General report

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II

General Report.

Generalreferat.

Rapport Général.

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Part I: The influence of stationary, permanent and repeated loading.

1) The carrying capacity of reinforced concrete beams in relation to the reinforcement provided has been the subject of investigation during the last few years by the Austrian Reinforced Concrete Committee, notably through the work of *Emperger, Haberkalt* and *Gebauer,* to whom great credit is due.

If, as in Fig. 1, the carrying capacity $\frac{M}{b \cdot h^2}$ is plotted as an ordinate and the amount of reinforcement $\mu = \frac{F_e}{b \cdot h}$ as an abscissa, two well known regions may be clearly distinguished in the resulting diagram, namely: 1) The region representing weakly reinforced beams — the more usual condition — wherein failure is determined by the yield point of the steel, and 2) the region representing the rarer case in which failure is determined by the compressive strength of the concrete.

It is indicated in Fig. 1 at the point marked II that according to the present method of calculation the first of these regions is not fully exploited. The suggestion put forward by *Emperger* and *Haberkalt* was intended to overcome this disadvantage by providing that the stress in the concrete might be increased by 20 % for the purpose of calculating the limiting amount of reinforcement which divides the two regions from one another. This solution, however, is not altogether satisfactory, because it is applicable only to rectangular cross sections, and because cases may arise in which the carrying capacity so calculated decreases when more reinforcing steel is added. This occurs, for instance, when on adding reinforcement the appropriate reading in the diagram is made to fall further to the right of point III, and is on a lower step in the line which denotes carrying capacity.

The experiments lately carried out at Dresden have served to bring out this point very clearly (Fig. 2). A series of reinforced concrete beams were examined in which merely the amount of reinforcement μ was altered. In the case of the usual commercial steel St. 37 the line AC in the diagram, Fig. 2, became practically straight, fixing the point C for the limiting amount of reinforcement, which divides the first mentioned region AC from the second region CD. The parabola which represents the carrying capacity as calculated by the ordinary

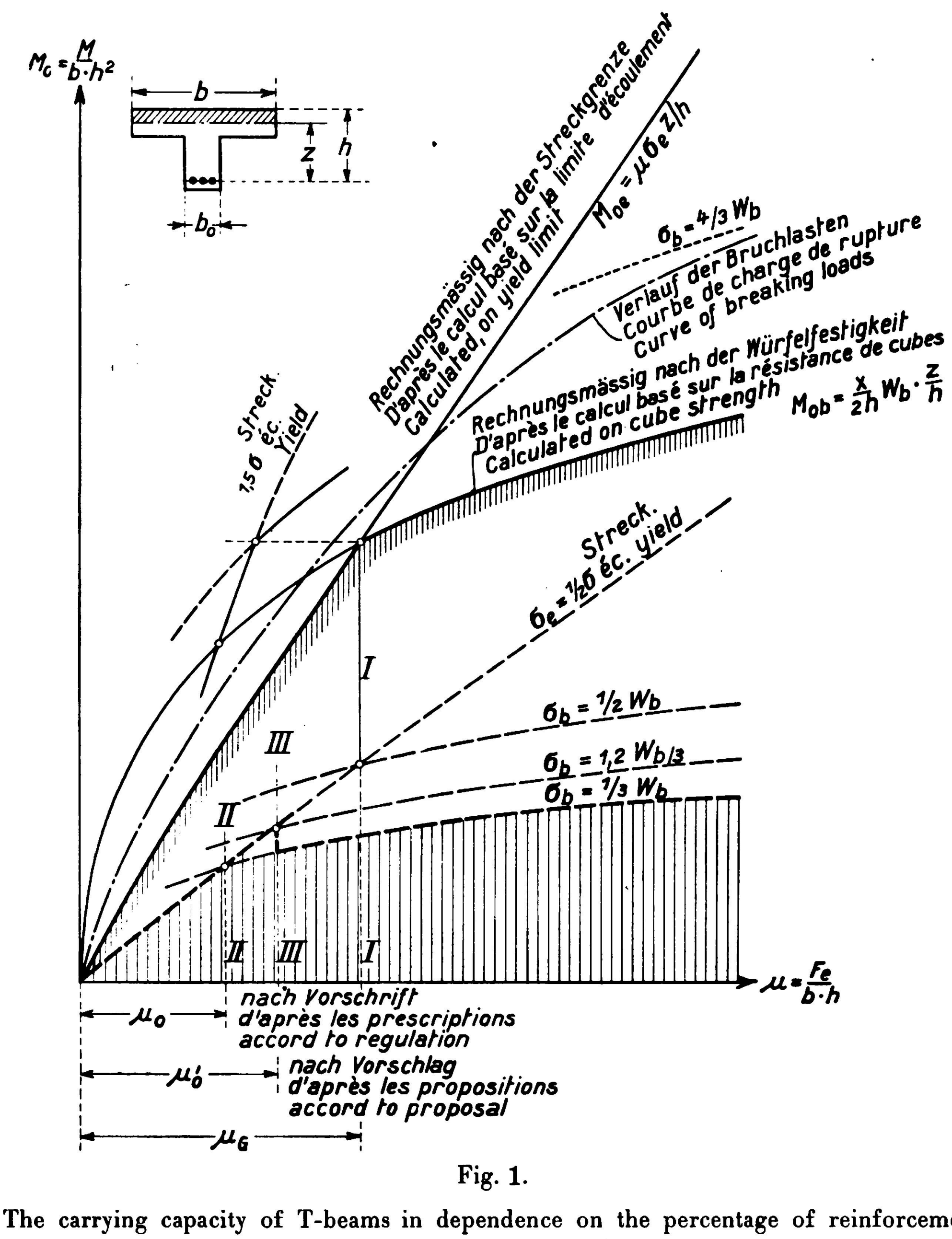


Fig. 1.

