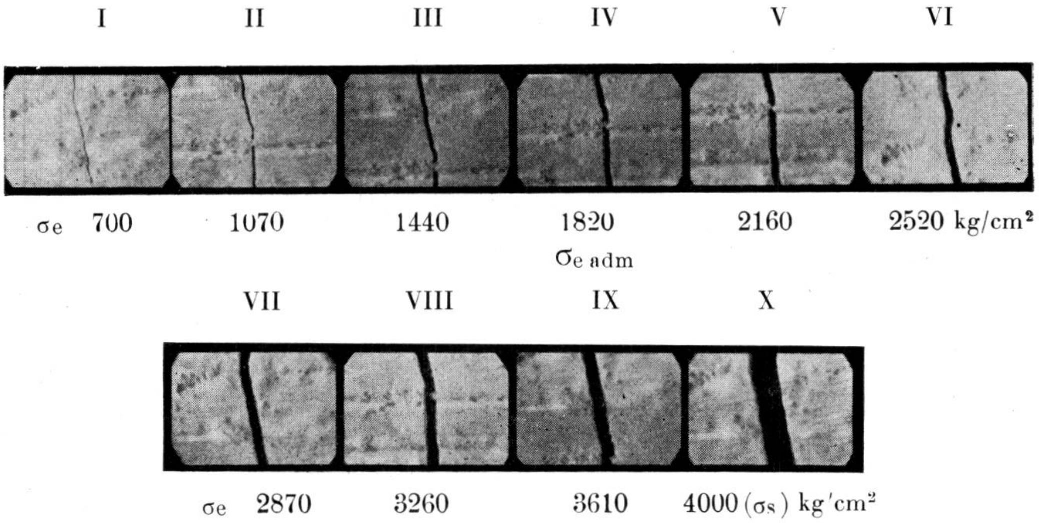

Fig. 5.

Measuring of crack-widths with microscope, enlargement 11,5 times, Dresden tests 1935—36.

b) At the moment when the test beams had attained the calculated permissible stressing value, alcohol was injected to moisten the cracks, after which a coloured liquid was injected. After completion of the test the cracks were opened to see how far the liquid had entered. The depth measured is called "depth of crack t".

II. The dependence of the cracking point stress σ_{eR} on shape of cross section, type of supporting, quality of concrete and percentage of reinforcement.

1) *The quantities on which the safety against cracking depends.*

a) As regards the shape of cross section, for uni-axial stress conditions (e. g. beams over two and more supports) distinction must be made between the following points:

α) Slabs with rectangular section (Issue 66)[7],

β) T-beams with broad flanges, and T-beams with narrow web (Dresden tests 1935),

γ) different forms of cross sections (e. g. factory-made reinforced concrete building elements, Issue 75)[8].

b) *The supporting* on all sides of crosswise reinforced slabs leads to duo-axial stress conditions, which are favourable in respect to the safety of cracks. (Dresden slab tests, Issue 70)[9].

c) The quality of concrete is best indicated by the cube strength after 90 days[9]

$$W_{b90} = 1,15\, W_{b28} \qquad (11$$

and the corresponding tensile strength[9] is expressed by

$$K_z = 0,09\, W_b. \qquad (12)$$

d) *The percentage of reinforcement* ist expressed as usual by[10]

$$\mu = \frac{F_e}{b \cdot h} \qquad (13)$$

F_e = cross sectional area of steel,
b = width of compressive area of concrete,
h = effective height of rectangular section or T-beam.

2) *The cracking point stress σ_{eR} for St 37 and St 48 for slabs (with rectangular section) in dependence on the cube strength W_{b90} and the percentage of reinforcement μ, as attained from Dahlem tests 1928,* are shown in Fig. 6.

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7 See Issue 66, German Commission for Reinforced Concrete (D.A.f.E.B.) *H. Burchartz* and *L. Krüger:* Dahlem Tests with steel reinforced beams Part I, p. 31, Berlin 1931. Publ. W. Ernst and Son. „Dahlemer Versuche mit stahlbewehrten Balken."

8 See Issue 75 of D.A.f.E.B. *W. Gehler* and *H. Amos:* Tests with factory-made reinforced concrete building elements, p. 42. Berlin 1934. W. Ernst and Son. „Versuche mit fabrikmäßig hergestellten Eisenbetonbauteilen."

reinforced slabs, p. 119, Berlin 1932, W. Ernst and Son." Versuche mit kreuzweise bewehrten

9 See Issue 70. D.A.f.E.B.: *W. Gehler, H. Amos* and *M. Bergsträsser:* Tests with crosswise Platten."

10 See *W. Gehler:* Explanations on the Regulations relating to Reinforced Concrete 1932. 5th edition, pp. 33, 300 and 302, Berlin 1933. W. Ernst and Son. „Erläuterungen zu den Eisenbetonbestimmungen 1932."

The corresponding values of safety against cracking are given in Fig. 4. In spite of the irregular positions when plotting the test results due to the difficulty of noticing in time the appearance of cracks, and only decreased in course of time by improving the technique of measuring, the following results are found:

a) The safety against cracking (Fig. 4) as well as the cracking point stress σ_{eR} has higher values for slabs of rectangular sections reinforced in one direction

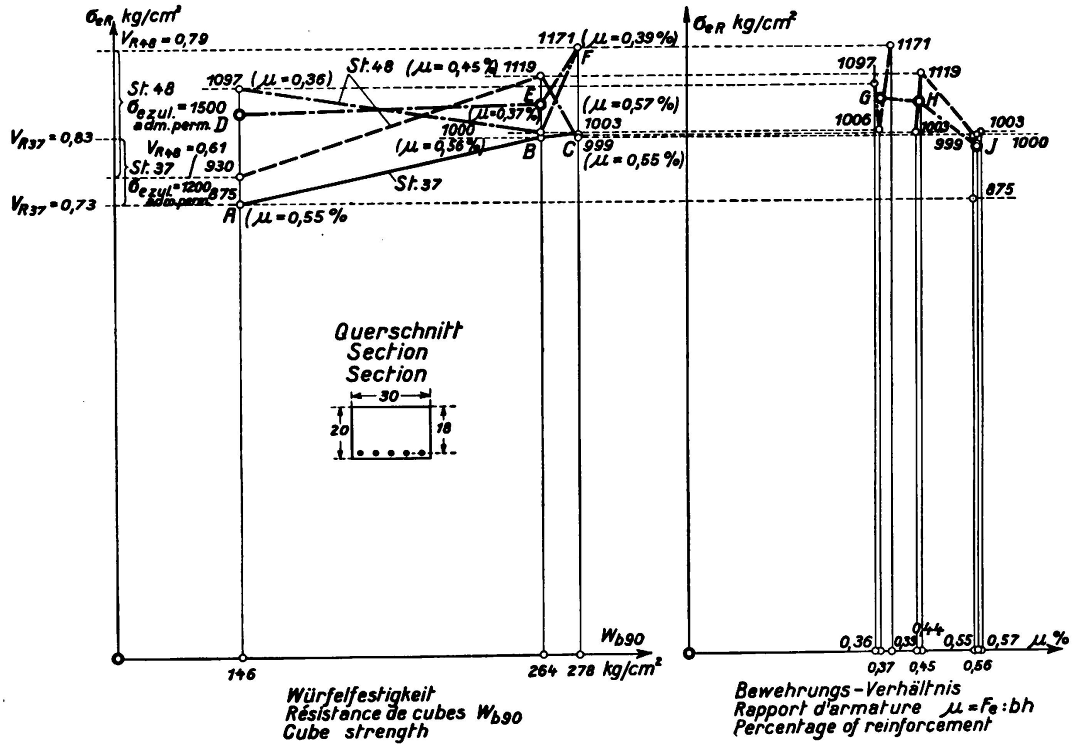

Fig. 6.

Cracking point stress σ_{eR} of St 37 and St 48 for slabs in dependence on the cube strength W_{b90} and the percentage of reinforcement μ.

(h = 16 to 18 cm, d = 18 to 20 cm, b = 30 cm) than for T-beams (see Fig. 6). According to the lines ABC and DEF (mean values) the following results are received for such slabs:

$$\text{for St 37 } \sigma_{eR} = 875 \text{ to } 1000 \text{ kg/cm}^2,$$
$$\text{for St 48 } \sigma_{eR} = 930 \text{ to } 1175 \text{ kg/cm}^2,$$

the yield stress for these cases was $\sigma_{s37} = 3000$ kg/cm² and $\sigma_{s48} = 3900$ kg/cm² with elongations at rupture of 34 % and 28 % respectively. The safety against cracking assumes the following values, according to Fig. 4:

$$\text{for St 37 } v_R = \frac{q_R}{q_{adm}} = 0.73 \text{ to } 0.83, \text{ or a mean of } 0.78,$$

$$\text{for St 48 } v_R = \frac{q_R}{q_{adm}} = 0.61 \text{ to } 0.79, \text{ or a mean of } 0.70.$$

Based on this the mean value for *safety against cracking* for slabs can be put down as:

$$v_R = \frac{q_R}{q_{adm}} = \frac{3}{4} \qquad (16)$$

b) *With increasing cube strength the cracking point stress* σ_{eR} *increases also* (see lines ABC and DEF).

c) but decreases with increasing percentage of reinforcement $\mu = \frac{Fe}{bh}$ (see line GHJ).

d) For the arbitrarily selected permissible stress $\sigma_{e\,adm} = 1500$ kg/cm² for St 48 *the safety against cracking is almost that of St 37* with $\sigma_{e\,adm} = 1200$ kg/cm² (see Eq. 15).

3) The safety against cracking v_R for crosswise reinforced, rectangular slab, supported on all sides, has been extensively studied on the basis of the Dresden slabs tests of 1932[11]. The surprisingly high values v_R for square slabs, supported on all sides were:

$$v_R = \frac{q_R}{q_{adm}} 1.36 \text{ to } 2.05 \text{ or a mean of } 1.8 \tag{17}$$

for St 37 with $\sigma_{e\,adm} = 1200$ kg/cm² and a corresponding stressing of steel of

$$\sigma_{e\,R} = 1630 \text{ kg/cm}^2 \text{ to } 2460 \text{ kg/cm}^2$$
$$\text{or a mean of } 2160 \text{ kg/cm}^2. \tag{18}$$

The loading-deflection line in Fig. 2c shows the appearance of the first cracks at a point marked R, which lies level with the permissible loading q_{adm}. For the static effect of slabs the fracture point R′ is decisive. This point is the intersection point of the two lines $\overline{OR'}$ and $\overline{CR'}$, and shows itself distinctly in the load-deflection line. This point has a similar significance as the limit of proportionality in the stress-strain diagram for building steel (Fig. 1). This fact is of fundamental importance, as it allows reinforced concrete slabs to be calculated as isotropic slabs, up to the loading of $q_R = q_{adm}$, and it is therefore admissible to base the safety against cracking on equation 10 as well as on equation 9.

For *slabs supported at the four corners* (advance-tests for mushroom-slabs) the following results were found for square and rectangular slabs ($l_x : l_y = 2:1$)

$$v_R = 1.38 \text{ to } 1.40 \tag{19}$$

hence for $\sigma_{e\,adm} = 1200$ kg/cm² St 37:

$$\sigma_{eR} = 1650 \text{ to } 1680 \text{ kg/cm}^2 \tag{20}$$

4) The cracking point stress for St 37 and St 52 for T-beams in relation to the cube strength $W_{b_{90}}$ (Dresden Tests of 1928, issue 66 and of 1935/36)[12] (see Fig. 7 and 4).

a) For St 37 with low grade concrete ($W_{b_{28}} = 104$ kg/cm² and 145 kg/cm²) and $\mu = 0.34\,\%$ was found:

$$\sigma_{eR} = 590 \text{ to } 615, \text{ or a mean of } 600 \text{ kg/cm}^2 \text{ and}$$
$$v_R = 0.4 \text{ to } 0.5 \tag{21}$$

(see line CD in Fig. 4). These results apply for webs of $b_o = 20$ cm and $b_o = 12$ cm, and again prove the correctness of the well-known values of the

[11] See Footnote 9.

[12] See Issue 66, D.A.f.E.B., *W. Gehler* and *H. Amos*, 2_{nd} Part, Berlin 1931, W. Ernst and Son.

test series with T-beams made by D. A. f. E. B. (see AB in Fig. 7). The earlier tests of 1928 (issue 66) for St 37 and St 48 yielded considerably smaller values: σ_{eR} = 424 to 520, or a mean of about 500 kg/cm² (See points K to M in Fig. 7 and GH in Fig. 4).

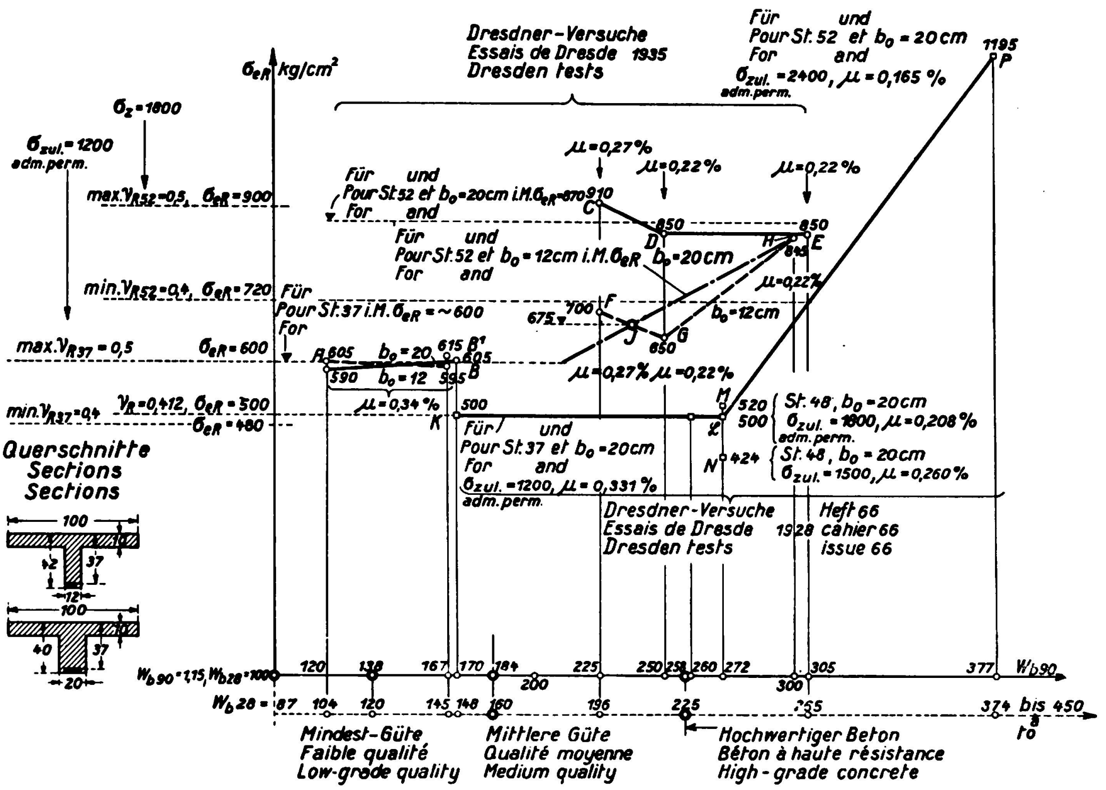

Fig. 7.

Cracking point stress σ_{eR} of St 37 and St 52 for T-beams with web of b_0 = 20 and 12 cm respectively in relation to cube strength.

b) Corresponding tests with St 52 (for σ_s = 4310 kg/cm²) with concrete of medium and high quality and webs of b_o = 20 cm (line CDE in Fig. 7 and EF in Fig. 4) led to

$$\sigma_{eR} = 830 \text{ to } 910 \text{ kg/cm}^2 \text{ or a mean of } 870 \text{ kg/cm}^2$$

$$\nu_R = \frac{\sigma_{eR}}{\sigma_{adm}} = \frac{870}{1800} = \text{about } \frac{1}{2}, \tag{22}$$

But for webs of b_o = 12 cm (see line FGH in Fig. 7 or the rising line JH, where J represents the centre of gravity of FG) we found

$$\sigma_{eR} = 650 \text{ to } 845 \text{ kg/cm}^2$$
$$\text{hence } \nu_R = 0.36 \text{ to } 0.47. \tag{23}$$

From this results: *For T-beams with St 52 not only the quality of concrete, but also the width b_o of the webs is of influence as regards the safety against cracking. If the width of the ribs is sufficient (here b_o = 20 cm) and for $W_{b28} \geqq 200$ kg/cm² then the safety against cracking $\nu_R = \frac{1}{2}$ is maintained, whilst for narrower ribs (here b_o = 12 cm) this is only the case if $W_{b28} \geqq 250$ kg/cm².*

c) The Dresden tests of 1927 (Issue 66) for T-beams with $b_o = 20$ cm used St 52 and a special quality cement with $W_{b28} = 374$ kgkg/cm² and were designed for a reinforcement with $\sigma_{e\,adm} = 2400$ kg/cm² giving a percentage $\mu = 0.165$ %. For this case the cracking point stress had a value of

$$\sigma_{e\,R} = 1195 \text{ kg/cm}^2 \text{ and}$$

$$v_R = \frac{1195}{2400} = \text{about } \frac{1}{2}. \tag{24}$$

(see point P in Fig. 7 and H in Fig. 4). *The higher the value of W_{b28} and the smaller the percentage μ, the higher the cracking point stress $\sigma_{e\,R}$.*

5) *The cracking point stress $\sigma_{e\,R}$ of St 37 and St 52 for T-beams in relation to the percentage μ of reinforcement* (See Fig. 8).

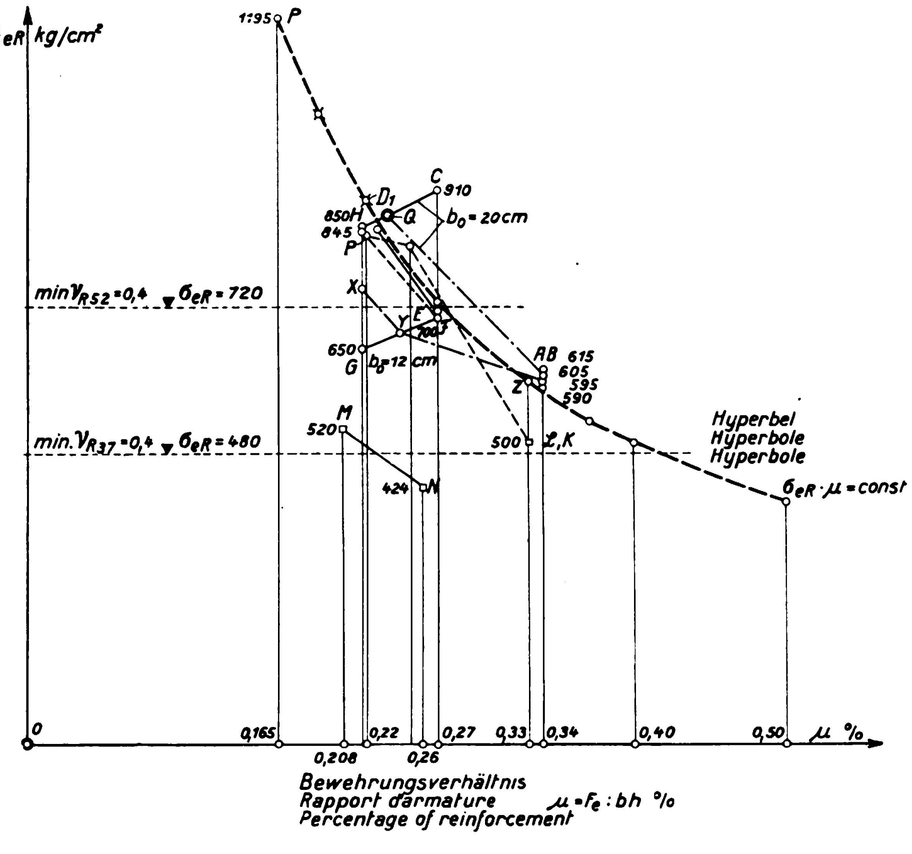

Fig. 8.

Cracking point stress σ_{eR} of St 37 and St 52 and special steels in relation to the percentage of reinforcement μ for T-beams with width of web $b_0 = 20$ and 12 cm respectively.