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

method indicates that the first region is far from being fully utilised, and this is true, in principle, also as regards the lines AEF and AGH which refer to high tensile steel such as Isteg. The conclusions which emerge from these experiments are the following:

So far as concerns the first region in the diagram, representing the usual case of weakly reinforced beams in which failure is conditioned by the yield point of the steel, there appears no justification to depart from the methods of calculation hitherto in use. Once the critical amount of reinforcement which marks the boundary between the two regions has been established by the experiments now in hand it will be permissible to extend the practical utilisation of the first

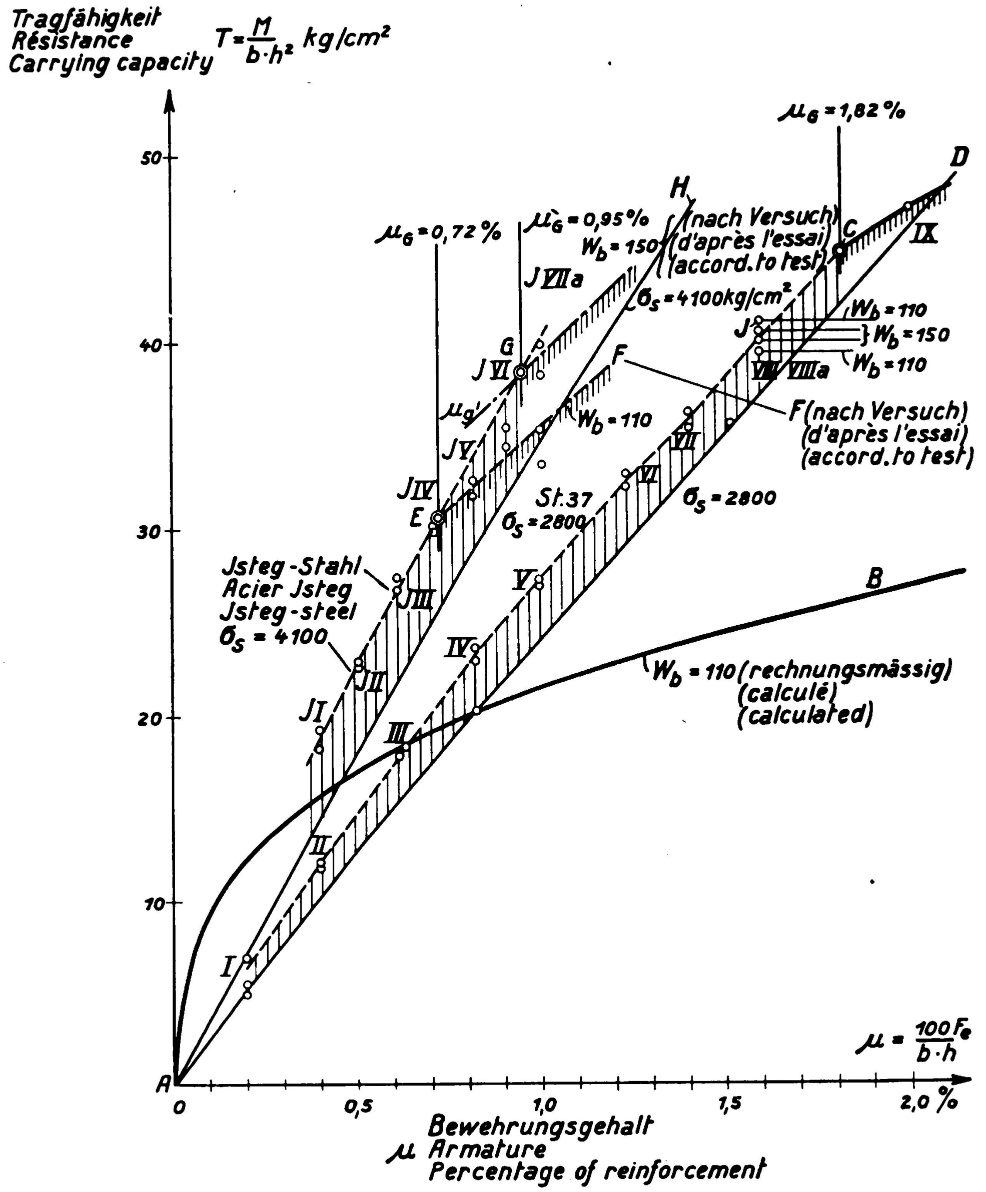


Fig. 2.

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

region up to the limit so ascertained, and the usual simple method of calculation will be applicable accordingly.

As regards the second region, representing the rarer case in which the governing condition is the compressive strength of the concrete, a new method may be introduced which will ensure more complete utilisation of the material and will serve the purpose of fixing the critical amount of reinforcement which separates the two regions. In this way the necessity for compression steel and for inclined haunches to the beams can be reduced to a minimum, and the design thereby improved.

Referring again to the first region — the case of lightly reinforced beams — a number of writers have joined in attacking the current methods of calculation, and especially the assumption of a fixed value for n whether this be 10, 15 or 20. *Emperger*, for instance, has shown in his latest paper that on the basis of the Austrian experiments the value of n as found graphically may be anything from 1 to 100, so that no justification exists for preferring a figure such as 10 or 15. Such a calculation, as indicated in the paper by *Saliger*, can be relevant only to the particular case where, at the moment of failure, the yield point of the steel in tension happens to be reached simultaneously with the prism strength of the concrete in compression. Even so the problem of the second region in the diagram, wherein failure is governed by the compression in the concrete, remains unsolved, for hitherto there has been no means of evaluating the stress that exists in the steel at the moment of failure.

Even if, as suggested by *Emperger*, the calculation for a rectangular cross section is to be simplified by the arbitrary assumption that the neutral axis is situated at the middle of the depth of the beam, the problem remains unanswered as regards tee beams and as regards the case where bending is combined with a longitudinal force.

The German Committee for Reinforced Concrete has not, up to the present, seen any justification for abandoning the old method of calculation with the assumption $n = 15$, particularly since the results so obtained are in very satisfactory agreement with those indicated by the Dresden experiments on beams of rectangular cross section.

In a paper by *Brandtzaeg* of Norway a notable suggestion is put forward for determining the safety of reinforced concrete sections subject to eccentric loading, based on experiments carried out at Stuttgart, in America, and by the author himself. The values of the stresses in members so loaded, calculated by this method, have led to the Norwegian regulations limiting the permissible compressive stress in such cases more strictly than the German rules. In this method, again, an arbitrary value is adopted for the elastic ratio, determined by the tangent to Talbot's parabola at the zero point; the problem does not admit of solution from considerations of equilibrium alone.

On the basis of the Dresden experiments mentioned above, *Friedrich* of Dresden has proposed a new method of calculation for beams comprised in the second of the two regions; a method which further offers the advantage of providing a simple determination of the limiting reinforcement which differentiates the two regions.