For the points A to P in Fig. 7 the abscissae give the μ-values and the ordinates the values of $\sigma_{e\,R}$, and in spite of the irregular positions of these points we find lines distinctly falling to the rigth e. g. line PLK, and MN. Establishing further the gravity centres X, Y and Z, it will be noticed that presumably these falling lines will not be straight. It follows, therefore, that the smaller the cross sectional area of reinforcement the conditions remaining otherwise unchanged, (under consideration of the required safety against rupture), the higher the value for cracking point stressing σ_{eR}. If the yield limit σ_s is higher for one kind of steel than for another, it is permissible in respect to the safety against rupture to adopt a higher value of permissible stressing $\sigma_{e\,adm}$. On doing so we receive smaller values for the required cross sectional area of steel and consequently a smaller percentage $\mu = \frac{Fe}{bh}$, but the cracking point stress σ_{eR} will increase. The rising of $\sigma_{e\,adm}$ is, however, limited in so far as $v_R = \frac{\sigma_{ek}}{\sigma_{e\,adm}} = \frac{1}{2}$. Since Young's modulus (modulus of elasticity) is equal for all qualities of steel, the elongation increases and with it the danger of cracking; this is proportional to the stressing, but *independent of the yield limit.* The yield limit, however, is of *direct importance for the safety against rupture* but only *indirectly responsible for the safety against cracking.*

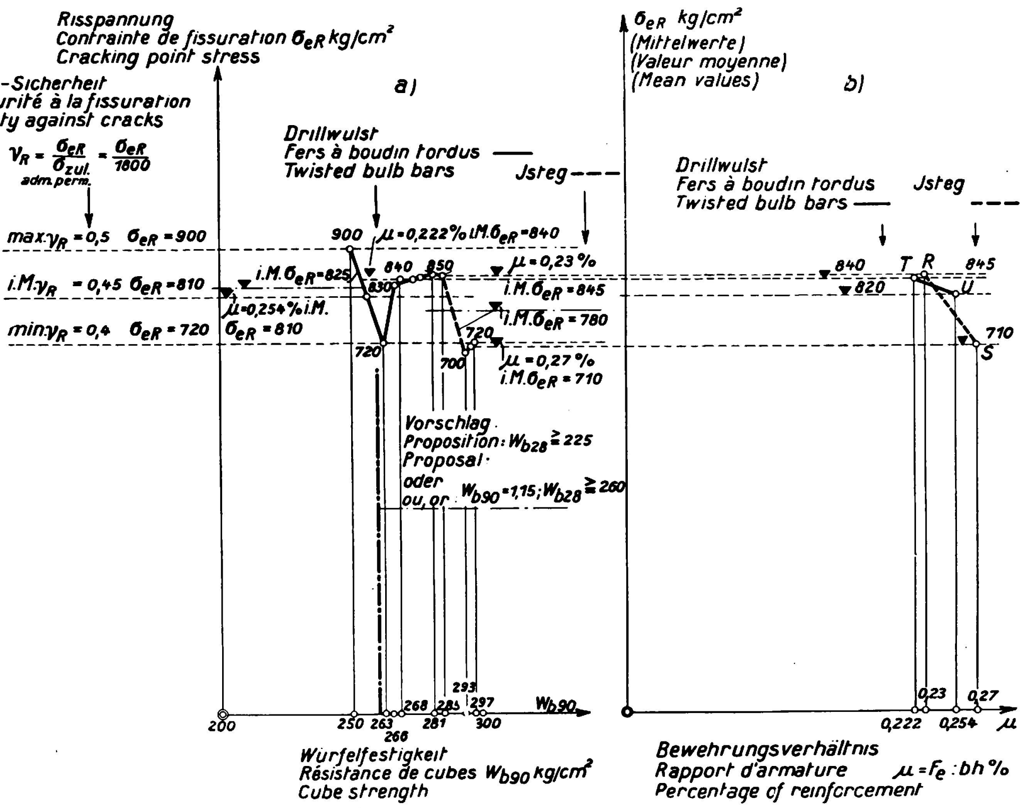

Fig. 9.

Cracking point stress σ_{eR} of special steels for T-beams with web $b_0 = 20$ cm wide, in relation to: a) cube strength W_{b90}. b) percentage of reinforcement μ.

6) *The cracking stress* σ_{eR} *for T-beams reinforced with special quality steel* (Fig. 9).

a) For the Dresden tests of 1936 carried out with two types of steel (Twisted bulb bars with $\sigma_s = 4640$ kg/cm² and $\sigma_B = 6050$ kg/cm² and Isteg-steel with $\sigma_s = 3720$ kg/cm² and $\sigma_B = 4940$ kg/cm²) and by plotting the σ_{eR} — W_b line and σ_{eR} — μ line, Figs. 7 and 8, we receive:

$$\sigma_{eR} = 700 \text{ to } 900 \text{ kg/cm}^2 \text{ and}$$
$$v_R = 0.4 \text{ to } 0.5 \text{ or a mean of } 0.45 \qquad (25)$$

or about the same results as for St 52.

b) Also in this case the σ_{eR} values decrease with increasing μ (see lines TU and RS).

III. Attempt to establish a function for the cracking point stress in relation to the quality of concrete, shape of section and percentage of reinforcement.

1) The tests (Fig. 4 to 9) reveal that:

a) σ_{eR} increases proportionally to the quality W_B of concrete and also with the tensile strength $\sigma_{bz} = 0.09\ W_b$.

b) σ_{eR}' however, decreases with increasing values of μ.

c) These two conditions can be expressed by the following function:

$$\sigma_{eR} \cdot \mu = (0.09\ W_b) \cdot C \tag{26}$$

Considering that $\mu = \frac{F_e}{F_b}$ and that the term on the left side contains $(\sigma_{eR} \cdot F_e) = Z_e$ (tensile force in steel) which accordingly demands on the right side the tensile force in concrete of $Z_b = \sigma_{bz} \cdot F_{bz} = (0.09 \cdot W_b) \cdot F_{bz}$ we can express 26 in the following manner:

$$k \cdot \sigma_{eR}\left(\frac{F_e}{F_b}\right) = 0{,}09 \cdot W_b \left(\frac{F_{bz}}{F_b}\right) \tag{27}$$

or:

$$k \cdot \sigma_{eR} \cdot F_e = (0.09 - W_b) \cdot F_{bz} \tag{28a}$$

$$k \cdot Z_e = Z_b \tag{28}$$

In this equation Z_b represents the fractured tensile zone = depth t of crack x width of rib b_o and k is a coefficient or percentage still to be determined.

The equation 28 established entirely by statistical interpretation of test results allows for the following physical conception. The tensile area of concrete F_{bz} cracks at the very moment the stressing in steel has the value σ_{eR}, due to sudden exhausting of the tensile strength in concrete σ_{bz}. Therefore the tensile force

$$z_b = \sigma_{bz} \cdot F_{bz} = (0.09\ W_b) \cdot F_{bz}$$

previously held by concrete will be handed over to the steel as an additional stressing. The magnitude of the tensile force Z_b in concrete can be expressed as a certain fraction (in %) of the tensile force in steel $Z_e = \sigma_{eR} \cdot F_e$ acting at that very moment in such a way that

$$Z_b = k \cdot Z_e.$$

On the left side of the basic equation 28 (steel side) the reinforcing ratio $\frac{F_e}{F_b} = \mu$ enters into account, which we shall call *"Form coefficient of the sectional area of steel"* (Formziffer des Eisenquerschnittes), correspondingly on the right side (concrete side) a new ratio appears:

$$\frac{F_{bz}}{F_b} = \alpha, \tag{29}$$

This new ratio will be termed *"Form coefficient of the concrete tensile area"* (Formziffer der Betonzugzone). With these explanations equations 27 can now be written:

$$k \cdot \sigma_{eR} \cdot \mu = (0.09\ W_b) \cdot \alpha \tag{30}$$

2) a) Now only the factor k is left for determination. In compressed sections of reinforced concrete ($F = F_b + 15\,F_e$), the concrete area is only capable of taking $\frac{1}{n}$ of the stresses of steel; this is expressed by:

$$n = \frac{E}{E_b} = \frac{2\,100\,000}{140\,000} = 15 \quad \text{or}$$

for $E_b = 210\,000\ \text{kg/cm}^2$ is $n = 10$.

But for the tensile area of concrete (under consideration) we shall have to introduce a corresponding coefficient:

$$n_z = \frac{E}{E_{bz}} = \frac{2100000}{250000} = 8{,}4 \qquad (31)$$

(modulus of elasticity E_{bz} for tension according to issue 66).

b) The formation of a separating crack in the tensile area not only depends on the elastic behaviour, but also on the *brittleness of concrete.* As is well known, the tensile strength does not keep pace with the increasing compressive strength of concrete, therefore it seems advisable to indroduce for each of the *three most common qualities of concrete with minimum cube strengths of* W_{b28} = *120, 160 and 225 kg/cm²* a separate *coefficient s expressing the brittleness.*

Hence according to equation 31 we receive

$$k = \frac{s}{n_z} = \frac{s}{8{,}4} \qquad (32)$$

c) In Fig. 10 the tensile strength K_{bz} of un-reinforced concrete beams (chiefly for $55 \cdot 15 \cdot 10\ \text{cm}^3$ loaded with two point loads) is plotted in relation to W_{b90} (see e. g. line DE and FG). For the three qualities of concrete mentioned (see points A, B and C) we may assume the values:

$$K_{bz} = 20,\ 30,\ \text{and}\ 40\ \text{kg/cm}^2\ \text{respectively.} \qquad (33)$$

The values of the tensile strength of concrete prisms ($75 \cdot 20 \cdot 26\ \text{cm}^3$) which are shown as well, remain far behind, for increasing cube strength, chiefly on account of the great difficulty of establishing a fully centric tensile action (see lines D′E′, F′G′ and H′J′). These values are not useful for strength consideration.

Substituting in Eq. 28, in place of ($0.09\ W_b$), the tensile strength K_{bz} sought, we see that K_{bz} can be calculated without difficulty from *the measured depth of cracks t* simply by assuming a fixed value of k (or for s) for each of the three qualities of concrete. By choosing for s the following values, which are easy to remember,

$$\left.\begin{array}{ll} s = \frac{1}{3}\ \text{hence}\ k = \frac{s}{8{,}4} = \frac{4}{100} & \text{for low-grade concrete } (W_{b28} = 120 \text{ to } 160\ \text{kg/cm}^2) \\ s = \frac{2}{3}\quad ,,\quad k = \frac{8}{100} & \text{for medium grade-concrete } (W_{b28} = 160 \text{ to } 225\ \text{kg/cm}^2) \\ s = 1\quad ,,\quad k = \frac{12}{100} & \text{for high-grade concrete } (W_{b28} = 225\ \text{kg/cm}^2) \end{array}\right\} \qquad (34)$$

we receive the following mean values from our tests for these three ranges:

$$K'_{bz} = 18.29 \text{ and } 39 \text{ kg/cm}^2.$$

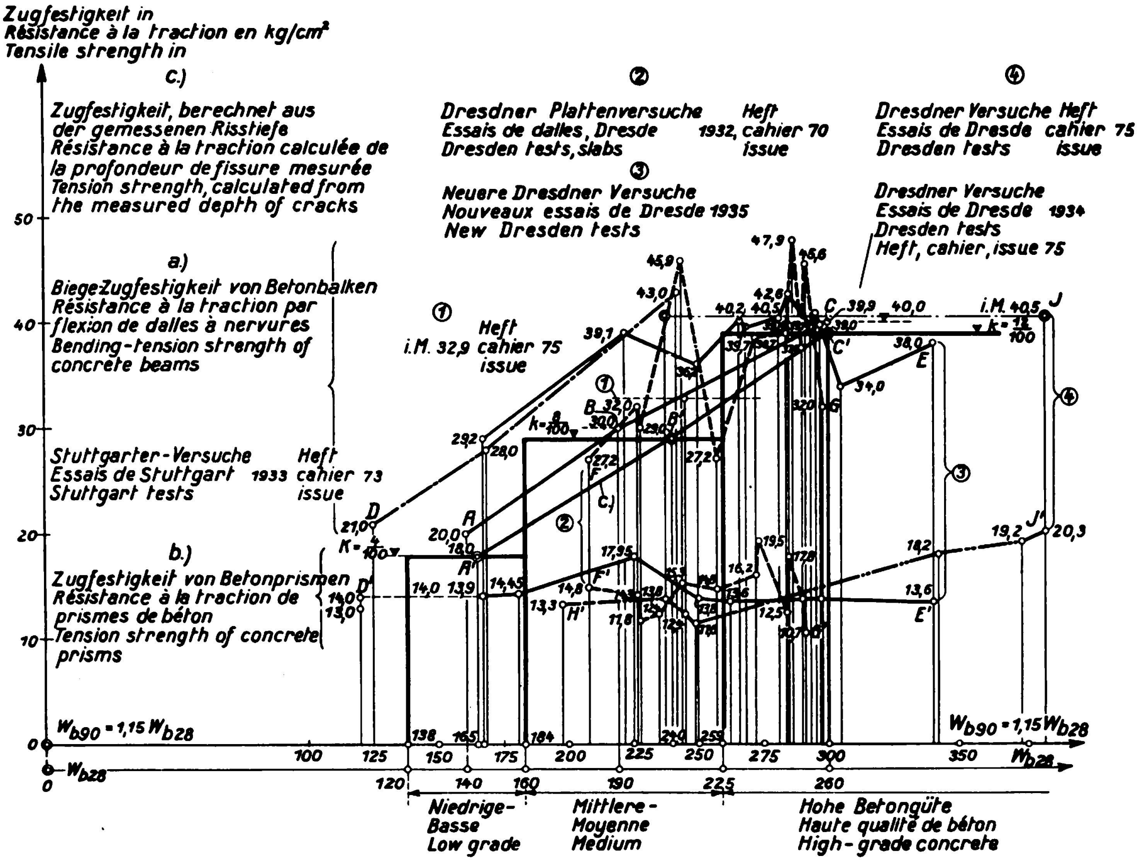

Fig. 10.

Tension strength of concrete in relation to cube strength (Dresden Tests 1928 to 1936).

The points A′, B′ and C′ represent these values, and have the same incline as for the points, A, B and C. Hence according to Eq. 30 the relation expressing the cracking point stress for the three ranges can be written:

$$\sigma_{eR} = \frac{9}{4} W_b \cdot \frac{F_{bz}}{F_e}, \quad \sigma_{eR} = \frac{9}{8} W_b \cdot \frac{F_{bz}}{F_e} \quad \text{and} \quad \sigma_{eR} = \frac{9}{12} W_b \cdot \frac{F_{bz}}{F_e}, \tag{35}$$

wherein $F_{bz} = b_o \cdot t$ stands for the fractured tensile area.

3) *Examples.*

a) For point B_1 in Fig. 7 (St 37) with $b_o = 12$ cm, $F_e = 12.72$ cm², $W_{b90} = 167$ kg/cm² and a load corresponding to the permissible stress $\sigma_e = 1200$ kg/cm² shall be calculated the depth of the crack. With Eq. 34 for low-grade concrete and $k = \frac{4}{100}$ from Eq. 28 we receive a depth of crack of:

$$t = \frac{F_{bz}}{b_o} = \frac{1}{b_o} \cdot \frac{k \cdot \sigma_{eR} \cdot F_e}{0{,}09 \cdot W_b} = \frac{1}{12} \cdot \frac{4}{100} \cdot \frac{1200 \cdot 12{,}72}{0{,}09 \cdot 167} = 3{,}4 \text{ cm}, \tag{36a}$$

while $t_1 = 3.5$ cm was measured in the test.

b) For point D in Fig. 7 (St 52, with $b_o = 20$ cm, $F_e = 8.15$ cm^2, $W_b = 150$ kg/cm^2) and again for a load corresponding to the permissible stress, in this case $\sigma_{e\,adm} = 1800$ kg/cm^2, shall be calculated the depth of crack. Since in this case $W_{b28} = W_{b90} : 1.15 = 217$ kg/cm^2, hence for medium grade concrete we receive with $k = \frac{8}{100}$ (according to Eq. 34) from Eq. 28a:

$$t = \frac{1}{20} \cdot \frac{8}{100} \cdot \frac{1800 \cdot 8{,}15}{0{,}09 \cdot 250} = 2{,}6 \text{ cm}, \tag{36b}$$

the crack measured was $t_1 = 3$ cm.

c) For point E in Fig. 7, for a load which for $\sigma_{e\,adm} = 1800$ kg/cm^2 produced a depth of crack $t_1 = 3.00$ cm (based on $W_{b\,90} = 305$ kg/cm^2, $b_o = 20$ cm and $F_e = 8.17$ cm^2) the stress shall be calculated at the moment of the appearance of the first cracks. Assuming that the depth of cracks is more or less proportional to the stress, and that the safety against cracks averages about $\nu_R = 0.5$ then the depth for the first crack of $t = 0.5 \cdot 3.0 = 1.5$ cm has to be assumed. Hence according to Eq. 34 and for $k = \frac{12}{100}$ (high-grade concrete) we receive for the stress sought:

$$\sigma_{eR} = \frac{0{,}09 \cdot W_b \cdot b_o \cdot t}{k \cdot F_e} = \frac{9}{12} \cdot \frac{305 \cdot 20 \cdot 1{,}5}{8{,}17} = 840 \text{ kg/cm}^2$$

instead of $\sigma_{eR} = 850$ kg/cm^2 as obtained from the test.

4) By giving the equations 28 and 30 the following form:

$$\sigma_{eR} = \frac{1}{k} \cdot 0{,}09 \cdot W_b \cdot \frac{F_{bz}}{F_e} = \frac{1}{k} \cdot 0{,}09 \cdot W_b \cdot \frac{\alpha}{\mu}, \tag{37}$$

the conclusion below can be derived:

a) The cracking point stress σ_{eR} and with it safety against cracking $\nu'_R = \frac{\sigma_{eR}}{\sigma_{e\,adm}}$ is, under otherwise identical circumstances, proportional to the cube strength W_b and also, since $F_{bz} = b_o \cdot t$, proportional to the width of the rib b_o,

b) but inversely proportional to the section of steel F_e or the percentage of reinforcement. As, under similar circumstances

$$\sigma_{eR} \cdot \mu = \text{const.} \tag{38}$$

the σ_{eR} — μ — lines in Figs. 6, 8 and 9 are parts of a quadratic hyperbola, which is shown dotted in Fig. 8.

c) The *new "form coefficient" for concrete sections* (See Eq. 29):

$$\alpha = \frac{F_{bz}}{F_b}$$

corresponds to the "form coefficient" of the sectional area of steel, which is known as the ratio of reinforcement $\mu = \frac{F_e}{F_b}$ (mostly expressed in %) wherein $F_b = b \cdot h$.

IV. The importance of the shape of cross section in respect to the safety against cracking.

For the Dresden tests with factory made reinforced concrete building elements (1934, Issue 75, German Commission on Reinforced Concrete) the safety against cracks for very slender beams ($1:h = 5.82:0.81 = 32$) according to Eq. 9b was fixed by the ratio $\nu_R = M_R : M_{adm}$. Based on a safety factor of 3, the permissible bending moment M_{adm} was stipulated to be $M_{adm} = \frac{1}{3} M_B$. In addition to this, the stress σ_{e_1} had been calculated based on the moment M_{adm}. With those assumption the cracking point stress can be expressed approximately by

$$\sigma_{eR} = \nu_R \cdot \sigma_{e_1} \qquad (39)$$

It is now possible, based on Eq. 36a, to calculate for the eight sections of Fig. 11 the cracking depth t, and to state by means of the ratio t : e (e = concrete cover measured from bottom edge to centre of bars) a *quality factor for*

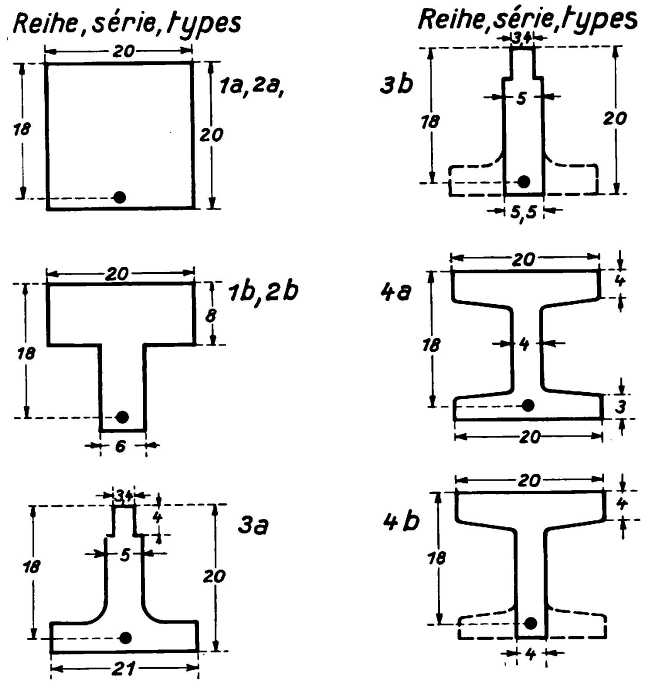

Fig. 11.
Shape of cross sections and safety against cracks.

the safety against cracks for each particular type of section. For these cases as shown in Table II the elastic ratios $n_z = \frac{E}{E_{bz}}$ (Eq. 31) were worked out, based on the values E_{bz} received from tests for the various qualities of concrete. It has to be pointed out, however, that the cross sectional form Ia, with $W_{b28} = 198$ kg/cm² was for medium quality concrete with a brittleness coefficient of $s = \frac{2}{3}$ (Eq. 34); the other forms 1b to 4b are for high quality concrete with s = 1. Based on a constant cross sectional area of steel $F_e = 2.55$ cm² were calculated the cracking depth t according to Eq. 36a and the *quality factor of the safety against cracks* e : t (concrete cover over bars 1 = 1.9 cm).