2) In the paper by Brice of Paris special emphasis is laid on the fact that from the point of view of the safety of a structure the dead load and the live load possess an entirely different significance, and that this significance is determined by the nature of the material used. As long ago as 1910 Caquot of Paris put forward the consideration that a structural member is to be regarded as sound when it not only takes up the elastic and reversible strains below its fatigue limit, but is also adequately resistant to those permanent deformations which

progressively increase over a notable period of time before, finally, reaching a limit at which their amount remains steady. The effect of this plastic type of deformation is to bring about an equalisation of the stresses, and it is, therefore, to be looked upon as a process of the structural member adapting itself to the loading imposed.

Reinforced concrete, however, is able to adapt itself, in this sense, only to the permanent or dead loading and not to the variable or live loading. Consequently,

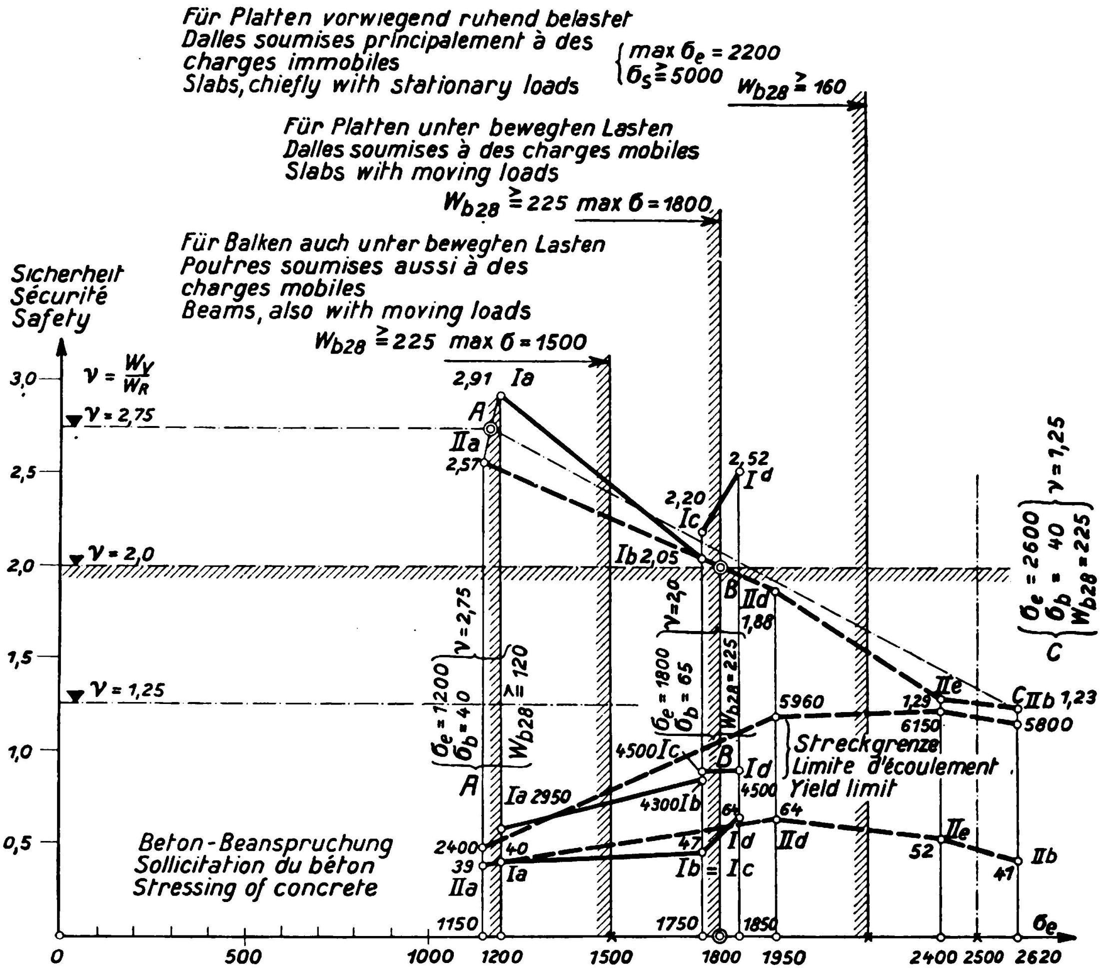


Fig. 3.

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

in the code adopted by the French Association of Reinforced Concrete Constructors the allowance for live load in the design is increased in relation to the dead load, as a means of taking account of the unfavourable circumstance described.

The experiments indicate that under frequently repeated loading the elastic deformations may be permitted to occur a large number of times but the plastic deformations only a relatively small number. This leads to the corollary that once a member has been fully loaded any further variable live loading to which it may be subjected by traffic should be such as to provoke only elastic deformations. Massive structures such as the heavier types of floor, mushroom floors, and bridge works come into a different category since in these the proportion of dead load is relatively high, compared with other types of construction. The smaller the fluctuations in stress attributable to live load, in proportion to the dead load, the more durable will be the construction.

3) The paper by *Graf* of Stuttgart deals with the effects of permanent and frequently repeated loading. According to the Stuttgart experiments the resistance to a permanent and stationary load may be taken as equal to at least 80 % of the strength as determined in ordinary breaking tests.

It is found, in confirmation of earlier experiments by *Probst* and *Mehmel* of Karlsruhe and *Roš* of Zurich, that resistance of concrete to frequently repeated loading in tension, compression or bending amounts to at least one half of the strength as determined by the ordinary compression test. As in the case of structural steelwork, however, if the frequently repeated load is superimposed upon a pre-existing static load the effect is to reduce the range of stress which can be withstood an indefinite number of times to below the value which it would have if there were no permanent load. Where a beam is exposed to this kind of dynamic stress care should be taken to make the radius to which the reinforcing bars are bent as large as possible and to anchor their hooks as carefully as possible in the concrete at the ends.

In the paper by *Gehler*, the Stuttgart fatigue tests on variously reinforced slabs are evaluated, and the concept of the "factor of safety under traffic", $v = w_v : w_r$, is introduced (Fig. 3). Here w_v denotes the maximum amplitude that can be resisted an indefinite number of times, as determined by the fatigue tests, and w_r denotes the greatest amplitude possible on which the statical calculations could be based. The value $v = 2$ was adopted as a suitable value for the slabs tested. It was found that this double factor of safety was realised in slabs reinforced with high tensile steel subject to a permissible stress of 1800 kg/cm² (25600 lbs. per sq. in) and made with concrete showing a minimum cube strength of 225 kg/cm² (3200 lbs. per sq. in).