Table II.

Type of section	b_o	W_b	$v_R \cdot \sigma_{e\,adm} = \sigma_{eR}$	$n_z = E : E_{bz}$	s	$\frac{1}{k} = \frac{n_z}{s}$	t according to Eq. 36 cm	e:t (e=1,9 cm)
1 a	20	198	965	11,05	²/₃	16,6	0,41	4,6
1 b	6	237	998	9,46	1	9,46	2,10	0,9
2 a	20	367	1440	7,14	1	7,14	0,78	2,4
2 b	6	384	1270	7,14	1	7,14	2,18	0,9
3 a	21	394	875	7,14	1	7,14	0,42	4,5
3 b	5,5	377	680	7,14	1	7,14	1,30	1,5
4 a	20	374	980	7,27	1	7,27	0,51	3,7
4 b	4	342	785	7,50	1	7,50	2,16	0,9

From table II it can be seen that the quality factor of the safety against cracking e : t > 2 not only occurs for the two rectangular sections of types 1a and 2a but also for type 3a (reversed T-beam section) and type 4a (I-shaped section). The most unfavourable quality factor of the safety against cracks (e : t = 0.9 < 1) stands for T-beam sections of types 1b, 2b and 4b, the section type 3b with small width of rib b_o = 5.5 cm ranges in between the other quality classes[13].

The foregoing investigations might permit the conclusion to be drawn that the sections e to h proposed by Fig. 12 may give increased safety against cracking for wide span girder bridges.

V. *The permissible width of cracks* was laid down empirically by the Dresden tests of 1936, experience made in practice having shown that T-beams dimensioned according to regulation with $\sigma_{e\,adm}$ = 1200 kg/cm² for St 37 had proved safe against the danger of rust. The width of cracks photographed in 23 times enlargement and measured at the level of the reinforcement, are shown in table III.

Table III

Measured width of cracks b_R, at $\sigma_{e\,adm}$, in ¹/₁₀₀₀ mm.

Kind of steel	St 37	St 52	Isteg	Twisted bulb bar
Nos. of beams	2 + 3	4 + 4	4	4
width of ribs b_o = 20 cm	70 to 70 or mean *70*	40 to 130 or mean *90*	80 to 110 or mean *94*	75 to 120 or mean *89*
b_o = 12 cm	25 to 60 or mean *41*	10 to 70 or mean *35*	—	—

[13] As regards the carrying capacity for the various types of section another order naturally applies, as will be seen also from Issue 75.

From the width of the cracks measured for a stressing of $\sigma_{e\,adm}$, the distance between the cracks and the number of cracks on a testing specimen, it is possible by using the law of proportionality $\frac{\Delta l_1}{l_1} = \frac{\sigma_e}{E}$ to arrive at the stressing of steel (elongation $\Delta l_1 = b_R$, observed length, or measuring length) and to draw further conclusions (e. g. according to Fig. 17, see E 2). From table III it follows that:

1) The permissible width of cracks can be accepted as about:

$$b_{R\,adm} = \frac{125}{1000}\,mm = \frac{1}{8}\,mm \tag{40}$$

2) The striking difference in the greater crack widths for $b_o = 20$ compared with $b_o = 12$ forms a confirmation of the physical conception (see under VI 1)

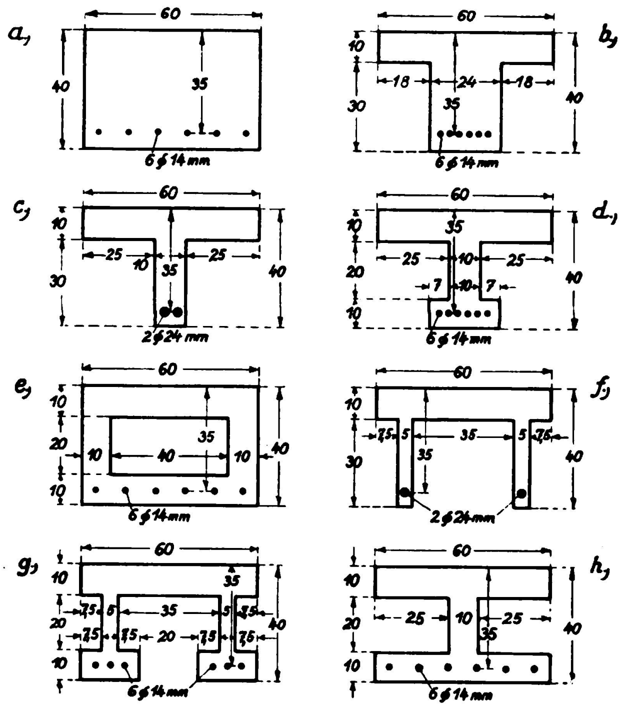

Fig. 12.
Shape of cross sections for future tests.

which has its expression in our Ev. 28. In both cases at the moment of first appearance of cracks, for equal depth of cracks (e. g. t = 3 cm) the ratio between the released concrete tensile forces is the same as the ratios of the areas $b_o \cdot t$; hence

$$Z_{b20} : Z_{b12} = 20\,t : 12\,t = 5 : 3 = 1.7,$$

according to table III for measured width of cracks we receive $W_{20} : w_{12} = 70 : 41 = 1.7$ or the same value as above. *The greater the suddenly released tensile force Z_b, the wider the crack opens. If according to Eq. 40 the permissible width of cracks is limited to $b_{R\,adm} = \frac{1}{8}$ mm, then this limit prevents the constriction of the gross section of steel F_e going too far* (we had $\mu = 0.34$ to 0.22 %). These intricate relations, however, can only be solved by further experiments.

VI. *Summary of conclusions in respect to the safety against cracking, based on the Dresden tests of 1928 and 1935.*

1) *Physical conception.* At the moment of the appearance of the first cracks the part section $F_{bz} = t \cdot b_o$ and with it the corresponding tensile force are eliminated. The amount of tensile force in the section is governed by Eq. 28 and is expressed as a fraction of the tensile force in steel $Z = \sigma_{eR} \cdot F_e$ and amounts to 4, 8 or 12 % according to the quality of concrete.

It may be mentioned that this additional amount of tensile force only occurs at the place of rupture, but not at unfractured places.

2) The tests show that with regard to safety against cracking the *cross sectional shape of the reinforcement* diminishes in importance. But contrary to this, the *percentage of reinforcement* μ is of a decisive nature. The *smaller the cross section of steel* in comparison to the section of concrete and the width b_o of the concrete tensile area, *the greater the safety against cracking.* A restriction is only given by the fixing of the *permissible width of cracks* (according to Eq. 40) which is limited to $b_{R\,adm} = \frac{1}{8}$ mm. The wider the rib the wider the crack (F_e remaining constant).

3) Safety against cracking increases considerably with *increasing quality of* concrete. But since the brittleness becomes more pronounced with using cement of high compression strength (or the ratio between tensile strength Z and compressive strength D becoming smaller), this increase in compressive strength can only show itself in a very restricted limited manner in respect to the safety against cracking with the brands of cement in use nowadays.

4) As regards *the shape of cross section,* it is to be expected that the *application of I-shaped and box-shaped cross sections* to structures of wide spans proves favourable in respect to the safety against cracking and the carrying capacity. On account of this possibility the German Commission for Reinforced Concrete suggested that tests be carried out with such sections, using high-grade concrete of about $W_b = 450$ kg/cm² and high-quality steel (dimension of test pieces to be about half the actual sizes required) (See Fig. 12).

5) In view of the higher safety against cracking (see Eq. 16 and 18) of slabs with rectangular section ($v_R = \frac{3}{4}$) compared with T-beams ($v_R = 0.4$ to 0.5), it is advisable to employ *high-quality building steel* for the reinforcement of slabs. At the same time the cross sectional area of steel F_e should be kept down as much as possible, i. e. as far as is permitted by the crack-width $b_{R\,adm}$. Such tests with slabs are urgently required.

6) As regards *safety against cracking for statically stressed T-beams with reinforcement* of St 52, the stipulation for permissible stresses: $\sigma_{e\,adm} = 1800$ kg/cm² is quite justified, as also follows from comparison tests with St 37 ($\sigma_{e\,adm} = 1200$ kg/cm²).

D. The safety against rupture of slabs, and T-beams reinforced with high-grade steel.

I. *The line: carrying-capacity percentage of reinforcement.*

1) *The calculated carrying capacity of rectangular sections reinforced with different percentages of St 37 and St 52 respectively.*

The carrying capacity of rectangular sections has to be calculated for bending, according to the German Regulations for reinforced concrete under the following assumptions:

a) The tensile area of the concrete is not taken into account (so-called calculation according to state II).

b) The ratio of the moduli of elasticity between steel and concrete to be $n = E : E_b = 15$.

Further the permissible stresses to be as under:

c) For concrete to have a safety factor of 3,

$(\nu_B = 3)$, also $\sigma_{b\,adm} = \frac{1}{3} W_b$

$(W_b =$ cube strength$)$.

d) Steel reinforcement to have a safety factor of 2,

$(\nu_e = 2)$, also $\sigma_{e\,adm} = \frac{1}{2} \sigma_s$

$(\sigma_s =$ yield point stress of steel$)$.

In fig. 13 are given the calculated results for rectangular sections, reinforced with St 37 and St 52 respectively, in dependence on the percentage of reinforcement

$$\mu = \frac{F_e}{b \cdot h}$$

The ordinates have the values:

$$y = \frac{M}{bh^2} \text{ (in kg/cm)}^2, \tag{41}$$

(wherein $\sigma_B = \frac{M}{W_i}$ represents the rupture point stress and $W_i = \alpha \cdot bh^2$ the modulus of section[14]. Based on the above the two following ranges can be distinguished:

a) *The range of lightly reinforced sections* (rupture due to exceeding the yield point stress σ_s of steel).

b) *The range of heavily reinforced sections* (rupture due to exceeding the bending compressive stress of concrete).

The tests carried out by the German Commission for Reinforced Concrete particularly the Dresden tests with high-grade steel, showed that the calculated results for the range of lightly reinforced sections coincide well with test results. *The yield point stress of steel was decisive for rupture in these cases* (see

[14] *F. v. Emperger:* Austrian Reinforced Concrete Standards of 1935. „Die Normen für Eisenbeton 1935 in Österreich." Beton und Eisen 1935, Vol. 34, Issue 16, p. 254.

section B and Eq. 7). *For the second range of heavily reinforced sections* it was found that the actual carrying capacity was considerably greater than the calculated values.

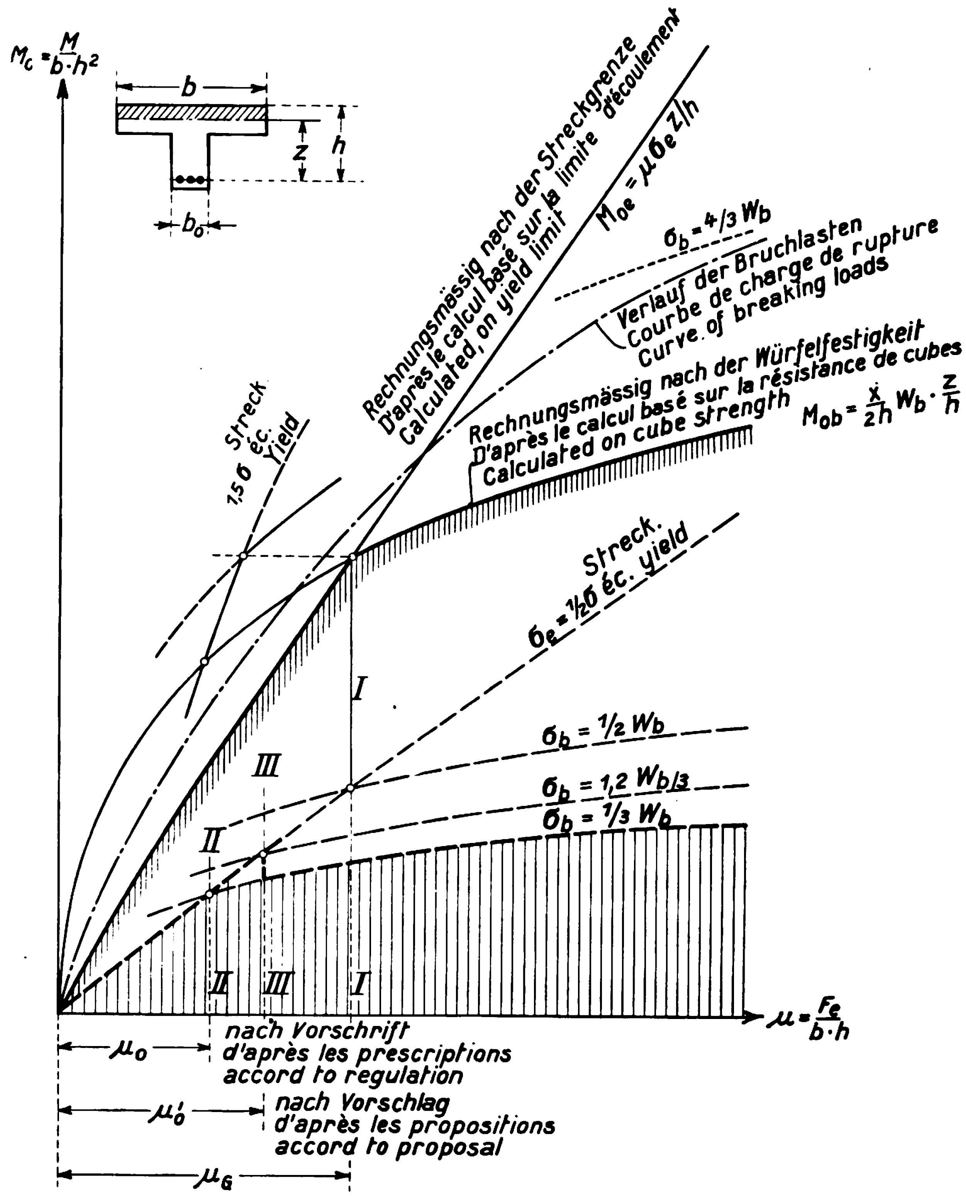

Fig. 13.

The carrying capacity of T-beams in dependence on the percentage of reinforcement (according to Emperger and Haberkalt).

The purpose of the Dresden tests of 1935—1936 was the following:

I. To find by experiment that limit of the percentage of reinforcement which divides the first from the second range.

II. To establish by how much the compression stress of concrete is exceeded. At the state of rupture for tests, compared with the carrying capacity as found by calculation based on cube strength.

II. The importance of tests for the methods of calculation.

Determination of limits.

The question of the carrying capacity of reinforced concrete slabs and T-beams in relation to the percentage of reinforcement was first brought to discussion by the Austrian Commission for Reinforced Concrete under the guidance of *F. Gebauer*. The problem was studied to the extent that a proposal made by *F. v. Emperger* and *C. Haberkalt* should be embodied in the Austrian Regulations (see Fig. 13). This proposal states that the limit between the two ranges — where the yield limit or the cube strength is decisive — should be raised (see point III) against the prescription of the present regulations (see point II). This raise corresponds to an increase of 20 % of the usual permissible stress. But since the permissible stress has been maintained the line for carrying capacity shows an offset. This solution of the problem is not quite satisfactory since it is only based on rectangular sections, and since cases may occur in which the calculated carrying capacity drops on account of additional bars. At places where high compressive stresses in concrete occur, it is customary to provide haunches in the vertical or horizontal direction, also bars are provided in the compressive zone if necessary, but full use can never be made of such bars, apart from being a great hindrance while concreting and reducing the compound action of steel and concrete. Since haunches are mostly not wanted for the sake of appearance, particularly if concrete is in competition with steel structures, it still remains to find a satisfactory solution of this problem[15].

a) For the first range, for which the yield point stress of steel is of a decisive nature, the Dresden tests of 1936 carried out with rectangular beams (Fig. 14) and reinforced with St 37 or Isteg-steel respectively, yielded the following resultats: The carrying capacity line is almost a straight line, for which the ordinates average only 12,5 % higher than the values calculated. Therefore a welcome margin of safety is established. The yield point stress again proves to be decisive for safety in case of the first range. There is therefore no need to change the customary mode of calculation. The Dresden tests (Fig. 14) have given the following limits for the percentage of reinforcement, dividing the first range for which the yield point stress of steel is decisive from the second range which is based on the cube strength of concrete:

for St 37 with $\sigma_s = 2800$ kg/cm² and $W_b = 110$ kg/cm² $\mu_G = 1.82$ %
for Isteg-steel $\sigma_s = 4100$ kg/cm² and $W_b = 110$ kg/cm² $\mu_G = 0.72$ %
for Isteg-steel $\sigma_s = 4100$ kg/cm² and $W_b = 150$ kg/cm² $\mu_G = 0.95$ %

The lines CD for St 37 and EF for Isteg-steel as found by tests for the second range lie considerably higher than the carrying capacities calculated and shown by line AB. It may be mentioned that for a percentage of reinforcement of 1.6 % two of the four points near J are for $W_b = 110$ kg/cm² and two for $W_b = 150$ kg/cm²; in other words for these cases the carrying capacity depends on the cube strength.

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[15] See also *R. Saliger-Vienna:* Experiments on the nature of concrete and with concrete with compressive reinforcement (Versuche über zielsichere Betonbildung und an druckbewehrten Balken) Beton und Eisen 1935, Nr. 1, p. 12.

b) New test series to determine the limit percentage of reinforcement μ_G for slabs and T-beams with different kinds of reinforcement are being carried out.

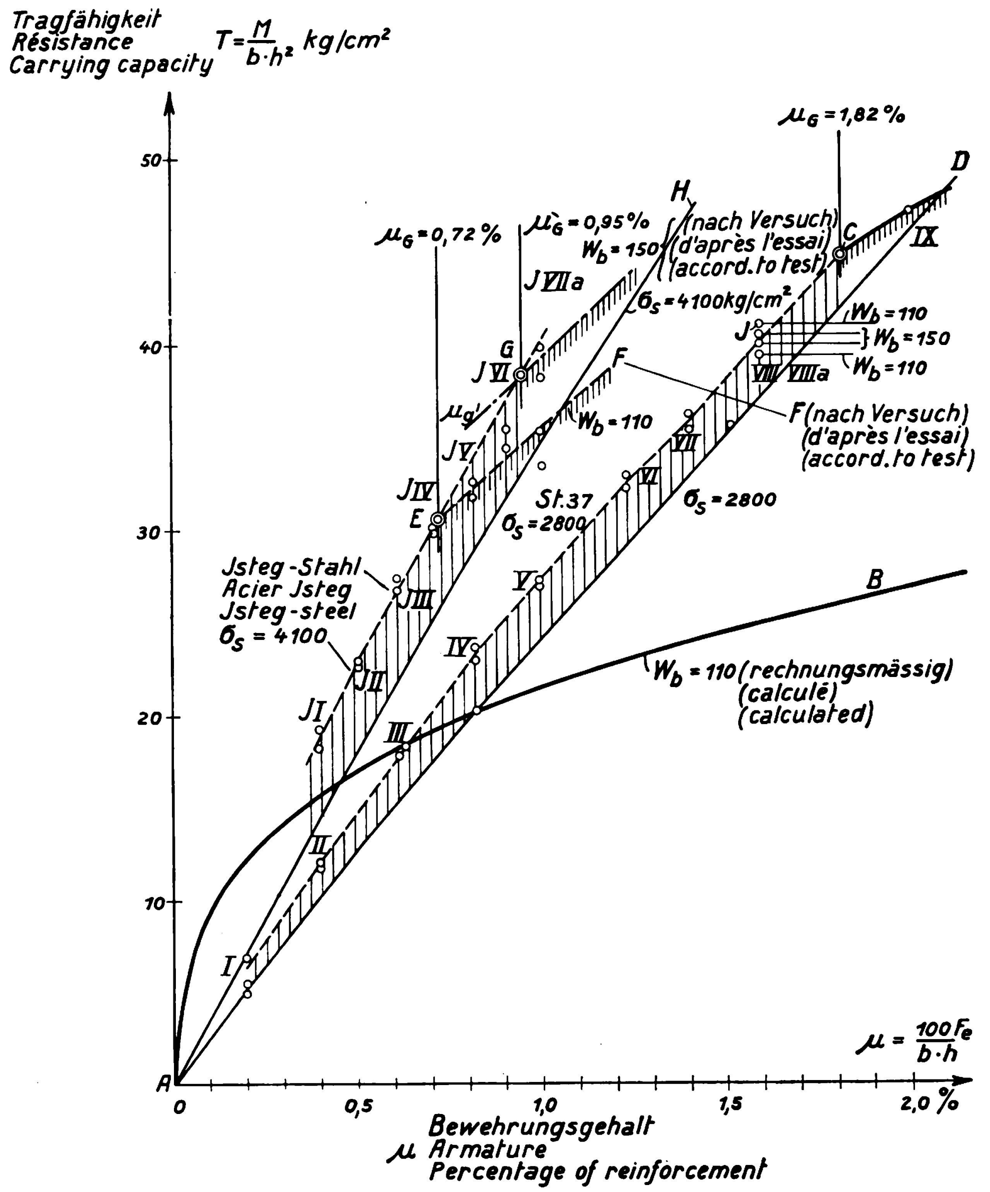

Fig. 14.

The carrying capacity of rectangular beams in dependence on the percentage of reinforcement, according to Dresden tests.

In connection with the Dresden tests a remarkable method of calculation to determine the limit reinforcement has been developed by Dr. *E. Friedrich*, Scientific Collaborator at the Dresden Test House[16]. A very clear arrangement is obtained if the ordinates represent the so-called carrying capacity $T = \frac{m \cdot h}{J_i}$ and the abscissae are given the values $\frac{l}{s'} = \frac{h}{x}$. If the state of cracking of the

[16] See: Contribution to discussion (Diskussionsbeitrag) by E. Friedrich-Dresden.

compression zone of concrete be based on the cube strength $\sigma_p = 0.75\ W_b$ and not on the usual state IIb with triangular stress-distribution, but on a new state IIc with rectangular stress distribution (under consideration of the plastic deformability of concrete), the limiting value can be expressed by the formula

$$s'_g = \frac{x}{h} = \frac{3}{2} - \frac{1}{2} \cdot \sqrt{\frac{3\,(1+3\,k)}{3+k}} \tag{49}$$

The coefficient k is expressed by the term

$$k = \frac{\sigma_s}{n \cdot \sigma_p} \tag{50}$$

and hence the limit of reinforcement is obtained from:

$$\mu_G = s'_g \cdot \frac{\sigma_{b\ adm}}{\sigma_{e\ adm}}. \tag{51}$$

The comparison with results received from beams with rectangular section (Dresden tests 1936) is quite satisfactory.

c) To make use of the carrying capacity in this second range for which the cube strength is the decisive factor for rupture, proposals to replace the exceptional permissible increase of stresses for frames and haunches (full rectangular sections § 29 Table IV and No. 5, b, β and δ) can only be made after completion of the Dresden tests now in progress for slabs and T-beams, reinforced with St 37, St 52 and other high-grade steels.

The result of these considerations is that for the first range of lightly reinforced beams no change in the usual mode of calculation is required. But it will be permissible in future to extend this range to the reinforcing limit μ_G, which still requires to be calculated and proved by tests which have not yet been concluded. It implies that for the second range beyond this limit a new method of calculation will be required for the compressive stresses in concrete if reinforcement on the compression side and haunches are to be avoided.

III. Objections have been raised against the acceptance of the yield limit σ_s as basis of the safety against rupture ($v_B = \sigma_s : \sigma_{e\ adm}$; Eq. 7) for lightly structural elements of reinforced concrete. The objection was that on account of the plastic deformabilitiy of concrete in rupture tests have given higher calculated values than yield limit i. e.

$$\max \sigma_e > \sigma_s,$$

or in other words that the safety margin could still be utilised[17]. These "excess values"

$$\beta = \frac{\max \sigma_e - \sigma_s}{\sigma_s} \tag{42}$$

[17] Compare *W. Gehler:* International Congress for the Testure of Materials Zurich 1931 (Paper: Strength, Elasticity and Shrinkage of Reinforced concrete pp. 1079 to 1087 [Festigkeit, Elastizität und Schwinden von Eisenbeton] where the nature of plasticity of concrete in comparison to building steel was studied very exhaustively). Congress publication, Zurich 1932, published by I.V.M.

are shown in Fig. 15 in relation to the cube strength $W_{b_{90}}$, as they were found by recent Dresden tests carried out with particular care. With increasing quality of concrete an increase in the value β can fundamentally be noticed; this varies

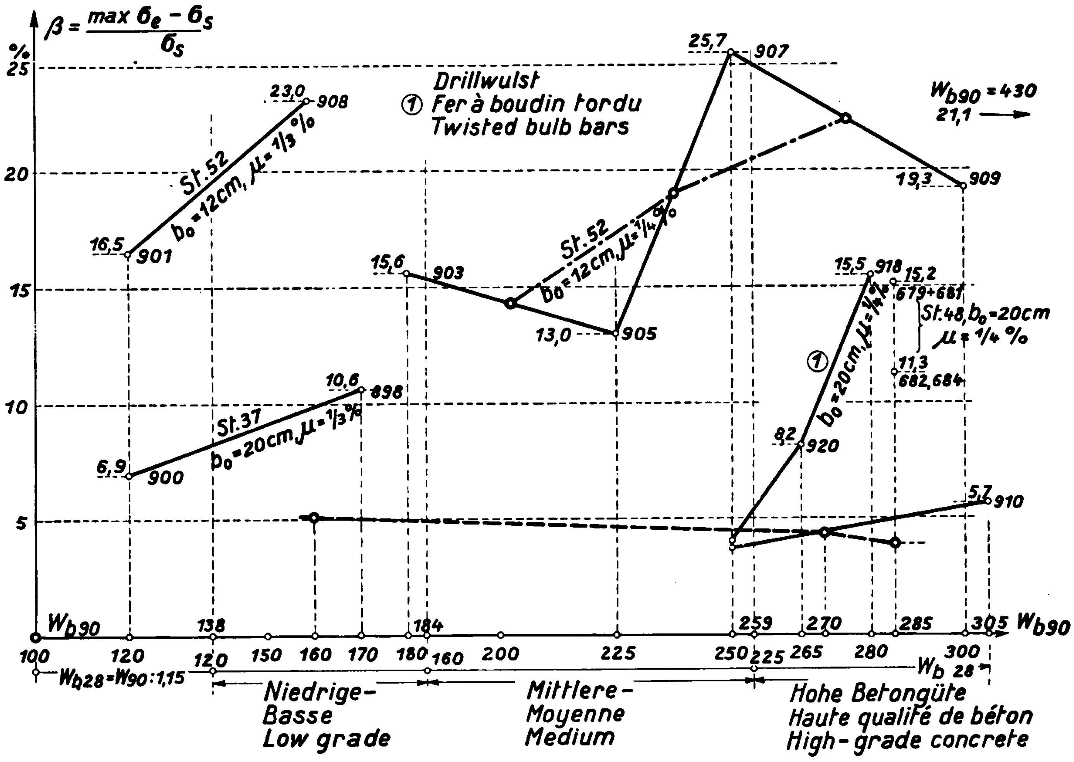

Fig. 15.

The value of the plastic deformability β in relation to the quality of concrete $W_{b_{90}}$ (Dresden tests 1936 for T-beams).

for T-beams between 4 % and 26 % (in the case of Fig. 14 β had an average value of 12.5 %). It was, however, not possible to find a law. *It is therefore advisable not to make use of this varying margin of safety, but to base as usual the safety against rupture on the yield limit for lightly reinforced concrete beams.*

IV. The *degree of safety against rupture* as found by the Dresden tests with T-beams is compiled in table IV.

Table IV

Type of steel	St 37	St 52	Isteg	Twirted bulb steel.
σ_s mean	2610 to 2935 2790 = abt. *2800*	3840 to 4445 3980 = abt. *4000*	4035 to 4425 4110 = abt. *4100*	4000 to 4390 *4200*
$\sigma_{e\,adm}$	*1200*	*1800*	*1800*	*1800*
$\nu_B = \sigma_s : \sigma_{e\,adm}$	*2,33*	*2,22*	*2,28*	*2,33*

The required minimum factor of safety of 2 against rupture is therefore sufficiently maintained for statical stressing for St 37 and 1200 kg/cm² and for high-grade steel reinforcement with 1800 kg/cm².

E. The application of high-grade steel as anti-shrinkage reinforcement in concrete road construction.

Stuttgart tests carried out by Prof. *Mörsch* have clearly proved that by providing reinforcement the degree of shrinkage of concrete is reduced to about half, which indicates that the use of reinforcement to counteract shrinkage in concrete roads is quite justified. The question arises whether for the same cross-sectional area of steel, Steel St. 37, or high-grade steel, e. g. the well-known types of building steel fabric should be given preference on the strength of the Dresden test results concerning safety against rupture.

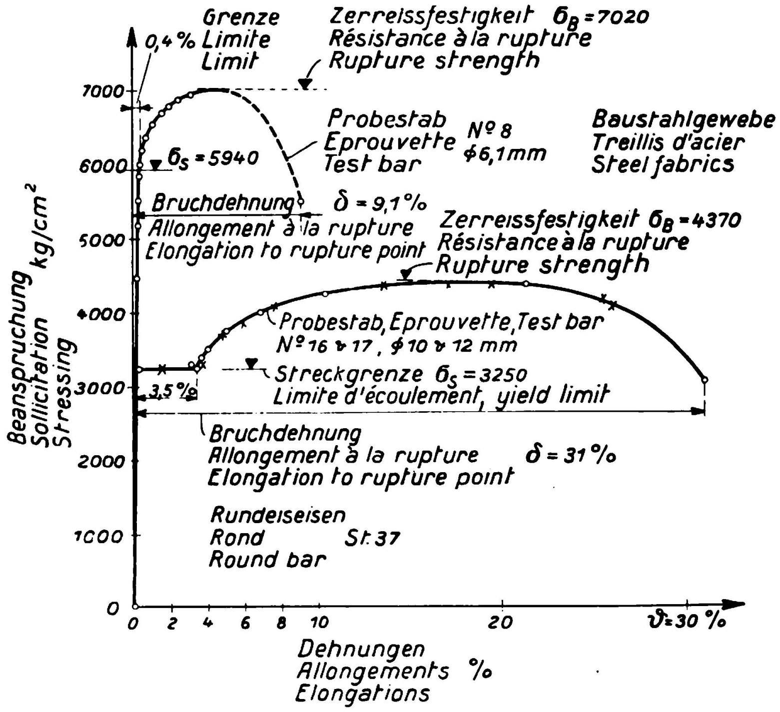

Fig. 16.

Stress-strain diagrams for steel fabrics and round bars of St 37.

1) *Comparison between the stress-strain line of building steel fabric and round bars of St. 37* (Fig. 16) shows that yielding of the building steel fabric cannot be observed whilst this is clearly noticeable with St 37. According to DIN 1602 the 0.2 % limit of permanent elongation has to be accepted as yield limit, which as proved by the 0.4 % limit of the total elongations, therefore we receive for building steel fabric $\sigma_s = 5940$ kg/cm² for $\sigma_B = 7020$ kg/cm² with a rupture point elongation $\delta = 9.1$ %.

2) Concrete roads are also subject to deformations arising from normal traffic loads. Dresden tests of 1934, carried out with concrete strips reinforced with building steel fabric, allow a distinction to be drawn between the three follow-

ing ranges (Fig. 17): *for slabs free from cracks (width of cracks* $b_1 = 0$*); slabs with hair cracks (width of cracks* $b_2 < b_{R\,adm} = \frac{1}{8}$ *mm, (compare Eq. 40) and slabs with fine cracks (*$\frac{1}{8}$ *mm* $< b_R < \frac{1}{4}$ *mm).*

1[st] *range: Slabs without cracks.* Up to an elongation of steel or concrete of $\varepsilon = 0.02\,\%$, which is equal to stress in steel of $\sigma = \varepsilon \cdot E = 420\ \text{kg/cm}^2$, the slab remains without cracking. (Lower limit for cracking based on Dresden

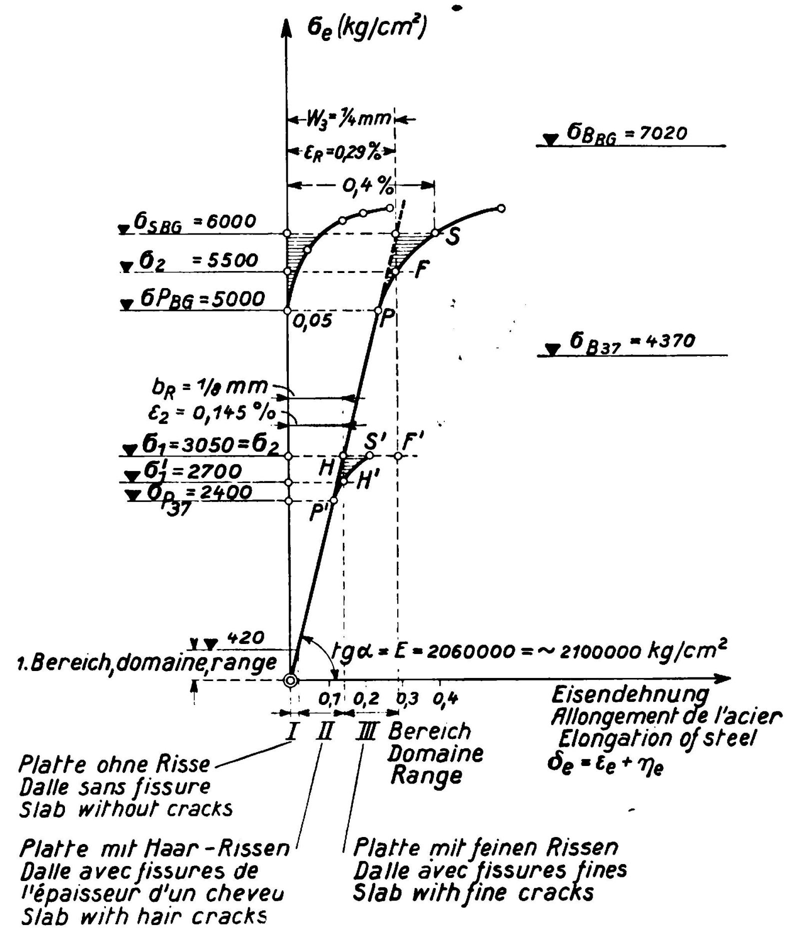

Fig. 17.

Stress-strain diagrams for St 37 of steel fabrics and width of cracks according to Dresden tests.

tests with non-reinforced concrete beams $55 \cdot 15 \cdot 10\ \text{cm}^3$ and slabs reinforced with steel fabric.)

*2*nd *range: Slabs with hair cracks.* Based on formula (40) for the permissible crack width $b_{R\,adm} = \frac{1}{8}$ mm, the sum of all crack widths, over a total fractured length of 950 mm with 11 cracks (of tested beams see Fig. 18) was received as

$$11 \cdot \frac{1}{8}\ \text{mm} = 1{,}375\ \text{mm}$$

and the elongation of steel for the same length:

$$\varepsilon_1 = 1.375 \text{ mm} : 950 \text{ mm} = 0.145\,\%.$$

In Fig. 17 OP′S′ represents the stress-strain line for St 37 and line OPS the corresponding line for building steel fabric (compare also Fig. 16), their inter-

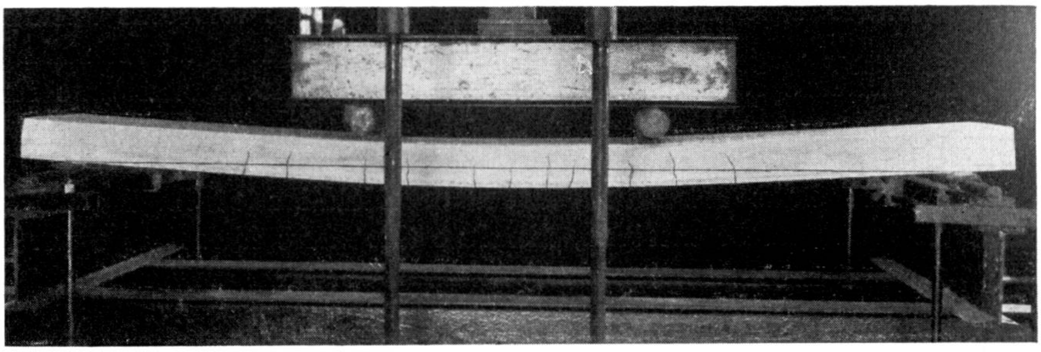

Fig. 18.

Slab reinforced with building steel fabric (Dresden tests 1934).

section points H′ and H with the vertical line at a distance $\delta = \varepsilon_2 = 0.145\,\%$ from the axis of ordinates yields the stress values $\sigma'_2 = 2700$ kg/cm² and $\sigma_2 = 3050$ kg/cm².

The upper limit for the hair crack width $b_R = \frac{1}{8}$ *mm therefore lies 13 % higher on stress scale than for St 37.*

3rd range: Slabs with fine cracks. The upper limit for the width of "fine cracks" may in this case be taken as double $b_{R\,adm} = \frac{1}{8}$ mm, hence $b_3 = \frac{1}{4}$ mm. For this range the protection against rust according to experience still exists although not fully. The upper limit $b_3 = \frac{1}{4}$ mm is attained for an elongation of steel of $\varepsilon_3 = 2\,\varepsilon_2 = 2 \cdot 0.145\,\% = 0.29\,\%$. With this value as abscissa we receive in Fig. 17 the points F′ and F of the two diagrams for which the coordinates are: $\sigma'_3 = \sigma_{s\,37} = 3070$ kg/cm² and $\sigma_3 = 5500$ kg/cm².

The upper limit ($b_3 = \frac{1}{4}$ mm) for fine cracks is reached for St. 37 already at the yield limit $\sigma_s = 3000$ kg/cm², but lies for building steel fabric at a stressing of $\sigma_e = 5500$ kg/cm², a value 80 % higher. *The safety against rust, depending on the crack-width* b_R, *is higher for building steel fabric than it is for St 37* (assuming same sectional area of steel).

F. Oscillation-stressing of structural elements of concrete reinforced with high-grade steel.

For the purpose of drawing conclusions in respect to the degree of safety from available bending test results of structural concrete elements reinforced with building steels with high yield limit, the following procedure is recommended:

1) The so-called "working safety" is expressed by the term

$$v = \frac{w_v}{w_R} \tag{43}$$

Herein w_v represents the maximum amplitude, as found by fatigue tests, a value which can just be produced infinitely often[18] while w_R represents the highest conceivable amplitude which is taken as the basis for static calculation. Il σ_o stands for the upper and σ_u for the lower stress limit of an oscillation test (in the DIN standard N° 4001 called upper stress and lover stress) we receive

$$w_v = \sigma_o - \sigma_u \tag{44}$$

According to the static calculation, under consideration of impact, the permissible stress shall not be exceeded, therefore this stress forms the upper limit of stressing, whilst the stressing due to permanent load only represents the lower limit in the static calculation, hence

$$w_R = \sigma_{adm} - \sigma_g. \tag{45}$$

2) *To determine the amplitude for the static calculation* a certain particularly unfavourable case is used which can be expressed by the two following assumptions:

a) $\sigma_p : \sigma_g = 2 : 1.$

b) As impact coefficient, according to DIN 1075 the maximum possible value $\varphi = 1.4$ is chosen. For this extreme case we receive

$$\sigma_{adm} = \sigma_g + \varphi \cdot \sigma_p = \sigma_g + 1.4\ (2 \cdot \sigma_g) = 3.8\ \sigma_g \tag{46}$$

and

$$w_R = \sigma_{adm} - \sigma_g = \sigma_{adm}\left(1 - \frac{1}{3,8}\right) = 0,737\ \sigma_{adm}. \tag{47}$$

With the expressions (44) and (47) the "working safety" (Eq. 43) assumes the following form

$$v = \frac{\sigma_o - \sigma_u}{\sigma_{adm} - \sigma_g} = \frac{\sigma_o - \sigma_u}{0,737\ \sigma_{adm}}. \tag{48}$$

3) The proposed degree of "working safety" is $v = 2$. This means that for fatigue rupture the amplitude is double the amplitude on which the static calculation is based. From the comparison with the rules (γ — procedure) laid down in "the Basis for Calculating Railway Bridges in Steel (B.E.) of the German State Railways 1934" we learn that the above determination of the degree of safety offers a higher degree of safety than demanded for steel railway bridges (especially if the unfavourable influences of riveted and welded connections are considered).