Part II: Means for increasing the tensile strength of concrete and for reducing the liability of cracking.

1) Results of experiments on the usual method of preparation of concrete are given in the paper by *Bornemann* of Berlin. The tensile strength of concrete is governed, in the first place, by the tensile strength of the cement, and according to the new method of testing the latter should be determined by reference to a tensile-bending test carried out on a sample of mortar made by using the cement together with fine aggregate of varying size of grain. The tensile strength of the concrete depends also on the quality of the concrete (as indicated by the cube strength), on the granulation of aggregates (as indicated by the fineness modulus, or proportion of dust in the sand and sand in the aggregate as a whole), on the cement content, and finally on the water-cement ratio. Tensile tests on samples of concrete afford no satisfactory criterion, but good results have been obtained from tensile-bending tests carried out on concrete beams 70 cm long spanning a distance of 60 cm between supports, 15 cm wide by

10 cm deep, subjected to two symmetrical point loads at 20 cm distance. *Gehler* suggests that using the cube strength as a basis the tensile-bending strength can be approximate evaluated from the formula:

$$K_b = \sqrt[3]{W^2}$$

The results obtained by the formula have to be increased by up to 10% in case of damp plastic concrete of best granulation and decreased by up to 20% in case of very soft concrete.

Assuming a given cement content and a given granulation, the attainment of a particular degree of workability will depend upon the amount of water used in relationship to the shape and surface properties of the aggregate. The more compacted the aggregate the smaller will be the amount of water required and

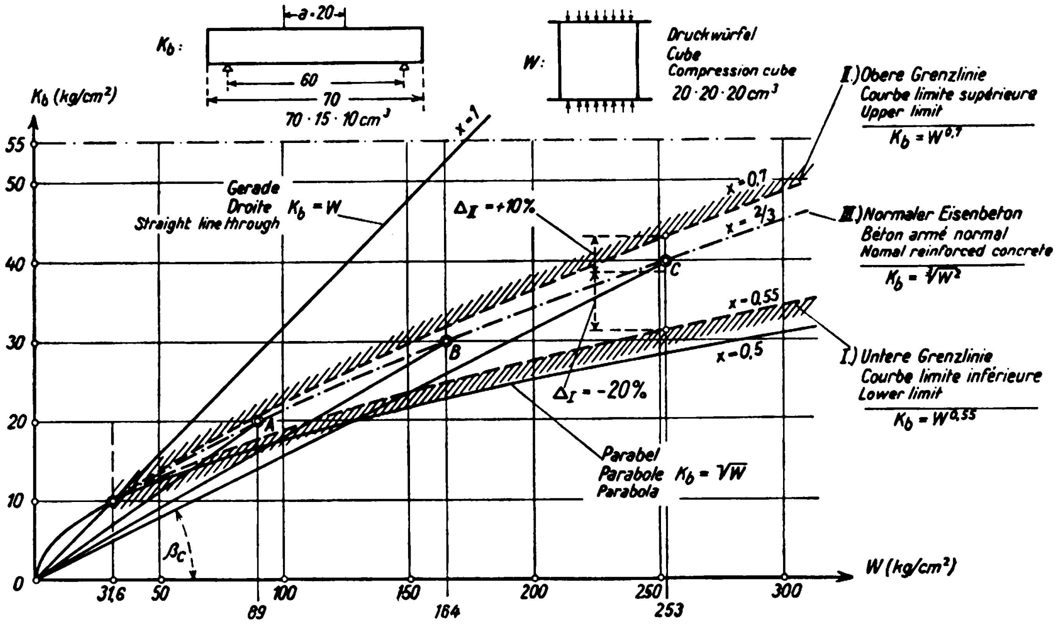


Fig. 4.

Relationship between the bending tensile strength K_b and the compressive strength W of concrete: $K_b = W^x$.

the higher will be the cube strength obtained. In the instructions laid down for the formation of roadway slabs on the Reichsautobahnen the proportion of length: width: thickness of the aggregate are prescribed as varying from 1: 1: 1 where the aggregate is of rounded shape, down to 1.0: 0.6: 0.2 where the shape is closely packed. Where concrete is worked in a wet condition the presence of chippings in the aggregate may have the effect of considerably reducing the tensile-bending strength. In order to increase the tensile strength a recent requirement is that the proportion of coarse sizes, above 7 mm in diameter, shall be graded by the employment of suitable sieves, even though from the point of view of compressive strength this measure is not so important.

The three usual qualities of concrete which give minimum cube strengths of 120, 160 and 225 kg/cm² (1707, 2276 and 3200 lbs. per sq. in) respectively showed average bending-tensile strengths of 20, 30 and 40 kg/cm² (285, 427 and 569 lbs. per sq. in), or, in the most favourable instance, 55 kg/cm² (782 lbs. per sq. in). Where, however, reinforced concrete pieces are being produced under factory conditions, it is possible by the use of special methods such as vibration to attain a considerably greater density, attended by bending tensile strengths up to 80 or 120 kg/cm² (1138 or 1707 lbs. per sq. in).

2) In the paper by *Colonetti* of Turin it is shown by calculations of strength that the grip is made much more dependable by the use of reinforcing bars of small diameter than by the use of a smaller number of larger bars.

3) The paper by *F. G. Thomas*, of England, deals with the development of cracks in reinforced concrete. It was shown by means of careful measurements of shrinkage stresses in concrete specimens that the risk of such cracks deve-

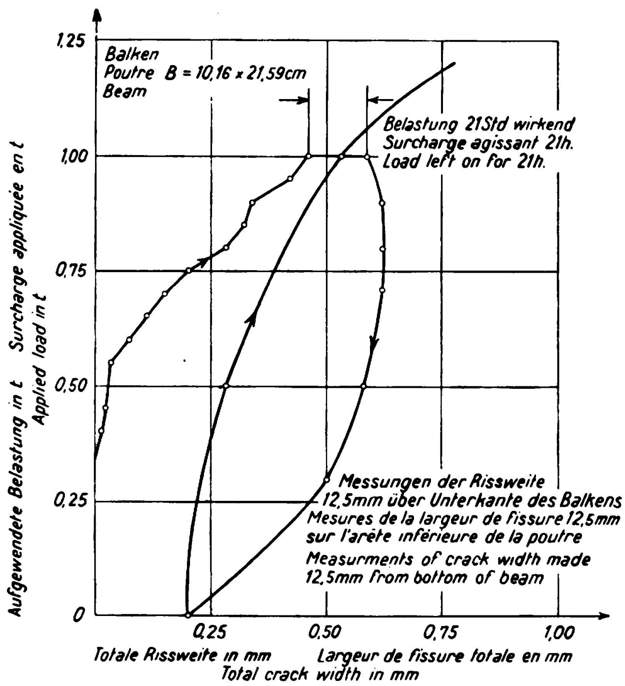


Fig. 5.