4) The Stuttgart fatigue tests carried out with Isteg steel on this procedure are compiled in table V which gives the basic values for slabs and the values

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[18] According to the numerous fatigue tests with building steels it can be assumed for the range under consideration that the amplitude is practically independent of the static pre-stressing (mean-stressing).

for w_v, w_R and v which were calculated with equations (44), (47) and (48) (Fig. 19).

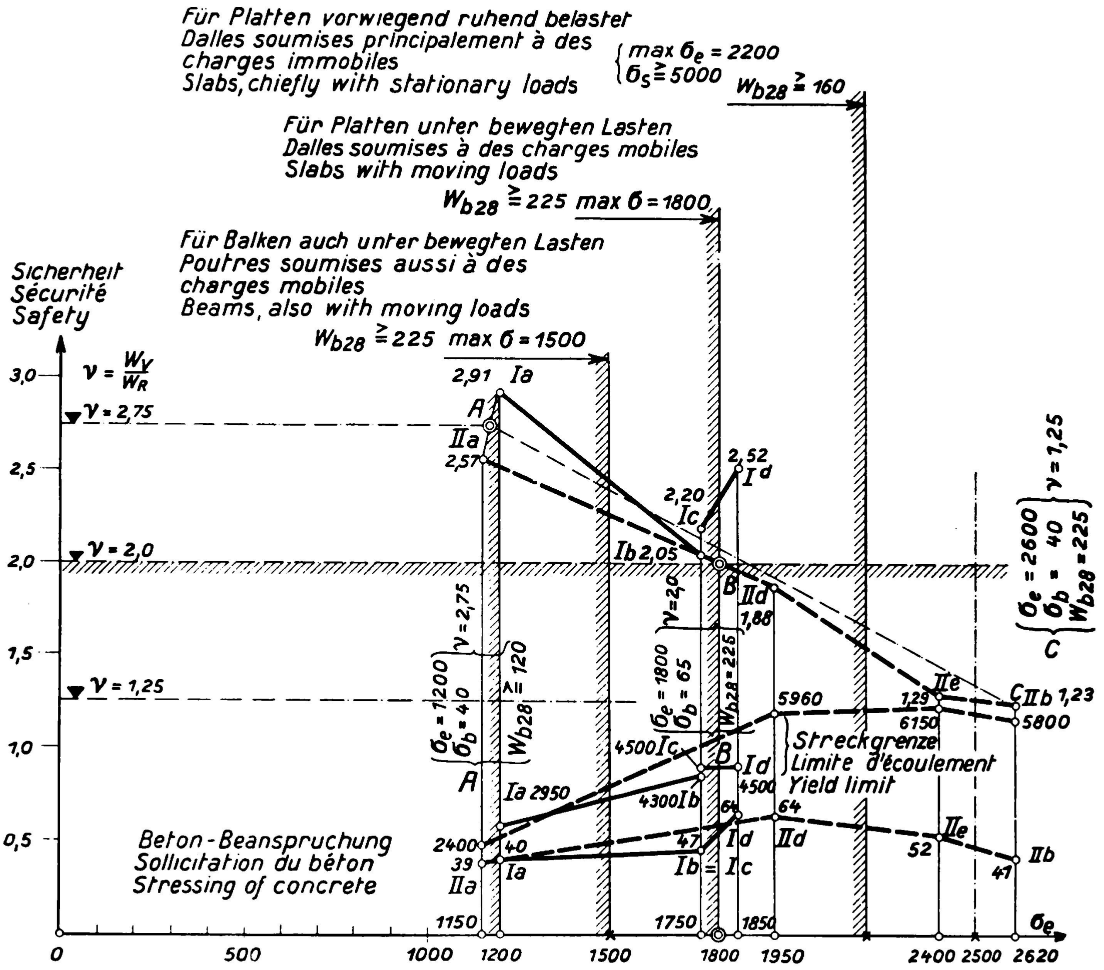

Fig. 19.

Results of Stuttgart fatigue tests on slabs with Isteg — and steel fabric reinforcement (I and II respectively).

Resutls: The high-grade steel St. 60 in combination with concrete of $W_{b\,28}$ = 120 kg/cm² yields a degree of safety v = 2.05 compared with v = 2.9 in case steel St 37 is used. The degree of safety obtained for two slabs reinforced with Isteg-steel giving v = 2.5 and 2.2 lies between the above mentioned values. By using high-quality concrete with $W_{b\,28}$ = 225 kg/cm² an increase in safety from 2.2 to 2.5 is noticed over concrete of ordinary quality.

5) The results of the Stuttgart fatigue tests of 1934 for slabs reinforced with Isteg steel were examined in the same way (series a′ to d′).

Results: A slab with St. 37 of ordinary concrete showed a somewhat smaller degree of safety namely v = 2.57. The two slabs reinforced with building steel fabric of the series b′ and e′, which were designed for the extraordinary high stressing of steel of σ_e = 2620 kg/cm² and 2400 kg/cm² respectively, show too small a degree of safety, namely v = 1.23 and 1.29, which is considerably below v = 2. But the slab of series d′ reinforced with building steel fabric and designed for σ_e = 1950 kg/cm² with high-grade concrete

$W_{b_{28}} = 225$ kg/cm² gave a degree of safety of $\nu = 1.88$. Assuming a more or less straight-line proportionality, the degree of safety for $\sigma_e = 1800$ kg/cm² and $W_{b_{28}} = 225$ kg/cm² works out at

$$\nu = 1.88 \cdot \frac{1950}{1800} = 2.03.$$

6) The examination of the Stuttgart fatigue test results have proved that the permissible stress of steel $\sigma_e = 1800$ kg/cm² in slabs is properly chosen and suitable also in the case of moving loads; provided, however, that the concrete possesses at least a cube strength of $W_{b_{28}} = 225$ kg/cm² .With this latter stipulation concerning the quality of concrete, according to experience gathered, sufficient safety against cracking can be assumed. Since corresponding tests for beams subjected to moving loads have not yet been, carried out we propose that the permissible stress for beams shall be left at $\sigma_e = 1500$ kg/cm².

Table V.

Type of reinforcement and nature of concrete	Test series	Thickness of slab a (cm)	Basis of dimensioning $\frac{\sigma_b}{\sigma_e}$	quality of concrete $W_{b\,28}$		quality of steel		Plasticity factor $\beta' = \frac{\max \sigma_e}{\sigma_s}$	w_V	w_R	Working safety $\nu = \tan = \frac{w_V}{w_R}$
				ideal	actual	σ_s	max σ_e				
Isteg + high-grade concrete	d	11,5	64/1850	*200*	*260*	4500	5800	1,29	3440	1365	2,52
Isteg + concrete	c	14,2	47/1750	120	118	4500	5600	1,24	2840	1290	2,20
St 60 + concrete	b	13,7	47/1750	120	123	4300	4920	1,14	2640	1290	2,05
St 37 + concrete	a	14,1	40/1200	120	123	2950	3440	1,16	2570	884	2,91
Building steel fabric + high-grade concrete	d'	13,8	64/*1950*	*210*	*239*	5900	7120	1,21	2700	1435	1,88 [19]
do. + concrete	b'	17,4	41/*2620*	160	219	5800	8160	1,41	2360	1920	1,23
do. + concrete	e'	10,8	52/*2400*	180	195	6150	7740	1,26	2280	1770	1,29
St 37 + concrete	a'	14,0	39/1150	130	115	2400	3200	1,33	2180	847	2,57

[19] $1{,}88 \cdot \frac{1950}{1800} = \mathit{2{,}03}$.

G. The permissible stresses of reinforcements with high yield limit for slabs and T-beams of reinforced concrete.

1) The permissible stresses given in table VI were laid down on January 14th 1935 by the German Commission for Reinforced Concrete and subsequently embodied in governmental regulations. Some knowledge gained since from tests and some further explanations on the foregoing are added herewith:

a) Based on the Stuttgart tests with slabs of concrete of $W_{b_{28}} \geqq 225$ kg/cm² and reinforced with St 52 or corresponding high-grade quality steels, *it is permitted for slabs (2nd line, 6th row) to increase* $\sigma_{e\,adm}$ *= 1500 kg/cm² to 1800 kg/cm²; this also in case of moving loads.*

Table VI.

Table of permissible stresses for reinforcement with high yield limit for slabs and T-beams in reinforced concrete.

1	2	3	4	5	6	7	8
No.	Type of steel	Min. yield limit[20]	Min. rupture point elongation	Min. cube strength of concrete	$\sigma_{e\,zul}$ for slabs	$\sigma_{e\,zul}$ for T-beams	Range of validity
	—	kg/cm²	%	kg/cm²	kg/cm²	kg/cm²	—
1	St 52	3600	20	120 225	1500 *1500*	1200 *1500*	Also for moving loads[22]
2	St 52	3600	20	120 160 225	1500 1800 1800	1200 1200 1500[23] 1800[24]	For chiefly stationary loads and only for Building constructions without atmospheric influences.
3	Special steel[21]	3600	14[25]	120 160 225	1200 1800 1800	1200 1200 1500[23] 1800[24]	
4	Special steel[21]	5000	14[26]	120 160 225	1200 2200 2200	1200 1200 1500[23] 1800[24]	

[20] *Yield limit,* According to the R. C. Regulations § 7 the properties of steel require to be proved. For steels having no distinct yield limit, till such time as the question receives final settlement by tests which are being carried out, it is permissible instead of the 0.2 % limit of the permanent elongation as stipulated by DIN 1602 to take as yield limit the 0.4 % limit of the total elongation.

[21] *Reinforcement of special steels* for special arrangements requires permission.

[22] Corresponds to existing regulations.

[23] If the cross section of one single bar is > 3.14 cm² (for twisted steels the compound section is decisive).

[24] If the cross section of one single bar is ≤ 3.14 cm² (otherwise as in 23).

[25] For slabs steels with a minimum rupture point elongation of 10 % are also permissible.

[26] For slabs steels with a minimum rupture point elongation of 8 % are also permissible.

b) The restrictive stipulations given in Footnotes 4 and 5 with the fixed limit-value $F_e = 3.14$ cm^2 should be based on our Eq. 28 and 40 — replaced by a more conclusively founded regulation; this however is only possible after the necessary tests have been carried out.

2) The critical examination of the safety against rupture of reinforced concrete beams (see under D II and Figs. 13 and 14) lead to the following results.

a) The usual mode of calculation need not be altered for structures coming under the first range of lightly reinforced beams for which the yield limit of the steel is decisive (normal case).

b) Should the reinforcing limit μ_G reach the point separating the two ranges as fixed by the Dresden tests for St. 37 and high-grade steel, it is permissible to extend the first range up to this limiting value and the usual method of calculation can again be employed.

c) Beyond this limit, i. e. in the second range for which the compression strength of concrete is decisive (rare case) a new method making increased use of the material properties would require to be laid down. The employment of compression reinforcement and haunches could then mostly be avoided and the appearance of structures would be improved.

Our tests with high-grade steels in reinforced concrete construction led to the realisation, that by maintaining the safety against cracking and rupture a considerable increase of permissible stresses, and a better exploitation of the properties of material, is justified (see table VI). Further it led to a critical examination of safety and finally to the incitement to supplement the usual mode of calculation and with it to improve designing. By these investigations based on test research an old gap in our knowledge of the nature of reinforced concrete has been filled.

Summary.

The question of employing high-quality steel in reinforced concrete was brought to a certain conclusion through the new Dresden tests carried out for the German Commission on Reinforced Concrete. The width of cracks were photographed with a 23-times enlargement, and the depth of cracks was measured too.

Safety against cracks, i. e. the ratio of loading at the moment of cracking to the working load has the value 1.8 for crosswise reinforced slabs supported on all sides, and for slabs supported on all 4 corners (Advance tests for mushroom slabs) the value 1.4 for slabs reinforced in one direction only 0.75 and for T-beams 0.5. The employment of high grade steel as reinforcement is therefore most suited for slabs. The safety against cracks increases also with increasing *quality of concrete,* but only to a very small degree on account of the increased brittleness of high quality cement. The smaller the percentage of reinforcement compared with the cross section of concrete and the width of the webs, the greater the safety against cracks and therefore the smaller the *depth of cracks.* Since on the other hand the width of cracks increases with the increased width

of web, a limit is given to the increase of web-width by stipulating a permissible width of cracks for working loads (e. g. $^1/_8$ mm).

There merely statistic facts lead to the *physical conception* that with the appearance of a crack the cracked portion of the cross section (depth of crack t x width of web b_o) is put out of action and with it also the tensile force belonging to this area. The magnitude of this force can be 4.8 or 12 % of the tensile force in steel at the moment of cracking for low, medium or high grade concrete respectively. Increased degrees of safety against cracks can be expected for I and box-type cross section.

T-beams reinforced with St 52 and subjected chiefly to stationary loads for a permissible stress of steel of 1800 kg/cm² offer the same safety against cracks as if designed for St 37 with a permissible stress of 1200 kg/cm².

Based on the determination of cracking-point loads, on the 14th Jan. 1935 a table showing the permissible stresses by using steel of high yield point, was arranged by the German Commission on reinforced concrete (Table VI), and further it gave rise to the following results.

For the first range of lightly reinforced beams, for which the yield point of steel is the decisive factor (normal case) nothing requires to be changed in the usual mode of calculation. As soon as the dividing limit which separates the two ranges are brought to conclusion by the Dresden test which are being carried out at present, then the first range can be extended up to this limit and with it the validity of the usual mode of calculation. For the second range for which the compressive strength of concrete is decisive (rare case) a new procedure can be introduced making extended use of the properties of materials with the purpose to eliminate in designing the application of haunches and Compression reinforcement. This would allow to improve the appearance of concrete constructions.

For moving loads the Dresden tests revealed that for slabs reinforced with St 52 a permissible stress of 1300 kg/cm² is applicable if the concrete has least a cube strength of 225 kg/cm². For T-beams the conditions remain the same as before, the permissible stress to be 1500 kg/cm².

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IIc 3

High-Grade Steel in Reinforced Concrete.

Hochwertige Stähle im Eisenbetonbau.

Aciers à haute résistance dans le béton armé.

Dr. Ing. R. Saliger,
Professor an der Technischen Hochschule Wien.

A. Columns with high-grade steel reinforcement.

The numerous experiments that have been made with columns, reinforced by longitudinal bars and stirrups[1] have shown that it is rarely possible to use longitudinal reinforcement to its full upsetting limit. The limit of upsetting is reached only where concrete columns are laterally reinforced. The reason for this lies in the fact that upsetting to destruction in concrete does not reach the degree of upsetting which exists in the upsetting stress of the reinforcement bars. The result is that the destruction of the concrete takes place earlier and that the longitudinal bars buckle. In such columns the stress in the longitudinal reinforcement is expressed by the ratio of $E_e : E_b$, and it makes no great difference whether the longitudinal reinforcement is of mild steel or high grade steel. *The use of high-grade steel is therefore justifiable in general only for laterally bound concrete columns* which can withstand a higher degree of upsetting.

a) Experiments with laterally bound reinforced concrete columns with high-grade steel insertion.

The investigation included, apart from research work carried out on concrete encased cast iron columns, a series of experiments which were carried out between 1929 and 1933 and which have been reported on elsewhere[2].

The first set of experiments refers to five different designs of column construction carried out in equal sets of two, in all ten columns, which had the

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[1] *Bach:* Mitteilungen über Forschungsarbeiten des V.D.I. (Communications concerning experimental work carried out by the Institution of German Engineers), Issue 29 and 166. *Saliger:* Zeitschrift für Betonbau 1915, Issue 2 to 4. *Commission du Ciment armé,* Paris 1907. *Emperger:* Versuche an Säulen aus Eisenbeton (Tests on reinforced concrete columns) 1908. *Spitzer:* Issue 3 of the Österr. Eisenbetonausschuß (Austrian Commission on Reinforced Concrete). *Mörsch:* Der Eisenbetonbau (German Commission on Reinforced Concrete) Issue 5, 10, 14, 21, 28, 34. *Probst:* Vorlesungen über Eisenbeton (Lectures on reinforced concrete) Vol. 1, Berlin 1917.

[2] Beton und Eisen 1930, Issue 1, 17. Bauingenieur 1931, Issue 15, 16. Österr. Eisenbetonausschuß (Austrian Commission on Reinforced Concrete) Issue 13. Bericht auf dem Internat. Kongreß für Beton und Eisenbeton in Lüttich 1930 u. a. (Report submitted to the International Congress on Concrete and Reinforced Concrete held at Liege, 1930 etc.)

following dimensions: 1.2 m length; 16-angled section; diameter-34 cm. The concrete core section was 700 to 740 cm². Prism resistance of the concrete: 227 kg/cm²; longitudinal reinforcement was of high grade round bar steel having an average yield point of 7.35 tons/cm²; percentage of reinforcement: 4.3 to 8.8 %. Lateral reinforcement for 8 columns with steel having a yield point of 5.2 tons/cm², percentage of lateral reinforcement 0.5 to 2.0 %. Two columns had lateral bindings of iron with $\sigma_s = 2.6$ tons/cm² and a percentage of lateral reinforcement of 2.1 %.

The second lot of experiments included six various types of column structure divided into three sets of two equal columns all columns being of 1.2 m long and of octagonal cross section with a diameter of 35 cm; cross section of concrete core: 760 cm²; prism resistance: 204 kg/cm²; longitudinal reinforcement: 3.8 to 11.2 %; yield point: 7.7 tons/cm². In six of the columns the longitudinal reinforcement had been butt-welded in the centre. The lateral binding consisted of hoop iron with yield point of 2.9 tons/cm²; the percentage of lateral reinforcement for all these columns was 1.1 %.

The third set of experiments was carried out with sixteen columns having a length of 3 m. The prism strength of the concrete was 116 kg/cm². The longitudinal reinforcement for fourteen of these columns consisted of round steel bars having a yield point of 4.25 tons/cm²; in the case of two of the columns this limit was 2.77 tons/cm². The longitudinal reinforcement percentage was 3.8 to 14.8. The lateral binding for six columns was round bar steel with a yield point of 5.2 tons/cm²; for ten of the columns it was steel $\sigma_s = 2.5$ tons/cm². The lateral reinforcement percentage lay between 0.5 and 2.2 %.

Further tests were carried out with ten columns each 3 m long reinforced with rolled section having a yield point of 2.67 tons/cm², and a reinforcement percentage of 3.7 to 11.9 % (concrete encased steel columns). The concrete core sections for eight of the columns were 680, or 952 cm²; for two columns of rectangular shape the core cross-section was 490 cm². The strength of concrete was 146 kg/cm². The lateral binding had a yield point of 2.5 to 2.9 tons/cm² and the reinforcement percentage was 0.6 to 1.3 %.

The fifth set of tests was carried out with six columns having a length of 1.5 m and fourteen which were 3 m long each; the shaft was 34 cm thick and had a core area of about 760 cm². The prism strength of the concrete was 211 kg/cm². Longitudinal reinforcement of round bar steel had a yield point of $\sigma_s = 2.4$ for four of the columns, for the remaining sixteen it was $\sigma_s = 6.16$ to 6.92 tons/cm² with reinforcement percentages of 4.6 to 11.0 %. Lateral reinforcement consisted of steel of $\sigma_s = 2.0$ to 2.3 tons/cm² representing a percentage of reinforcement of 0.5 to 2.1 %.

b) Conclusions drawn from the tests and from theoretical considerations.

1) *Extent of experiments.*

The longitudinal reinforcement in the above experiments consisted of steel having a yield point of 2.2 to 7.7 tons/cm² and reinforcement percentages of 4 to 14 %. For lateral binding steel of 2.0 to 5.2 tons/cm² and percentages of

0.5 to approx. 2.0 % was used. The experiments thus covered the whole extent of reinforcement percentages which are actually used in practice.

2) *Full utilisation of longitudinal reinforcement.*

The rupture point upsetting strength of the laterally reinforced concrete columns generally exceeds the compressibility of the non-reinforced concrete very considerably. If the lateral reinforcement is adequate this compressibility will be found to be so high that it reaches the upsetting limit of the longitudinal reinforcements or the yield point fixed by definition may even be exceeded. No noticeable difference between the upsetting limit during compression tests and the yield limit of the steel during the tensile test was observed. In the case of laterally reinforced columns it is possible to utilise to the full the longitudinal reinforcements of high-grade steel. The behaviour of the concrete encased rigid steel columns is analogous to that of flexible longitudinal reinforcement. Buckling of individual bars or of the steel columns encased in concrete can be disregarded where the column is properly designed and executed. In the case of very slender columns there is some risk of buckling of the whole columns.

3) *Requisite thickness of the spiral reinforcement.*

It was possible to utilise to the full the upsetting limit of the longitudinal reinforcement in all those columns in which the amount of lateral binding was:

$$F_u \cdot \sigma_{u\ streck} \geqq 0.05\ F_e \cdot \sigma_{stauch}$$

and if

$$F_u \cdot \sigma_{u\ streck} \geqq 0.1\ F_k \cdot \sigma_p$$

or

$$\mu_u \geqq 0{,}05\ \mu \cdot \frac{\sigma_{stauch}}{\sigma_{u\ streck}} \quad \text{and} \quad \mu_u = 0{,}1\ \frac{\sigma_p}{\sigma_{u\ streck}}.$$ [3]

in the above formula σ_p represents the prism strength of the concrete. F_u = lateral binding, F_e = longitudinal reinforcement, streck = yield limit, stauch = upsetting.

Where the amount of lateral binding decreases (expressed by $F_u \sigma_{u\ streck}$) below a certain figure, the upsetting limit of the longitudinal bars cannot be reached with certainty. If however the amount of lateral binding is considerably higher, then the column concrete is capable of resisting higher upsetting values and in that case the longitudinal bars are subjected to pressure which may exceed the upsetting limit as laid down by definition. Where conditions are otherwise similar, the lateral reinforcement with a high yield limit is more effective than where softer steel is used.

4) *Effects of lateral reinforcement.*

Lateral reinforcement has two objects: Increase of strength in concrete by means of circular hooping is $N_u = a \cdot F_u \sigma_{u\ streck}$. If the concrete had no inherent strength and behaved like a liquid, then $N_u = \frac{1}{2} F_u \cdot \sigma_{u\ streck}$, that is: $a = \frac{1}{2}$. Ex-

[3] The conception according to which a considerably stronger reinforcement, for instance, of 2 to 3%, is necessary in order to ensure full utilisation of prismic strength and of the upsetting limit of longitudinal reinforcement, is not covered by these experiments. See: ***Freudenthal:*** Verbundstützen für hohe Lasten (Composite columns for high loading), Berlin, 1933.

periment and theory have shown that for concrete, when m is the Poisson coefficient, $a = \frac{m}{2} = 1.5$ to 4, in which case the lower value refers to high compressive stresses in high-grade concrete and the higher value to lower compressive stresses in low-grade concrete. As the quality of the concrete rises, the coefficient a expressing the effect of reinforcement decreases As an average a = 2 to 3 is attainable. The effect of lateral reinforcement increases with the yield limit $\sigma_{u\,streck}$ of the lateral reinforcement. The first mentioned object of lateral binding is thus based on an increase of compressive strength of the concrete by

$$\Delta\,\sigma_p = \frac{N_u}{F_k} = 2{,}5\;\mu_u\,\sigma_{u\;streck}$$

as an average. The second object of lateral reinforcement is to ensure sufficiently high deformations of the concrete, attainment of the upsetting limit of the longitudinal reinforcement, joint action of the two materials generally and finally, prevention of buckling of the longitudinal bars. Where compression is carried to the limit of inherent strength of the concrete, the stressing of the lateral binding is low, when the upsetting compression rises, the stresses in the lateral reinforcement rise rapidly up to the yield limit and even leads to fracture.

5) *Formation of cracks.*

Up to the point of the formation of cracks the total sectional area of the concrete (core and cover) and of the longitudinal bars act in the same way as ordinarly longitudinally reinforced concrete columns, according to the ratio of $E_e : E_b$ of specific elongation, without any marked influence due to the lateral reinforcement. The longitudinal cracks appear with such compression in concrete which is approximately equivalent to the prismic strength. The transverse elongation ε_q of the concrete and thus of the concrete cover over the bars amounts to about

$$\varepsilon_q = \frac{\varepsilon}{m}.$$

If the capability of expansion of the concrete be taken as $\varepsilon_q = (1.5 \text{ to } 2) \cdot 10^{-4}$ and m = 7, the result will be $\varepsilon = 7\,(1.5 \text{ to } 2) \cdot 10^{-4} \approx (1 \text{ to } 1.5) \cdot 10^{-3}$, that is, the cover may be expected to crack with a shortening of the column by about 1 to 1.5 mm per metre. For less good quality of concrete ε_q will be smaller and m will be larger and *vice-versa* for high-grade concrete, so that the upsetting of the column mentioned can be taken as an average value. A shortening of 1 to 1.5 mm per metre corresponds to a longitudinal stress in the concrete of the column of about 100 to 250 kg/cm², that is the prism strength of the concrete. After exceeding the prism strength of the concrete, the cover begins to peel off. The cracking load can be expressed thus:

$$N_{Riss} = (F_b - n \cdot F_e)\,\sigma_p \qquad (\text{Riss} = \text{crack})$$

And thus the safety against crack formation becomes:

$$s_R = \frac{N_{Riß}}{N_{zul}}. \qquad \begin{array}{l}(\text{Riß} = \text{crack})\\ (\text{zul} = \text{permisible})\end{array}$$

When the load is increased beyond N_{Riss} the concrete cover over the bars breaks off.

6) *Limit-case.*

Encased columns, whose carrying capacity in the form of ordinarly longitudinally reinforced columns (including the cover of concrete surrounding the core) is greater than that of laterally reinforced concrete (without considering the concrete cover over the bars) fail when cracks begin to form and the cover falls off the column.,The cracking load in such cases is the maximum load. As the strength of the lateral reinforcement increases and the participation of the concrete cover in the total concrete area decreases, so the breaking load exceeds the cracking load.

7) *Breaking load and permissible working load.*

In every case in which the conditions of Point 3 are complied with, the carrying capacity of the columns with high-grade steel reinforcement will be the result of the sum of resistances set up by the prism strength of the concrete core, the strength of the longitudinal reinforcement (without buckling reduction) and of the tensile strength (of the yield limit) of the reinforcement.

$$N_{Bruch} = F_k \sigma_p + F_e \sigma_{e\,stauch} + 2.5\, F_u \cdot \sigma_{u\,streck} \tag{1}$$

with s times safety

$$N_{zul} = \frac{N_{Bruch}}{s}. \quad \text{(Bruch = failure)}$$

When the load is practically static the factor of safey $s \cong 2.5$ suffices. Experience has shown that where the workmanship is good a minimum value of concrete strength in the structure can be expected to have an average value of

$$\sigma_{p\,min} = \frac{2}{3} \sigma_{p\,mittel}. \quad \text{(mittel = average)}$$

Hence we receive:

$$N_{zul} = \frac{F_k\, \sigma_{p\,mittel}}{3{,}5} + \frac{F_e\, \sigma_{e\,stauch}}{2{,}5} + F_u\, \sigma_{u\,streck} \tag{2}$$

This relation can be applied *without determination of permissible stresses* in order to calculate the permissible working load or after to a corresponding transformation to determine the dimensions on the basis of quality of the material and the safety factor. If it is wanted to calculate in the usual manner by using permissible stresses then the following formula has to be employed

$$N_{zul} = F_k \cdot \sigma_{b\,zul} + F_e \cdot \sigma_{e\,zul} + 2{,}5\, F_u \cdot \sigma_{u\,zul} \qquad (2a$$

By substituting:

$$\frac{\sigma_{e\,zul}}{\sigma_{b\,zul}} = n \quad \text{and} \quad \frac{\sigma_{u\,zul}}{\sigma_{b\,zul}} = n_u,$$

we receive

$$\left.\begin{aligned} N_{zul} &= (F_k + n\, F_e + 2{,}5\, n_u\, F_u)\, \sigma_{b\,zul} \\ &= (1 + n\, \mu + 2{,}5\, n_u\, \mu_u)\, F_k\, \sigma_{b\,zul} \end{aligned}\right\} \tag{2b}$$

8) *Participation of the concrete and the steel reinforcement in the strength.*

The participition of the steel in the carrying capacity of the structure is greater, within the zone covered by tests, in proportion as the longitudinal reinforcement and the lateral binding are stronger and in proportion as the steel is of better quality. For instance,

$$\sigma_{e\,stauch} = 6000,\ \sigma_{u\,streck} = 4000 \text{ and } \sigma_p = 200 \text{ kg/cm}^2,$$

$$n = 30 \text{ and } n_u = 20 \text{ and assuming } \mu_u = \frac{\mu}{6}$$

the values given in the numerical table will result. Here σ_b represents the increased compressive strength of the concrete resulting from the lateral rein-

$\mu = \frac{F_e}{F_k} =$	0,03	0,06	0,12
$\frac{N_b}{F_k\ \sigma_p} =$	1	1	1
$\frac{N_e}{F_k\ \sigma_p} =$	0,9	1,8	3,6
$\frac{N_u}{F_k\ \sigma_p} =$	0,25	0,5	1,0
$\frac{N_{Bruch}}{F_k\ \sigma_p} =$	2,15	3,3	5,6
$\frac{\sigma_b}{\sigma_p} =$	1,25	1,5	2,0
Participation of the concrete	47	30	18 %
„ „ „ longitudinal bars }	42 } 53	55 } 70	64 } 82 %
„ „ „ lateral binding }	11 }	15 }	18 }
$\frac{N_{Riss}}{F_k\ \sigma_p} =$	2,03	2,65	3,90
$\frac{N_{Riss}}{N_{Bruch}} =$	0,95	0,80	0,70
$s_R =$	2,4	2,0	1,7

forcement and as compared to the prism strength. The cracking load is calculated at $F_b = 1.4 F_k$, the safety against cracking s_R with $s = 2.5$ times the safety against failure.

We can deduce from the Table that on the assumptions mentioned the average breaking stress (reduced to the area of the core) rises with a 12 per cent. longitudinal reinforcement and a 2 % lateral reinforcement to $\sigma = 5.6\,\sigma_p$. With a prism strength of $\sigma_p = 200$ kg an average breaking stress of $\sigma = 5.6 \cdot 200 = 1120$ kg/cm², which is more than the carrying capacity of a mild steel column of the same circumscribed sectional area and of having an average slenderness ratio. The participation of concrete resistance decreases as reinforcement increases, while the participation of the compressive resistance taken on by the steel reinforcement increases in the Table up to 82 %. Such columns behave almost like steel columns, although concrete is absolutely indispensable.

c) Use of high-tensile steel for compression members and columns.

The use of high-tensile steel for columns, arches and other compression members offers new possibilities in dimensioning of sections. With reference to the requisite external dimensions, competition with steel is all the more easy, if the reinforcement is of higher quality. It is pratically always necessary to use a good quality of concrete even if its participation in load-bearing is comparatively low.

Fig. 1 shows to scale the dimensions of the sections which, under various assumptions, are required for a load of 1000 tons.

1) An ordinary longitudinally reinforced column of structural concrete with $\sigma_{b\,zul} = 35$ kg/cm² and with a percentage of reinforcement of St 37 of $\mu = 0.8$ %.
2) The same conditions but with high-grade concrete of $\sigma_{b\,zul} = 70$ kg/cm².
3) A laterally bound reinforced concrete column of high-grade concrete with $\sigma_{b\,zul} = 70$ kg/cm² and reinforcement of steel St 37.
4) A concrete encased steel column made of rolled section of St 37 with $\sigma_{e\,zul} = 1400$ and $\sigma_{b\,zul} = 60$ kg/cm².
5) A laterally bound reinforced concrete column of high-grade concrete ($\sigma_{b\,zul} = 60$ kg/cm²) and of steel with: $\sigma_s = 6000$ ($\sigma_{e\,zul} = 2400$ kg/cm²) with a safety factor of 2.5.
6) A column made of steel only (St 37) with $\sigma_{e\,zul} = 1400$ kg/cm². (The dotted line shows the outline of a possible encasing).

The *structural design* of compression members with high-grade steel reinforcement call for special measures. A transmission of the longitudinal forces by adhesion (grip) alone, as exists in ordinary reinforced concrete construction, will not be possible. The longitudinal bars are best jointed by butt-welding. In order to induce greater individual forces in the columns, special design would seem indispensable. The uniting of the longitudinal bars in order to obtain a rigid skeleton is generally effected by welding on hooping straps. *Dr. Bauer* has made a number of suggestions concerning the connections for joining the girders to the columns and for assemblage in general. The reinforcing skeleton consisting of the longitudinal bars and the lateral reinforcement has to be tied together in the workshop and then erected on the building site as though it were a steel column.

Quite often it will be advantageous to combine the light steel framework of columns and girders of welded, riveted or bolted, execution with the ordinary

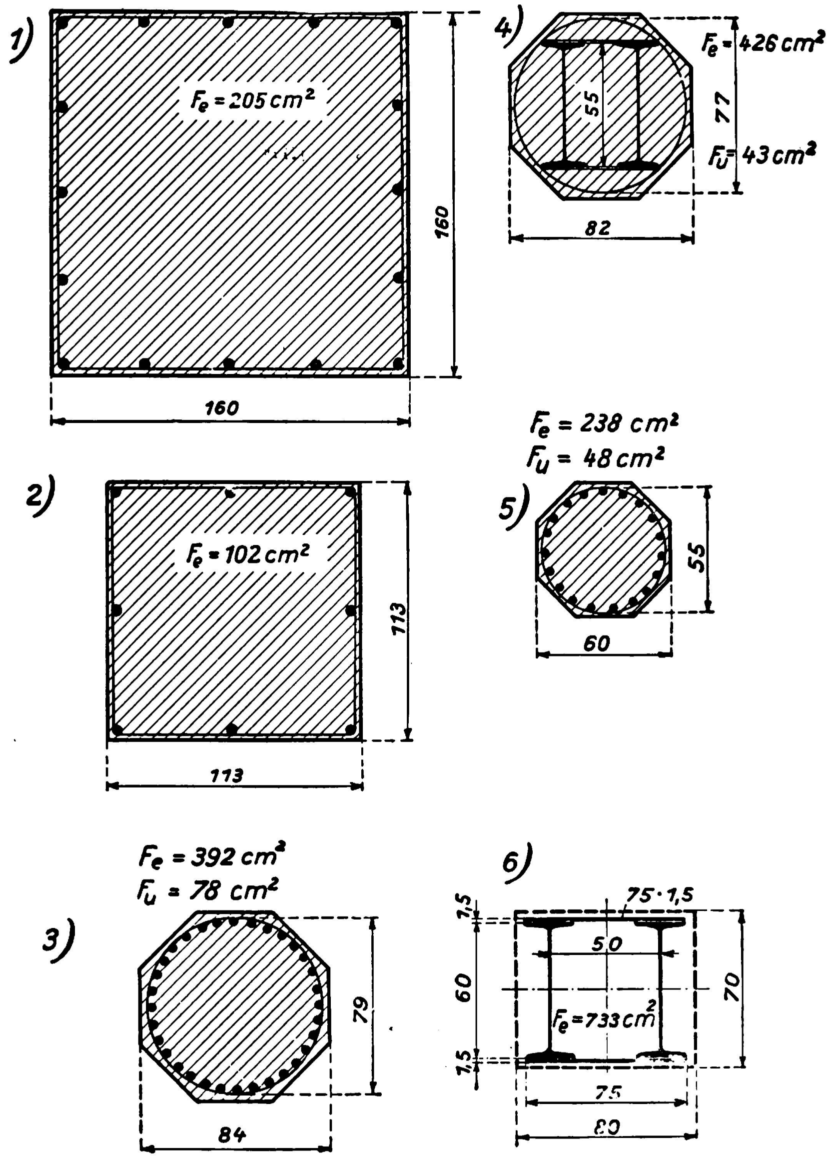

Fig. 1.

type or high-grade round bar steel reinforcement, in order to obtain a far reaching adaptation of all materials to the effects of forces, and to obtain this way more economy. This composite style of construction, which has been carried out quite frequently in Austria, England and America, represents a healthy development of both steel construction reinforced concrete construction, because it offers a technically correct combination of both these types of construction and because the inert concrete encasing added to steel structures, which is often necessary as a means to withstand fire and corrosion, is used for the transmission of forces.

B. Beams with high-grade steel reinforcement.

a) *Tests.*

The experiments carried out from 1912 to 1913[4] on T-beams with a length of 2.7 m and a reinforcement percentage of 1.5, where yield stresses of the steel were 2.5 to 3.5 tons/cm², showed a ratio of maximum stress in the steel of $\sigma_e : \sigma_s = 1.05$ to 1.09 to the yield limit.

The fundamental experiments carried out with Isteg steel reinforcement[5] were made in the years 1927 and 1928 and the results showed that with a twisting having a pitch 12.5 times the thickness of the bar, the maximum effect was attained with 1.5 increase of the yield limit as compared with the non-twisted steel, and a 1.1 increase of tensile strength, while the modulus of elasticity of the steel was reduced to $E_e = 1700$ tons/cm². Eight girders with a cross-section of 20 · 30 cm were used for purposes of comparison of the grip resistance of the Isteg steel; while experiments with 12 reinforced concrete beams with cross sections of 10 · 20, 15 · 20, and 20 · 20 cm and with three different percentages of reinforcement varying between 0.4 and 1.8 % showed that the reinforcement of the Isteg steel absorbed 1.43 to 1.5 times the amount of stress as similar girders with St 37 reinforcement, and that in addition, the ratio $\sigma_e : \sigma_s = 1.2$ was observed in the case of light reinforcement and 1.1 in the heavier reinforcement. Four slabs were tested for comparison purposes, they were reinforced with Isteg steel and St 37 in percentages of 0.24 or 0.38 and the ratio of steel stresses at moment of fracture was found to be 1.53. The ratio $\frac{\sigma_e}{\sigma_s}$ was an average of 1.3 for all four slabs.

The tests[6] carried out in 1928 concerned 36 T-beams each 2.7 m long of which 8 beams were reinforced with St 48, six with St 80, four with Isteg steel, and eight, for purposes of comparison, with St 37. Two qualities of concrete — 150 and 300 kg/cm² cubic resistance were provided. The percentages of longitudinal reinforcement was abt. 0.5, 1.1 and 1.7 %. The following were the most important results: the quality of the reinforcement steel used had no influence on the deflections or the formation of cracks with identical stressing of the steel below the yield limit. The quality of the concrete had also no important influence within the zone of comparable steel stresses. The carrying capacity for all high-grade steels, where the fracture was caused by the effects of moments, depended on the yield point stress, just the same way as for reinforcement of St 37. The demands on the composite structure prove greater the higher the steel stresses. Where other conditions are identical high concrete strength increases the effect of composite action and raises the carrying capacity when the latter is dependent on the compound action of the two materials.

In 1932 experiments were carried out[7] on eight beams of T-section, rein-

[4] Schubwiderstand und Verbund (Shearing strength and composite structures), Springer, Berlin, 1913; and Zeitschrift für Betonbau, 1913, Issue 8, 9; 1914, Issue 1.

[5] Beton und Eisen, 1928. P. 233 et seq.

[6] Bauingenieur, 1929, Nr. 7.

[7] Issue 14 of the Austrian Commission on reinforced concrete. Versuche an Balken mit hochwertiger Stahlbewehrung und an streckmetallbewehrten Platten (Tests on beams with high-grade steel reinforcement and slabs reinforced with expanded metal). Vienna, 1932.

forced with St 55. The yield limit was: $\sigma_s = 3.7$ tons/cm², the tensile strength of the reinforcement 6.2 tons/cm². The cube strength was 265 and the prism resistance 218 kg/cm². The upsetting was measured to be $2\,^0/_{00}$ on an average. The main results were as follows:

Percentage of reinforcement	$=$	0.34	0.73	1.1	1.45 %
Formation of cracks based on calculated steel stresses	$\sigma_e =$	1200	800	800	650
Rupture based on calculated steel stresses	$\frac{\sigma_e}{\sigma_s} =$	1.3	1.2	1.1	1.03.