Measurement of width of cracks by F. G, Thomas.

loping increases according to the rate of hardening of the cement. In the diagram, Fig. 5, the load is shown plotted vertically and the width of the cracks plotted horizontally. It will be seen that if the load is reduced to zero the width of the cracks becomes less, but not, as frequently assumed, in direct proportion, the relationship being that indicated by a strongly bent curve; if, for instance, the load was reduced by one half no change at all could be detected in the width of cracks. The complicated problem of relating the width of cracks to the tension in the steel under increasing and permanent loading is exhaustively studied in the paper. One notable result obtained is a confirmation of the fact observed by Professor Duff Abrams that very fine cracks existing in concrete may, in the course of time, completely heal up; this occurs not only where the specimens are stored under water but even where they are stored in air.

4) The paper by *Freyssinet*, in its present form, constitutes the third section of the book published by that author in the same year as the Congress, which has already been widely discussed: it may, therefore, be expedient to give here a brief summary of the main principles treated in the first two parts of the book.

Improvement in the materials employed must be a matter of decisive importance for the further development of reinforced concrete construction, and this is true not only as regards what may be called the raw materials — the cement, aggregate and steel — but also as regards the methods of preparing and using concrete. A parallel to this is to be seen in steel construction where, at the present time, much attention is being paid to the further improvement of high tensile structural steel with special reference to welding and to fatigue effects. In his new book, "Une Révolution dans les Techniques du Béton" [Léon Eyrolles, Paris 1936] *Freyssinet* has performed the service of putting forward a thermodynamical basis for explaining the preparation of concrete, in the shape of a hypothesis from which he has deduced a large number of conclusions that have a bearing on the improvement of concrete.

At the time of the Bridge Congress in Vienna, *Freyssinet* showed from his experience in the construction of the bridge at Plougastel in 1928 that the law governing elastic deformation and the law governing non-reversible plastic deformation were altered when reinforced concrete members were stored under load in the open air so as to be exposed to the action of heat and humidity. He divided the total amount of shrinkage into two portions, one depending on warmth and dampness and a portion called the water-elasticity part. It is a question, therefore, of properties of material which are functions of the time T, of the temperature t, of the relative humidity of the air ε, and, especially, of any compressive stress that may have been preimposed. By the application of known principles of thermo-dynamics it becomes possible to establish certain fundamental equations:

A) I) The term "pseudo-solid" is applied to a material such as cement or concrete which appears outwardly to be a solid but which actually consists of a network of very fine pores containing water and air, the presence of which confers upon it mechanical properties different from those of a true solid or dense body. It is proposed to derive a mathematical expression for the condition under which the water will evaporate out of these capillary pores.

II) *The capillary phenomenon of the meniscus in the saturated pores of such a body.* According to Laplace the surface tension in the meniscus is given by:

$$\pi = A \cdot \left(\frac{1}{R_1} + \frac{1}{R_2}\right)$$

where R_1 and R_2 represent the principal radii of curvature of the meniscus. In the case of a very small interstice bounded by parallel walls of width D, putting $R_2 = \infty$ and $R_1 = \frac{1}{2} D$ we obtain:

$$\pi = \frac{2A}{D}. \qquad (1)$$

III) *Influence of the humidity of the air contained in the pores.* According to *Carnot* and *Lord Kelvin*, a second relationship may be derived from the conditions of equilibrium existing between the capillary stress in the meniscus and the relative humidity of the surrounding atmosphere, the ratio between the vapour pressure at any given level and at zero level being denoted by ε. For water at 15^0 C the equation is

$$\pi = \frac{2\,A}{D} = 1300 \log e \frac{1}{\varepsilon} \tag{2}$$

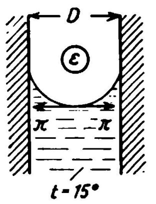
or

$$\frac{1}{\varepsilon} = e^{\left(\frac{2\,A}{1300\,D}\right)} \tag{2a}$$

Here A denotes what is known as the capillary constant, the value of which can be determined by experiment and may amount to, for instance, A = 8 mg/mm. It should also be noted that the width of pores as calculated by this formula to correspond with a condition of equilibrium when evaporation is taking place under a relative humidity of ε = 20 to 95 % is extremely small, being of the order of one to 25 millionths of a millimetre or between three and 100 times the diameter of a molecule of water, which measures 0.26 millionths of a millimeter.

IV) *Hygrometric equilibrium in a pseudo-solid.* In accordance with Equation (2) the hygrometric condition of a body may be stated in terms of the ultimate surface tension π_ε of the meniscus, or of the limiting width $D = D_\varepsilon$ at which evaporation is in equilibrium, or, finally, of the relative degree humidity ε. In the limiting case for an atmosphere saturated with water vapour we have $\varepsilon = 1$; hence $D = D_\varepsilon = \infty$ and $\pi = 0$.

Fig. 6: Notation used by M. Freyssinet:



t = temperature in °C.
T = time.
π = tension at the surface of the meniscus of a pore filled with water (in kg/mm^2).
A = capillary constant.
R_1 et R_2 = principal radii of curvature of the meniscus.
D = width of a lamelar interstice with parallel walls (in millionths of a mm).
D_ε = limiting thickness of pores (Definition: when $D > D_\varepsilon$ the water vanishes from the pore).
ε = relative humidity in %, or ratio of vapour pressures between a given level and zero level.
H_1 = vapour pressure in concrete saturated with water.
H_{max} = saturation pressure in the pores of the concrete at the temperature of the experiment t_1.

Pseudo-solid body (cement, concrete) = 1) Externally solid, 2) Internally a network of infinitely small pores filled with air *or* water.

Principles: molecular theory. Velocity of gaseous molecules.