During the years 1930—1932[8] fatigue and ordinary static bending tests were carried out with 32 T-beams with eight different kinds of reinforcement, in sets of four identical test pieces. The most important results were:

Percentage of reinforcement	$\mu =$	0.56	0.85	1.4 %	
		Istegsteel	St 37	St 55	Istegsteel
Stress at the moment of cracking		1100	900	700	700 kg/cm²
Maximum stress $\frac{\sigma_e}{\sigma_s}$	$=$	1.45	1.12	1.10	1.23

In reinforced concrete slabs with expanded metal reinforcement[9] of 4300 to 5300 kg/cm² tensile strength without yield limit and with a percentage of reinforcement of 0.27 to 0.57 %, the highest steel stresses for rupture were $\sigma_e = 4600$ to 5000 kg/cm² (average = tensile strength).

Comparison tests carried out with beams reinforced with St 37 and on girders with Tor-steel reinforcement[10] gave the following results:

	St 37 with $\sigma_s = 2.8$		Tor-steel of $\sigma_s = 4.6$ tons/cm²			
$\mu =$	0,69		0,37	0,70		1,43 %
σ_p	94	162	94	94	162	162 kg/cm²
$\frac{\sigma_e}{\sigma_s} =$	1,11	1,27	1,19	1,07	1,30	1,10

Further important data concerning the behaviour of high-grade steel was obtained by experiments carried out by the German Commission on reinforced concrete[11] in which St 37, St 48 and Si-Steel were used and further those of the Austrian Commission on reinforced concrete[12].

8 Issue 15 of the Austrian Commission on reinforced concrete, 1935.

9 Issue 14 of the Austrian Commission on reinforced concrete, 1935.

10 Not yet published.

11 Issue 66, 1920.

12 Issue 7 (1918), Issue 14 (1933).

We would mention information taken from these latter tests:

Percentage of reinforcement	$\mu =$	0.39	0.78	1.77	2.65 %
With St 55 and inferior quality of concrete	$\frac{\sigma_e}{\sigma_s} =$	1.35	1.14	< 1	< 1
With St 55 and high-grade concrete . .	$\frac{\sigma_e}{\sigma_s} =$	1.45	1.31	1.21	1.08
With Isteg-steel and high-grade concrete	$\frac{\sigma_e}{\sigma_s} =$	1.60	1.48	1.34	1.17

b) *Deflection and formation of cracks.*

Within the range of tests which includes steel of $\sigma_s = 2.2$ to nearly 5 tons/cm², beams of high-grade steel reinforcement, behave in the same way as girders of St 37 as regards deflection and formation of cracks under similar stresses below the yield limit, provided that the shape and surface of the reinforcing bars and the quality and composition of the concrete are the same. The first cracks occurred for bending tensile stresses (calculated according to the condition I with a coefficient n, corresponding to the ratio $E_e : E_b$ for low stresses), which more or less correspond to the bending-tensile stresses of the non-reinforced concrete beams. The actual steel stress in existence just before the first crack occurs depends on the ductility of the concrete. At extreme fibres of the beam the ductility has a value of 1 to $3 \cdot 10^{-4}$, to which corresponds an actual tensile stress of the reinforcing bars of $\sigma_{ez} = 150$ to 500 kg/cm². The stress in steel $\sigma_{e\,II}$, acting after (assumed) complete cracking of the concrete tensile zone, (calculated according to condition II for $n = 15$), is very variable with the percentage of reinforcement $\mu = \frac{F_e}{b\,h}$. In this connection b presents the width of the tensile zone of the concrete. According quantitive values this observation also holds good for T-beams where b stands for the width of the web. Where the tensile strength of concrete is $\sigma_{bz} = \frac{\sigma_p}{6}$ [13]

he ratio $\frac{\sigma_{e\,II}}{\sigma_p}$ given in Fig. 2 can be approximately applied: [14]

$$\sigma_{e\,II} = \left(1 + \frac{0{,}035}{\mu}\right) \sigma_p \tag{3}$$

Before the first crack forms the stresses in the concrete are transmitted to the steel by adhesive forces. On account of a certain lack of homogeneity in the composition of the concrete and the steel surface these forces are unevenly distributed. In one place the reinforcing bars show signs of yield and this signifies that the first crack on the point to be formed. The distribution of the (grip) adhesive stresses τ, (slipping and friction) of the tensile stresses in concrete σ_{bz} and the tension in the steel σ_e are shown in Fig. 3 as being situated between two

[13] Average resulting from a number of experiments.

[14] Cp. *Saliger:* Der Eisenbeton (Reinforced concrete), 6th Edition, P. 165 et seq.

neighbouring cracks. If the curved line τ, is replaced by straight lines, as shown dotted in the Fig., we receive for the rectangular section, for the difference in tensile ΔZ which is transmitted by the concrete to the tensile reinforcement, having a circumference u, the following relation

$$\Delta Z = F_e \cdot \Delta \sigma_e = \frac{b d^2 \sigma_{bz\,max}}{6 z} = \frac{u e \tau_{1\,max}}{4}$$

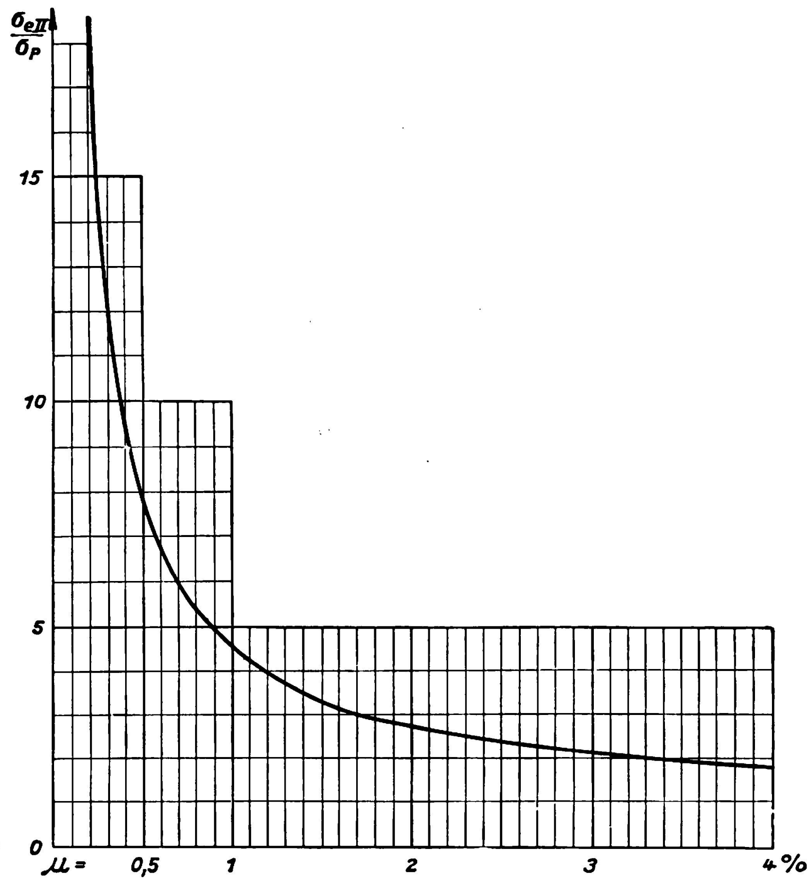

Fig. 2.

From this probable average distance of the cracks can be calculated as follows:

$$e = \frac{2 b d^2 \sigma_{bz\,max}}{3 u z \tau_{1\,max}} \tag{4}$$

with $z = 0.9 h$ and $h = 0.9 d$ we obtain:

$$e = \frac{0,9 \, bh \, \sigma_{bz\,max}}{u \tau_{1\,max}} \tag{4 a}$$

These expressions show that the distance between cracks with the same adhesive surface u increases with increased depth and width of the beam. Taking m as the number of reinforcing bars, we receive for round bar reinforcement

$$u = m \, d_e \, \pi = \frac{4 F_e}{d_e}$$

If $\mu = \frac{F_e}{bh}$ then:

$$e = \frac{0,23 \, d_e \cdot \sigma_{bz\,max}}{\mu \, \tau_{1\,max}} \tag{4b}$$

From this ratio it will be seen that the interval between the cracks diminishes as the grip resistance of the reinforcement in the concrete (that is where the surface is very rough, as with Isteg-steel and indented bars) increases and in proportion as the reinforcement has more strength, also when the bars are thinner and when the concrete tensile strength is lower. When the tensile strength

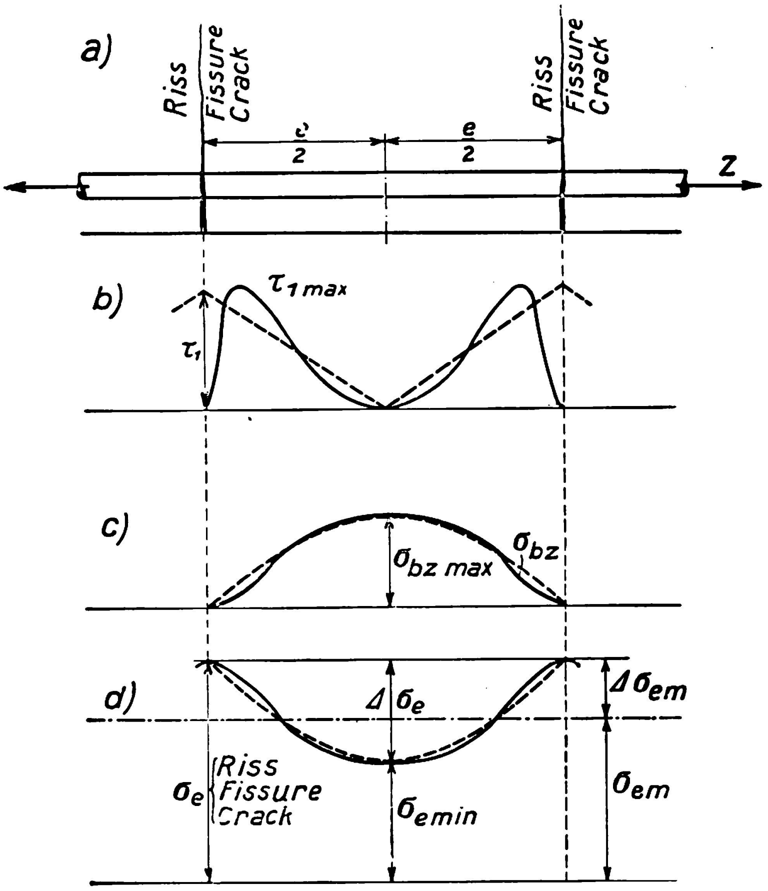

Fig. 3.

is lower there will be less adhesive resistance, and in this way the influence of the quality of the concrete is practically eliminated. If the approximations $\sigma_{bz} = \frac{\sigma_p}{6}$, and also the tensile strength $\sigma_z = \frac{\sigma_{bz}}{2} = \frac{\sigma_p}{12}$, are introduced we receive $\tau_1 = \sqrt{\sigma_p \sigma_z} = 0.3\ \sigma_p$ then the average distance between cracks in the case of round bar reinforcement will be:

$$e = \frac{0{,}13\ d_e}{\mu} \tag{4c}$$

For instance, with $d_e = 2$ cm and $\mu = 0.01$ the distance between the cracks will be $e = 26$ cm.

When the load on the girder is increased, the slipping movement of the steel reinforcement will also increase and the result will be the distribution of stresses shown in Fig. 4. The comparatively slight resistance to friction or to the movement of slipping of the steel will be the only action exerted in the neighbourhood of the cracks, and the entire grip resistance will only come into play at some further distance from the cracks. The length e_1 is determined by the ductility of the concrete and approaches a definite minimum value for the maximum stresses in the steel. As the stresses in the steel increase, the length e_1, over which the whole of the grip resistance can develop, becomes shorter.

The *width of the crack* is:

$$\Delta e = k_R \varepsilon_R e_2.$$

In this formula $k_2 < 1$ is a coefficient which represents the distribution of steel stresses in the neighbourhood of the crack and the distortion of the concrete which is caused by the local shearing stress resulting from the grip

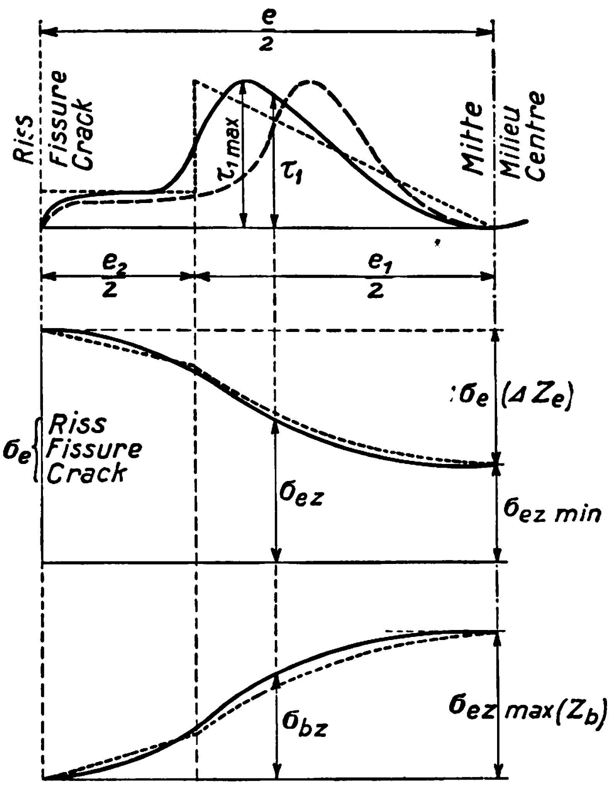

Fig. 4.

resistance[15]. The factor ε_R is the specific elongation of the steel at the crack. For instance, if $\sigma_e = 2100$ kg/cm², then $\varepsilon_R = 10^{-3}$. With e = 260 mm under the assumption that $e_2 = \frac{e}{2} = 130$ mm and $k_R = \frac{2}{3}$, then $\Delta e = \frac{2}{3} \cdot 10^{-3} \cdot 130 = 0.09$ mm. If $\sigma_{eR} = 3150$ kg/cm² (with high-grade steel, below yield limit) then $\varepsilon_R = 1.5 \cdot 10^{-3}$ and with $k = 0.9$ the width of the crack will be: $\Delta e = 0.9 \cdot 1.5 \cdot 10^{-3} \cdot 130 = 0.18$ mm. If the intervals between the cracks are smaller than those assumed above or those in the present example, for instance, in the case of artificially increased surface roughness, then the individual cracks will be less wide[16]. In the most unfavourable circumstances the cracks

[15] The shearing distortion of the concrete at the steel reinforcements may increase the apparent ductility of the concrete very considerably.

[16] Ausführliche Rißbeobachtungen (Observations on Cracks) N° 15 of the Austrian Commission on reinforced concrete (Fatigue tests) and punching tests (not yet published).

might have the width which corresponds to the elongation of the steel in the zone between two cracks. According to the experiments[17] which have been carried out, the widths of the cracks from 0.2 to 0.3 mm are not important from the point of view of protection against corrosion of the steel when high-grade concrete is being used. From this it follows, that from the point of view of crack formation the use of high-grade steel and in particular steel with artificially roughened surface, and the admission of high stresses up to 2200 kg/cm² in heavily reinforced structural elements (webs of T-beams) and up to 2500 kg/cm² in less strongly reinforced beams (rectangular beams and slabs) is permissible without the durability being reduced. Of course it is assumed in this connection that the work is of good quality and that sufficient safety against shearing effects is guaranteed.

c) *Demands made on shearing resistance.*

As the quality of the steel increases, so does under otherwise equal conditions the resistance to shear stresses and compound action in reinforced concrete beams on account of greater shear forces. No new rules and regulations are required for this. The thesis enunciated on the basis of tests and theory referring to reinforcement of St 37 suffice. The measures deduced from tests with St 37 reinforcement and applied to structural designing have been confirmed in every respect by tests made with beams reinforced with high-grade steel. The tests made with high-grade steel have shown, in particular, that the use of better quality concrete is only necessary when the compressive and shearing stresses of the concrete are so high that the quality must be raised. The same can be said of the grip forces. In order to keep these below the permissible limits, it will be necessary to increase the adhesive surface by employing a greater number of the reinforcing bars of smaller diameters. The radius of the bends at the bending points of the inclined bars and of the hooks at the end must be increased in order to prevent local overstressing of the concrete, which may result in cracks. At the points of bending a radius of 5 d_e is rarely sufficient. It should be increased to at least 10 d_e. The measures aiming at improving the compound action are similar to those taken for ensuring satisfactory shearing resistance.

d) *The plastic range under rupture conditions.*

With heavily reinforced beams the cause of fracture is connected with the surmounting of the compressive resistance of the compression zone of concrete, while the tensile strength of the reinforcement is not being fully utilised. The use of high-tensile steel is therefore useless in such a case (unless it were possible to produce high-grade steel with a still higher modulus of elasticity E_e.

Beams which are only *lightly reinforced* fracture when the tensile resistance is overcome. It is with these where the properties of high-tensile steel are of importance. For this reason we are now only considering light reinforcement. Later on we shall explain what is meant by tensile resistance and also take up

[17] Tests made by *Honigmann:* Beton und Eisen 1935. P. 307.

the question of the limit which determines one or other of these resistances and which depend on the good quality of the concrete or the steel.

The tests proved that in lightly reinforced beams the cause of fracture was also the crushing of the compressive zone in the concrete. The reason of this phenomenon lies in the fact that owing to great expansion of the tensile reinforcement, the pressure zone becomes considerably reduced, the consequence of this being that compressive stresses grow to such an extent that the zone of pressure is destroyed and the carrying capacity exhausted. This confirms the fact that for heavily and lightly reinforced members the compressive resistance of the concrete is overcome in the state of fracture, this being at once where the reinforcement is heavy and more gradually where it is light.

In the state of fracture the pressure in the compressive zone is distributed according to the curved line seen in Fig. 5. The maximum stress is the prism

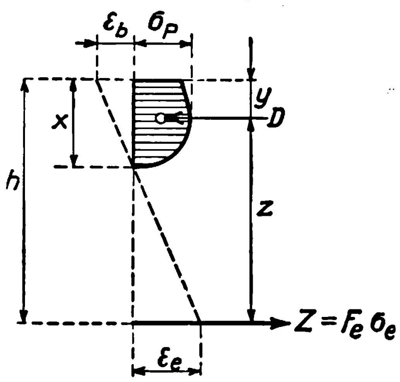

Fig. 5.

strength σ_p of the concrete. For well known reasons, it is lower than the cube strength σ_w. A number of experiments have shown that $\sigma_p = 0.7$ to $0.9\,\sigma_w$ and $0.75\,\sigma_w$ can be taken as the average figure. The effective compressive force is $D = k\,b\,x\,\sigma_p$.

Here the coefficient k, in accordance with experiments which have been made and according to the behaviour of the concrete under deformation, is 0.8 to 0.9, $k = 0.85$ can be taken as an average. The position of the centre of compression D is related to k and may be expressed as $y = \frac{k\,z}{2}$. The tensile force of the reinforcement is $Z = F_e \sigma_e$. We indicate the proportion of the depth of the compressive zone to the depth of the beam by $\xi = \frac{x}{h}$, the participation of the reinforcement by: $\mu = \frac{F_e}{b\,h}$ and the ratio of the tension in steel σ_e, present when fracture occurs, to the prism strength of the concrete with $\beta = \frac{\sigma_e}{\sigma_p}$. Thus we receive with the above terms:

$$\xi = \frac{\beta\,\mu}{k} \qquad (5)$$

The nominator $\beta\,\mu$ is the measure for the reinforcement and the depth from which we see that the ratio of the zone of pressure to the depth of the beam is in direct relation to the amount of reinforcement. For the state of fracture of lightly reinforced beams we have: $\sigma_e \geqq \sigma_s$, therefore $\sigma_e = \alpha\sigma_s$, and hence the ratio $\beta_s = \frac{\sigma_s}{\sigma_p} = \frac{\beta}{\alpha}$ indicates a definite criterion of the qualities of the material.

With steel having an actual yield limit, for instance, St 55 in Fig. 6, the stress in the steel within the zone of yield from ε_s to ε'_s is a fixed value, thus: $\sigma_e = \sigma_s$. With greater elongations $\varepsilon > \varepsilon'_s$ (zone of hardening) is $\sigma_e > \sigma_s$.

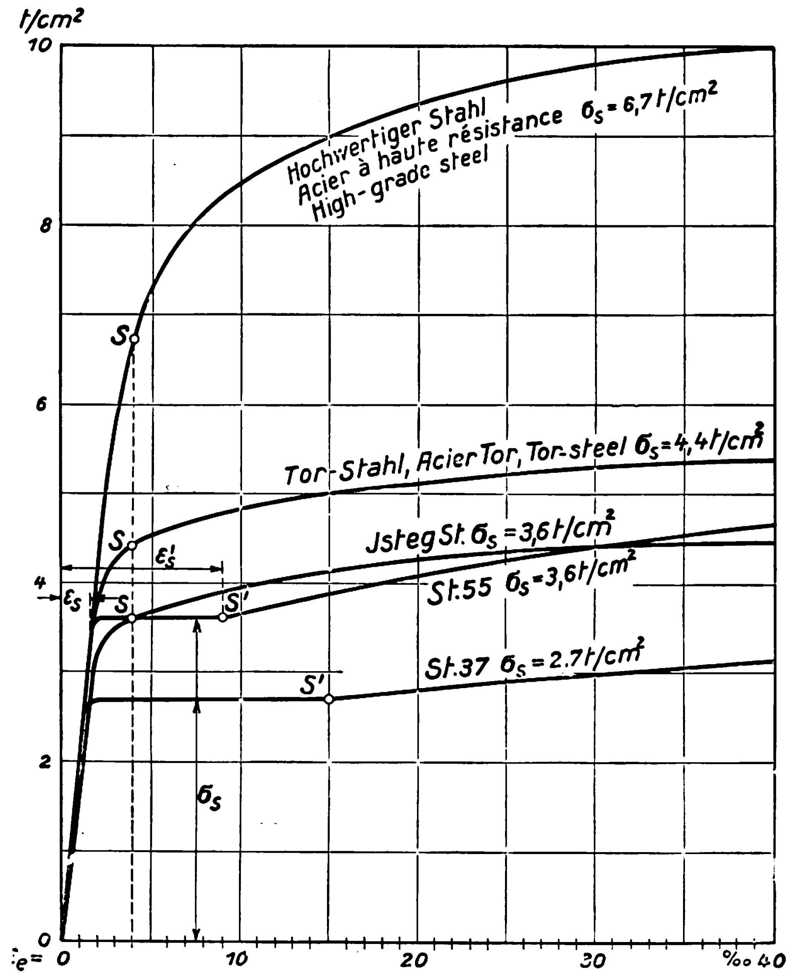

Fig. 6.

Where steel has a yield limit by definition for instance, Isteg steel with a yield limit of 0.4 %, as shown in Fig. 6, is $\sigma_e > \sigma_s$, if the elongation is greater than the yield expressed by definition.

Fig. 5 shows $\xi = \frac{x}{h} = \frac{\varepsilon_b}{\varepsilon_b + \varepsilon_e}$.

The quantity ε_b is 2 to 7 $^0/_{00}$ as has been received from tests $\varepsilon_b = 5\,^0/_{00}$ can be taken as the average value. The lever arm of the internal forces is

$$z = h - y = h - \frac{kx}{2} = \left(1 - \frac{k\xi}{2}\right)h = \left(1 - \frac{\beta\mu}{2}\right)h$$

With $M = Dz - Zz$ and substituting the above coefficients we obtain the carrying coefficients:

$$\left.\begin{aligned} m &= \frac{M}{bh^2\sigma_p} = \frac{\beta\mu}{2}(2-\beta\mu) = \frac{\alpha\beta_s\mu}{2}(2-\alpha\beta_s\mu) \text{ and} \\ m' &= \frac{M}{bh^2\sigma_s} = \frac{m}{\beta_s} = \frac{\alpha\mu}{2}\cdot(2-\alpha\beta_s\mu). \end{aligned}\right\} \quad (6)$$

First case. $\sigma_e = \sigma_s$ *or* $\alpha = \frac{\sigma_e}{\sigma_s} = 1$ *(light reinforcement).*

As $\beta_s\mu = k\xi$, we find in the zone of yield that for St 55 with about $\varepsilon_e = 1.7$ to $9\,^0/_{00} = 0.74$ to 0.35, $\beta_s\mu \cong 0.6$ to 0.3 for Isteg steel and other high-grade steels with yield limit fixed by definition $\varepsilon_e = 4\,^0/_{00}$ $\xi = 0.55$ $\beta_s\mu \cong 0.45$.

For instance, given a beam with $= 1.4\,\%$ reinforcement of St 55 $\sigma_s = 3500$ and $\sigma_p = 150$ kg/cm², then $\beta_s = \frac{3500}{150} = 23.2$ and $\beta_s\mu = 23.3 \cdot 0.014 = 0.33$. This value is situated between 0.6 and 0.3. For that reason the maximum stress in steel, for the state of fracture, is equal to the yield limit. The limit of reinforcement will exist when $\beta_s\mu = 0.6$. With the above ratios $\mu = \frac{0.6}{\beta_s} = \frac{0.6}{23.3} = 0.026$, i. e. a beam with reinforcement which is less than $2.6\,\%$ will break by overcoming the tensile resistance of the steel; where the reinforcement is higher than $2.6\,\%$, the tensile resistance will not be utilised to the full and the direct cause of fracture will be found in the crushing of the concrete. In dealing with these steels we speak of light reinforcement, when, assuming St 55 $\beta_s\mu < 0.6$ and for Isteg girders and other high-grade steels with yield limits expressed by definition: $\beta_s\mu < 0.45$.

For $\alpha = 1$ the carrying coefficient mentioned above:

$$m = \frac{M}{bh^2\sigma_p} = \frac{\beta_s\mu}{2}\cdot(2-\beta_s\mu) \quad (6a)$$

From this it will be seen that the carrying coefficient depends only on $\beta_s\mu$. For equal quality of concrete σ_p the ratio $= \frac{\sigma_s}{\sigma_p}$ increases, the higher the quality of the steel; thus μ can be proportionately smaller in order to obtain the same carrying capacity. If therefore we replace a steel having a yield limit of σ_{s1} and a percentage of reinforcement μ_1 by another steel with a higher yield limit σ_{s2} and μ_2 as its percentage of reinforcement, the bearing capacity of the beam will remain the same if $\sigma_{s1}\cdot\mu_1 = \sigma_{s2}\cdot\mu_2$ or if $\mu_2 = \frac{\sigma_{s1}}{\sigma_{s2}}\cdot\mu_1$.

As a matter of fact in this case the compressive stress in the concrete laid down by the usual calculation for n (= 15) increases. However as this does not represent a measure of compression resistance and with it the safety of the concrete pressure zone, the permissible stresses in the usual calculation can be increased, without reducing the safety factor in the concrete pressure zone.

If, for instance, $\frac{\sigma_{s1}}{\sigma_{s2}} = \frac{2400}{3600} = \frac{2}{3}$, then $\mu_2 = \frac{2}{3}\mu_1$.

An increase of the concrete stresses rated at n corresponds by an average of 15 % to the reduced quantity of reinforcement μ_2. The permissible stress can be increased by this amount. The depth of the compression zone while in a state of fracture is not changed by an equivalent reinforcement capable of equal resistance. (See formula 5). The carrying coefficient $m' = \frac{M}{b h^2 \sigma_s} = \frac{\mu}{2}(2 - \beta_s \mu)$ shows clearly what the influence of the quality of the concrete is. As the concrete resistance σ_p decreases, so the $\beta_s = \frac{\sigma_s}{\sigma_p}$ increases if the steel remains of unchanged quality. At the same time there is a reduction, even if only slight, of the carrying capacity of the beam or else a larger amount of reinforcement will be necessary in order to obtain the same carrying capacity.

Second case. $\sigma_e > \sigma_s$ *or* $a = \frac{\sigma_e}{\sigma_s} > 1$ *(very light reinforcement).*

When reinforcement is only light the stress in the steel of the beam when in a state of fracture reaches the zone of hardening and for that reason it exceeds the yield limit. The tensile resistance of the reinforcement is greater than what would correspond to the yield stress. The ratio α follows the formula given below for reasons explained elsewhere[18]:

St 55 $\alpha = \frac{\sigma_e}{\sigma_s} = 0{,}9 + \frac{0{,}03}{\beta_s \mu}$ valid for $\beta_s \mu = 0{,}07$ to $0{,}3$

for Istegsteel and other high-grade steels without pronounced yield limit $\alpha = \frac{\sigma_e}{\sigma_s} = 0{,}93 + \frac{0{,}035}{\beta_s \mu}$ valid for $\beta_s \mu = 0{,}1$ to $0{,}6$

To these coefficients correspond elongations in steel for the state of fracture up to about 40 ‰. When making experimental observations concerning distribution of cracks and taking into consideration the statements made in connection with Point b) they led to various crack-widths of the order of 2 to 5 mm. Elongations of steel above 40 ‰ do not occur as a rule in steel when used for reinforcing. Therefore it is immaterial for the carrying capacity of ferro-concrete beams how far the fracture elongation of the test piece exceeds the maximum elongation of the steel as used in reinforced concrete beams. The further course followed by the stress strain curve is of no importance as far as reinforced concrete beams are concerned. It is therefore not justified to demand for the purpose of reinforcement, steels of much higher rupture elongation than 40 ‰, it suffices with say 60 ‰. If the reinforcement steel has a lower rupture elongation, then the maximum load will have been reached when the reinforcement bars fracture[19].

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[18] Beton und Eisen 1936.

[19] 6. 14 of the Austrian Commission on reinforced concrete. Versuche mit Streckmetallplatten (Experiments on ductile metal plates). P. 102 et seq.

Fig. 7 gives the $\alpha = \frac{\sigma_e}{\sigma_s}$-coefficients for St 55 and high-grade steel without the yield limit and also the coefficients of St 37 for purposes of comparison. Fig. 8 gives the carrying capacity $\frac{M}{b\,h^2}$ for concrete with $\sigma_p = 150$ kg/cm² and reinforcement of high-grade steel of $\sigma_s = 6.7$ tons/cm², $\sigma_s = 3.6$ tons/cm² (St 55 and Isteg-steel, in the latter case also for concrete with $\sigma_p = 100$ and 200 kg/cm², dotted), and St 37 $\sigma_s = 2.4$ tons/cm² for purposes of comparison.

From this we deduce:

1) When in a state of fracture the tension in the reinforcement exceeds the yield limit to a greater extent in proportion as the amount of reinforcement $\beta_s\,\mu$ is smaller. Where $\beta_s\,\mu$ is equivalent, Isteg steel and other high-grade steel

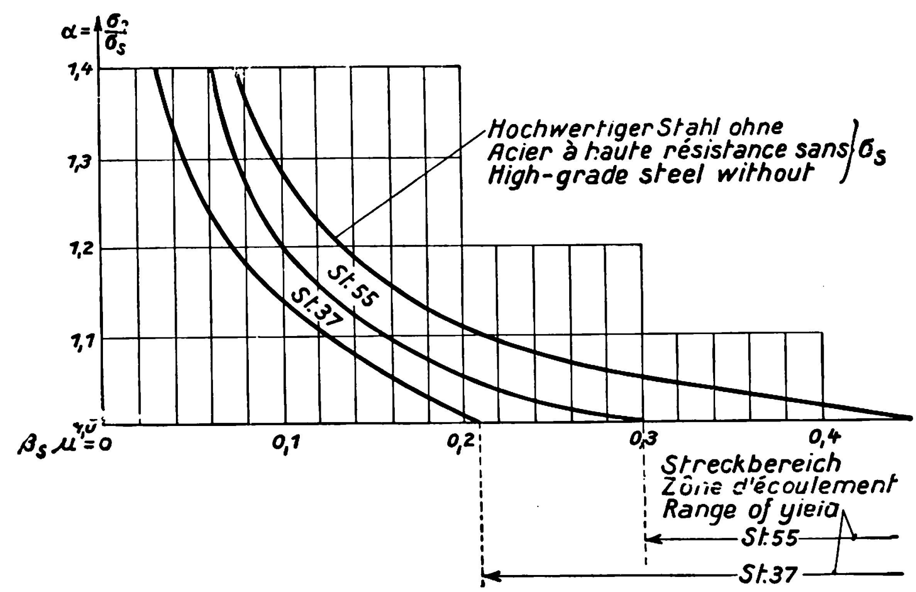

Fig. 7.

without any marked yield limit of higher stresses (corresponding to their yield limit by definition) are as satisfactory as St 55 and the steel with higher tension as satisfactory as St 37. Expressed in general terms this means that where the elongation of the reinforcement is greater, its utility will be less provided the conditions are equivalent.

2) Where the concrete strength σ_p is lower, β_s will be greater with the same quality of steel. Where the amount of reinforcement is equal $\beta_s\,\mu$ will be greater and thus $\alpha = \frac{\sigma_e}{\sigma_s}$ will be smaller, that is, the carrying capacity of very lightly reinforced girders will be considerably reduced with reduced quality of the concrete. High-grade concrete increases the carrying capacity considerably. For instance, for concrete of $\sigma_p = 100$ kg/cm², $\beta_s\,\mu = 0.20$ for concrete with $\sigma_p = 200$ kg/cm², $\beta_s\,\mu$ will equal 0.10. In this connection the stress of $\frac{\sigma_e}{\sigma_s} = 1.05$ rises to 1.20, that is, an increase of 14 % in the case of St 55. The carrying capacity rises by very much the same amount.

3) If a St 37 reinforcement of a yield limit of σ_{s1}, and the percentage of reinforcement μ_1 is replaced by a higher grade of steel with a ductile limit of σ_{s2} and with an amount of reinforcement of $\mu_2 = \frac{\sigma_{s1}}{\sigma_{s2}} \cdot \mu_1$, then, provided the $\beta_s \mu$ is the same, the coefficient α of the higher-grade steel will be greater, that is, the utilisation of the higher-grade steel will be greater in proportion to its yield limit and the carrying capacity will be higher too. If, for instance, a St 37-reinforcement with $\beta_s \mu = 0.20$ is replaced by Isteg-steel, then the coefficient of $\alpha = 1.00$ will rise to 1.11. The carrying capacity of the beam reinforced with

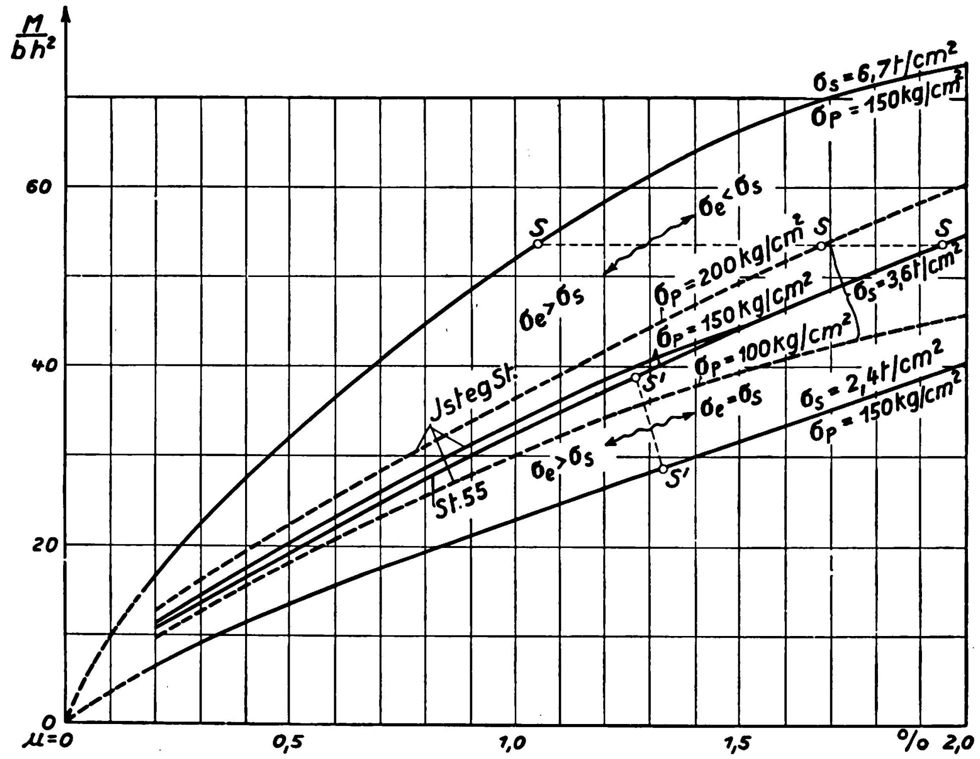

Fig. 8.

Isteg-steel will then be approximately 11 % higher or if attempts are made to reach the same carrying capacity, then the reinforcement percentage of the Isteg reinforced girder can be reduced to below the quantity μ_2 determined above.

4) A number of conclusions can be drawn from Fig. 8. For instance, the carrying capacity of beams with equivalent percentage of reinforcement μ increases less than what would correspond to the increased yield limit, that is, reinforcement with twice as high a yield limit corresponds to less than twice the carrying capacity.

If a reinforcement is replaced by one of higher grade, then its section required to reach the same carrying capacity will be considerably less than would be anticipated if from reversed ratios of their yield limits. If concrete with a strength of 100 kg/cm² is replaced by one having a strength of 200 kg/cm², the carrying capacity would be increased by 20 to 25 % provided the amount of reinforcement is kept the same or, in order to attain the same carrying capacity, the reinforcement can be reduced by the same amount.

When considering the plasticity of the concrete and of the reinforcement for the state of fracture, it will be seen that a whole series of significant facts concerning the influence of high-grade steels and of concrete quality will be noted which cannot be obtained with the n-process.

e) *Dimensioning.*

The calculation of dimensions of reinforced concrete beams with high-grade steel reinforcement can be effected in the same way as for St 37 reinforcement. Starting from the plastic zone in a state of fracture, the conception of permissible stress and the coefficient n loses its significance. The determination of permissible stresses always gives rise to conjecture and is the cause of complex divergencies of opinion. The simplest way of determining dimensions is by basing them on the quality of the material:

$$\beta_s = \frac{\sigma_s}{\sigma_p}$$

and of the requisite safety factor s, according to the deduction made by formula 6;

$$h = \sqrt{\frac{2}{\alpha\,\beta_s\,\mu\,\sigma\,(2 - \alpha\,\beta_s\,\mu)}} \cdot \sqrt{\frac{s\,M}{b\,\sigma_{p\,min}}} = \alpha \sqrt{\frac{s\,M}{b\,\sigma_{p\,min}}} \tag{7}$$

$$F_e = \frac{s\,M}{z \cdot \alpha\,\sigma_s} = \mu\,b\,h$$

Here, in accordance with a proposal made elsewhere[20] $\sigma_{p\,min} = \frac{2}{3}\,\sigma_{p\,average} = 0.5$ to $0.6\,\sigma_{w\,average}$ and in general, and if necessary, while taking into consideration an increase of impact added to the load, then s should be taken as equal to 2.

f) *Conclusion.*

Even if it is not to be expected that high-grade steel will oust St 37 reinforcement in ferro-concrete construction, there are many possibilities of application of high-grade steel bridge construction and structural engineering, generally used in combination with high-grade concrete which lead to further technical and economic development. The main obstacle to more general utilisation of high-grade steels in the past was due to misgiving entertained regarding an excess of crack formation, and that is why engineers were chary of raising tensile resistance of steel unduly, while maintaining the usual coefficient of safety. Another drawback lies in the commonly held opinion that high-grade steels, in particular those which have an artificially raised yield limit (Isteg-steel, Torsteel, etc.) corrode more readily, wear out more rapidly and cannot meet repeated fatigue tests. This view has not been confirmed by experiment[21].

Custom, sentiment and outlook often play a part in things of this kind and such prejudices will be thrown aside when the conviction gains ground that mis-

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[20] Beton und Eisen 1936.

[21] Dauerversuche an Balken mit St 37, 55, 80 und Istegbewehrung (Endurance fatigue tests on girders with St 37, 55, 80 and Isteg-reinforcement). Issue 15 of the Austrian Commission on reinforced concrete.

givings as to high stresses in steel and its effects upon durability are of little importance, or at any rate much exaggerated and that the main factor is to ensure that the structure be designed by experts and executed by skilled labour. So far Isteg-steel alone has made any real headway in this field of construction, while certain other high-grade steels are used for specialised work. However, this is only a first step on the road to further development of the ferro-concrete industry.

Summary.

This paper deals with the effects produced by high-grade steels used for columns and beams, the data being based on experiment and theoretical considerations. Elastic deformation, crack formation and stresses in composite construction when subjected to small loads (working loads) behave very differently than when subjected to maximum loads (state of fracture). In the latter case both materials come under plastic influence.

The determination of permissible stresses by the methods of calculation practised heretofore and providing for the use of the coefficient n (= 10 or 15) does not constitute a reliable standard of safety in the structure. The application of permissible stresses has lost its significance and the proposal is therefore put forward that the determination of the dimensions, both for beams and for columns, should be effected by taking into consideration the quality of the material used and the necessary safety factor as this results from experience.

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