It has been shown by the experiments of *Berthelot* and *Laplace* that a liquid in an air tight container is capable of retaining a considerable surface tension provided there are no air bubbles in the liquid; such tension may in fact amount to several tonnes per sq. cm. Thus the tensions in the meniscus are in equilibrium and the evaporation also is in equilibrium. If, then, a pseudo-solid such as a piece of concrete is placed in an atmosphere saturated with water vapour the small channels will become completely filled, with water only if the surrounding air is at a higher temperature. In the case of concrete, subject to the effect of setting heat, the contrary is always true and the temperature of the concrete t_1 is higher than the temperature t_2 of the surrounding air. It may then be supposed that the concrete will assume a moisture condition ε equal to the ratio of the vapour pressures H_1 and H_{max}, and we have:

$$\varepsilon = H_1 : H_{max} \leqq 1 \tag{3}$$

where H_1 is the vapour pressure of the atmosphere in the pores of the sample and H_{max} is the saturation pressure, at temperature t_1,for the liquid contained in the pores of the concrete. The liquid will, therefore, disappear from all those pores having a diameter D greater [see Equation (2)] than the limiting diameter

$$D_\varepsilon = \frac{2\,A}{1300 \cdot \log_e \frac{1}{\varepsilon}} = \frac{2\,A}{1300 \log_e \frac{H_{max}}{H_1}}. \tag{4}$$

In Fig. 7, Equations (1) and (2) have been represented for $A = 8$ mg/mm using a system of three coordinates π_ε, D_ε and ε so as to give a space-curve. ABC for the relationship $f(\pi_\varepsilon, D_\varepsilon, \varepsilon) = 0$. This representation serves to illustrate the three gradations which are of capital importance in considering the effect of climate on reinforced concrete:

Stage I: Continental climate, dry and keen with $\varepsilon = 20\,\%$ (very small), $\pi_\varepsilon = 2100$ kg/cm² (very large).

Stage II: Semi-Continental climate with $\varepsilon = 60\,\%$ and $\pi_\varepsilon = 665$ kg/cm² (average value).

Stage III: Maritime climate, very damp and mild, with $\varepsilon = 95\,\%$ and $\pi_\varepsilon = 65$ kg/cm².

If, for instance, in a concrete of high water content we have $\varepsilon = H_1 : H_{max} = 0.9$ the pores larger than $D_\varepsilon = 11.4$ millionths of a millimeter will be completely dried out for this value of the coefficient of cohesion A; on the other hand for $\varepsilon = 0.5$ we obtain $D_\varepsilon = 5.5$ millionths of a millimeter so that a much larger number of pores will completely dry-out. The smaller the limiting value of D_ε found in accordance with Equation (4) and the greater the number of pores drying out having a diameter $D > D_\varepsilon$ in any given distribution, the faster, in proportion, the whole test piece will dry-out. There is a risk, therefore, of insufficient water for setting. Thus an aluminous cement may easily dry out under water, its great setting heat operating to accelerate the process of drying.

V) *Calculation of shrinkage.* Consider a cross section of 1 cm² of a pseudo-solid body and assume that this contains a portion ω_p of solid matter in addition to a portion ω_s of voids and a portion ω_m of space filled with water, or partially so filled. The surface tension in the meniscus will then be represented by a force

$$P = \pi \cdot \omega_m$$

and, considering two such sections, the forces P and P′ will cause a contraction equal to

$$\delta = \frac{P}{1\,E_1} = \frac{\pi \cdot \omega_m}{E_1} = \left(\frac{\omega_m}{E_1}\right)\left(1300 \log_e \frac{1}{\varepsilon}\right) = a \cdot b$$

where P is known as the „shrinkage pressure“ and E, as the contraction modulus which amounts to 10^9 kg/cm².

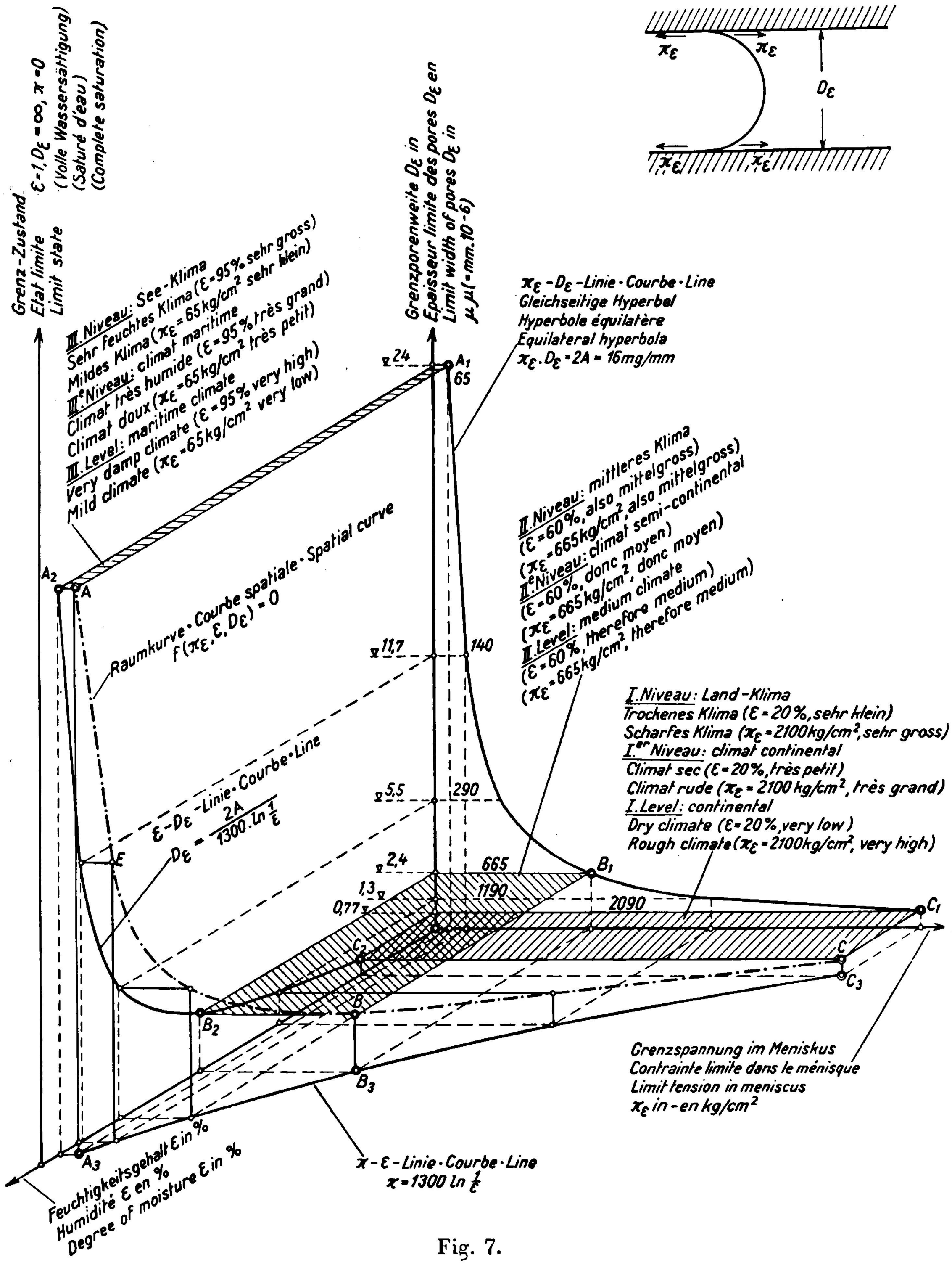


Fig. 7.

The total shrinkage is made up of two components: the hydro-elastic factor $a = \frac{\omega_m}{E_1}$ and the thermo-hygrometric factor $b = 1300 \log_e \frac{1}{\varepsilon}$. (See also *Freyssinet*, Vienna Congress, 1930; *Gehler*, Zurich Congress, I.A.T.M., 1932, page 1118; also *Gehler* and *Amos*, 1934, German Comittee on Reinforced Concrete, Publication N° 78.)

The only difference between shrinkage and creep is that the former occurs in consequence of a permanent external load whereas the latter takes place under dead load alone.

VI) The suggestion is made that the considerations arising from *Freyssinet*'s hypothesis, and their practical implications for improving the quality of concrete, should be checked by experiment.

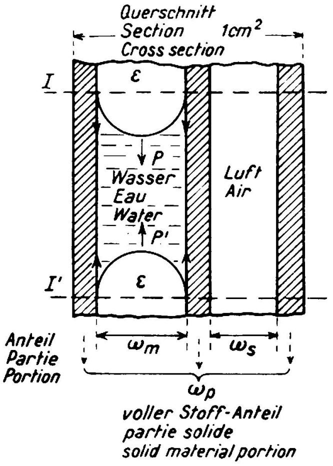


Fig. 8.

B) The present paper by *Freyssinet* contains a few indications of ways in which the foregoing might be applied in practice, derived from the third part of his book. *Freyssinet* envisages three methods of improving the quality of concrete work: the use of a very dense concrete, the adoption of prestressing as a means of ensuring that in any given cross section of a beam in bending there shall occur (as far as possible) only compressive stresses in the concrete, and, finally, the application of heating, perhaps by means of steam. He carried out experiments on concrete poles under repeated alternating stress and obtained extraordinarily high strengths. Railway sleepers subjected to a pressure at the time of casting of 100 to 300 kg/cm² (1422 to 4267 lbs. per sq. in) gave cube strengths as high as 1000 kg/cm² (14 220 lbs. per sq. in), with a perfectly hard smooth surface to the concrete. The paper contains a detailed description of the very difficult construction work involved in strengthening the port station at Le Havre wherein the method of pre-stressing followed by rapid hardening of the concrete was used with great success. *Freyssinet* expresses the opinion that this method will prove particularly useful for application to girders of large spans, and also to mushroom floors and to roadway slabs.

In this connection attention may be drawn to the paper by *Dischinger* of Berlin, presented at the fifth meeting of the Congress, and dealing with the question of long span bridges. This paper puts forward what appears, in principle, a sound solution to the problem of constructing long span bridges. Following the analogy of an arch bridge with a pre-stressed tie it is proposed to make use of a suspension chain under the beam composed of round steel bars of 60 to 100 mm diameter which will be structurally quite separate from the compression boom. The concrete compression member will receive only concentric stresses due to the dead weight of the bridge itself, and the steel will be pre-stressed to such an extent that when the shuttering is struck the whole system will conform as closely as possible to the proper intended geometrical shape. The reinforced concrete box section will be subjected to bending only on account of the live load. The effects of shrinkage and creep are to be taken up by subsequent further tightening of the suspended rod-system. It should be possible in this way to build girder bridges up to 100 or 150 m in single span.

This remarkable and indeed revolutionary proposal must give rise to numerous questions in the minds of both designers and constructors. One obvious question will be in regard to the nature of the steel to be used. *Freyssinet* has hitherto made use of round bars of up to 16 mm diameter, having the elastic limit raised by cold stretching from 24 to 80 kg/mm² (from 34,140 to 113,790 lbs. per sq. in). For the purpose of these long span concrete girder bridges *Dischinger* envisages round steel rods up to 10 cm diameter and 100 m in length, the joints in which would be formed by electric resistance welding. This suggests further problems from both the welding and the metallurgical points of view.

Part III. Application of high tensile steel.

1) The paper by *Gehler* of Dresden makes reference to the following conclusions emerging from the exhaustive experiments of the German Committee for Reinforced Concrete:

The introduction of high tensile steel into reinforced concrete practice has entirely fulfilled the expectations. The advantages, as indicated in Table I, include an increase in the permissible stress of the steel from 1200 kg/cm² (17,000 lbs. per sq. in) for commercial St. 37 to as much as 1800 kg/cm² (25,600 lbs. per sq. in) according to the elasticity and quality of the concrete, and in exceptional cases 2200 kg/cm² (31,290 lbs. per sq. in). In the case of tee beams carrying mainly stationary load and reinforced with St. 52, stressed up to 1800 kg/cm², the margin of safety against cracking is the same as by using St. 37 stressed to 1200 kg/cm², assuming a minimum cube strength of 225 kg/cm² (3200 lbs. per sq. in) in the concrete in both instances. In slabs of rectangular cross section reinforced with St. 52 and showing a cube strength of 225 kg/cm² the permissible strength in the steel may be raised to 1800 kg/cm² even under moving loads, but in tee beams only to 1500 kg/cm² (21,340 lbs. per sq. in).

These increases in the permissible stresses which may be realised where high grade concrete and high tensile steel are used result in a number of advantages: for instance, in a reduction of the cross section of steel necessary in the tensile boom of a girder, and therefore of its total width, and of its weight. Further, the disadvantage of having a large number of bent bars crowded together in the cross section over the supports is reduced to a minimum.

In the experiments at Dresden the cracking was recorded photographically using a magnification of 23, thus enabling the widths to be accurately measured, and the depths were also ascertained. In this way the question of safety against cracking has been placed on a sound scientific basis.

The criterion of safety against cracking, namely the ratio between the load at the time when the first crack appears and the load actually arising in service, amounts to 1.8 in the case of slabs supported along all four edges and reinforced

Paper I.

Table of permissible stresses in reinforcements of high yield points for reinforced concrete slabs and beams.

1	2	3	4	5	6	7	8
Manufacturer No.	Type of Steel	Minimum yield point[1]	Minimum elongation at fracture	Minimum cube strength of concrete	in slabs σ_e perm	in T-beams σ_e perm	Range of validity
	—	kg/cm^2	%	kg/cm^2	kg/cm^2	kg/cm^2	—
1	St. 52	3600	20	120 225	1500 *1500*	1200 *1500*	Also movable loads[3]
2	St. 52	3600	20	120 160 225	1500 1800 1800	1200 1200 1500[4] 1800[5]	
3	Special steel[2]	3600	14[6]	120 160 225	1200 1800 1800	1200 1200 1500[4] 1800[5]	Mainly stationary loading and in building frames not exposed to the weather.
4	Special steel[2]	5000	14[7]	120 160 225	1200 2200 2200	1200 1200 1500[4] 1800[5]	

[1] *Yield points.* In accordance with the reinforced concrete regulations, Section 7, account must be taken of the properties of the steel. In the case of reinforcements which have no definite yield points this may be taken as 0.4 % of the limit of total elongation instead of at 0.2 % as laid down in DIN 1602, pending the final decision of the question on the basis of experiments now in hand.

[2] Special steel arranged in a particular way in accordance with permission granted by the building authorities.

[3] Corresponds with the regulations hitherto in force.

[4] When the cross section of each reinforcing bar exceeds 3.14 cm^2 (in the case of twisted steel the total cross section of the twisted bar is to be taken).

[5] When the cross section of each reinforcing bar is not greater than 3.14 cm^2 (otherwise as for 23).

[6] In the case of slabs a steel having a minimum elongation at fracture of 10 % is also permissible.

[7] In the case of slabs a steel having a minimum elongation at fracture at 8 % is also permissible.

crosswise; to 1.4 in that of slabs supported at the four corners (preliminary experiments on mushroom floors); to 0.75 in that of slabs reinforced only in one direction and to 0.5 in the case of tee beams. It is, therefore, for use in slabs that high tensile steel is most emphatically to be recommended. Safety against cracking increases also with improvement in quality of the concrete, but unfortunately only to a small extent on account of the greater brittleness of high grade cement. The purely statistical conclusions reached from these experiments suggest the physical interpretation that when a crack occurs the cross section thereby broken (depth t, width of rib b_o) ceases to act, and the tension in the concrete which previously existed therein disappears. Its magnitude may amount to 4, 8 or 12 % of the tension in the steel at the moment of formation of the crack, according as the quality of the concrete is poor, medium or good. In the case of tee beams subject mainly to stationary loading and reinforced with St. 52, allowing a stress of 1800 kg/cm² in the steel and assuming the use of high quality concrete, the degree of safety against cracking will be the same as if St. 37 were used with a stress of 1200 kg/cm², embedded in ordinary concrete.

As regards the form of cross section of reinforced concrete beams it may be inferred, from the Dresden experiments, that reinforced concrete members produced under factory conditions for use in long span bridges may, with advantage, be given an I-shaped or box-shaped cross section, from the point of view both of freedom from cracking and of carrying capacity.

2) The paper by *Saliger* of Vienna refers in the first place to columns with high tensile steel reinforcement, and leads to the surprising conclusion that as regards columns having both longitudinal and lateral bindings the law of superposition (which has been so much under discussion) does not hold good where high tensile steel is used. This is attributed to the fact that the compression undergone by the concrete on fracture is not as great as the reinforcing bars themselves would permit, with the consequence that the concrete suffers disturbance earlier than it should be from the buckling of the longitudinal bars. It is only in the case of columns provided with spiral binding that the requisite amount of contraction before breakage is attainable, so as generally to justify the use of high tensile steel. For columns of this type, the author gives a formula based on his experiments, which corresponds to the law of superposition ordinarily used.

The second part of this paper by *Saliger* deals with beams having high tensile reinforcement, and confirms the indications given in the paper by *Gehler* as regards the parabolic nature of the relationship existing between the stress in the steel when the first crack appears and the percentage of reinforcement. The method of calculations has already been partly dealt with in Part I.

3) *Dr. Olsen* of Munich, who proposes to refer to this matter in the course of the discussion, has already published an account of numerous experiments in his book which appeared in 1923 under the title „Über den Sicherheitsgrad von hoch beanspruchten Eisenbetonkonstruktionen". The results which he then obtained on concrete beams reinforced with steel bars agree very well with those of the Dresden experiments so far as concerns the relationship between percentage of reinforcement, quality of concrete and safety against cracking. These experi-

ments by *Olsen* also indicate, that where high tensile steel is used, the amount that the stress in the steel exceeds the limit of elasticity at the moment of breakage increases with the compressive strength of the concrete. One particularly notable point which he has established is that when a strength of 100 kg/cm² in the concrete and 2000 kg/cm² in the steel was assumed and concrete was used having a cube strength of 250 kg/cm² with steel having a yield point of 4000 kg/cm², it was possible to maintain at least a factor of safety of 2 against rupture.

4) The paper by *Brebera* of Czechoslovakia discusses experiments carried out on two of the steels most used in that country, namely "Roxor" and "Isteg", and gives an account of some notable applications.

Part IV. Effect of construction and expansion joints.

In the paper by *Baravalle* of Vienna it is recommended that the principle followed in Lamellae construction for arches should be applied also in other structural work such as floors, water tanks, etc.: that is to say joints should be provided which will remain open only during the constructional period or for some weeks at least, before being filled up with concrete, these being additional to the permanent expansion or contraction joints which serve to separate one part of the structure from another and allow the necessary freedom of breathing.